News letter

Summer 2010

Special Investigation Techniques edition

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Colophon

Ingeokring, founded in 1974, is the Dutch association of engineering geologists. It is the largest section of KNGMG (Royal Geological and Mining Society of The Netherlands). Ingeokring also forms the Netherlands National Group of the International Association for Engineering Geology and the environment (IAEG).

With over 200 members working in different organisations, ranging from universities and research institutes to contractors, from consultancy firms to various governmental organizations, Ingeokring plays a vital role in the communication between engineering geologists in The Netherlands.

The objective of the Newsletter is to inform members of the Ingeokring and other interested parties about topics related to engineering geology, varying from detailed articles, book reviews and student affairs to announcements of the Ingeokring and current developments in the field of engineering geology. The Newsletter wants to make engineering geology better known by improving the understanding of the different aspects of engineering geology.

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Issue

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Cover photo

Beautiful Dutch polder landscape near Reeuwijk.

Photo taken by Sanne Brinkman on October 15, 2009 at one of the stops of the Ingeokring excursion to Reeuwijk. A report of this excursion is given in this Newsletter.

Guidelines for authors of Newsletter articles and information about advertising in the Newsletter can be found at the inside of the back cover.

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Editorial

Erik Schoute

Dear reader,

I am pleased to announce on behalf of the editorial board this latest issue of the Ingeokring Newsletter. This edition has as theme 'Special Investigation Techniques'. A year has passed since the previous Newsletter saw daylight. In the meantime, two new members were welcomed into the editorial board: Leon van Paassen, who with his Deltares background and in his present function as lecturer/researcher at the Geo-Engineering section of TU Delft is very familiar with all aspects of engineering geology, and Jan-Willem Vink, who has replaced Werner van Hemert as representative of 'De Ondergrondse', the Geo-Engineering student chapter. As it is always a challenge to get the next Newsletter out on time, the editorial board was ambitious enough to put the deadline to receive the contents for this issue in December 2009 (so, a mere six months after the last Newsletter came out). As we had only received a few articles around Christmas, it became clear that this target was indeed too ambitious and so it became a floating deadline. Fortunately, throughout the first few months of 2010 many articles were sent in (and about just as many were cancelled for various reasons), and companies could be convinced of the fact that with sponsoring the Newsletter by means of a professional (full-colour) advert new audiences and future work forces can be reached for a small amount of money.

The core of this Newsletter is formed by a trilogy of the senior member of our editorial team, Michiel Maurenbrecher. He is a constant factor throughout the history of Ingeokring Newsletters. There haven't been many issues without an interesting contribution written by Michiel. The three papers in this issue deal with site investigation for a building site in The Hague where problems arose when pile driving took place. Lessons learned from this case are that one should always perform 'special (site) investigation techniques' when dealing with these types of projects in an urban environment, what in this case could have been a search in the municipal archives and an extensive geological survey of the area surrounding the plot.

A truly special investigation technique is described in a paper by Deltares, namely the video cone. This device can be used to establish geotechnical properties in situ by literally looking into the soil using a camera. Other articles in this Newsletter that can be regarded as special investigation techniques, either site investigation techniques or laboratory techniques, are a paper about the Equotip Hardness Tester written by another Newsletter regular, Peter Verhoef, and a case study by Demco NV on application of a resistivity survey for a pipeline project. Three other contributions by companies active in the geotechnical field are dealing with site investigations for offshore wind farms (RPS/Global Geologic), determination of soil stiffness parameters (Lankelma), and the application of several sampling and testing types for marine site investigations (Marine Sampling Holland). Further, Wim Verwaal has contributed with an interesting article about a special high tech piece of equipment in use at TU Delft: the nanotom CT-scanner. Don't forget to check out in this article the magnificent images of foraminifera that were produced with this scanner. Anneke Hommels' paper deals with a sophisticated approach to the analysis of a road embankment using the Ensemble Kalman Filter, and Arjan Venmans, together with Dominique Ngan-Tillard, has contributed with a paper on site investigation for linear infrastructure on soft soils and a new approach to subsurface modelling in particular.

And the Newsletter wouldn't be complete without the regular items: the *engineering geologist abroad* reporting in this issue is Michiel Zandbergen, who is working in the oil sands industry in Alberta, Canada. *'De Ondergrondse'* has contributed with a description of activities undertaken so far with the St. Petersburg study trip as highlight; two *Ingeokring activities* are reported on (Reeuwijk excursion and mini-symposium), and Professor Michael Hicks is asking the question why we should bother with statistics in the *Professor's Column* for this edition. Further, two *book reviews* are published in this issue, as well as *thesis abstracts* from recent TU Delft graduates.

Many thanks go out to our sponsors and to the contributors to this Newsletter. They have made it possible that this Newsletter is again filled with almost 90 pages of interesting articles and advertisements.

We hope you will enjoy reading this Newsletter!

Equotip Hardness Tester

Peter Verhoef *(Royal Boskalis Westminster nv, Papendrecht, The Netherlands)*

Introduction

The Equotip Hardness Tester (EHT) is a small electronic battery-operated device that is developed to measure the rebound hardness of metallic materials. A small 3 mm diameter spherical shaped tungsten carbide test tip is mounted in an impact body and impacts under a spring-loaded force against the test surface from which it rebounds (Figure 1). Impact and rebound velocities are determined by measurement of the induction currents in the coil, generated by the impact ball. These velocities are used to calculate the Equotip Hardness value *L* which is displayed on the processor:

$$L = \frac{V_{rebound}}{V_{impact}} \cdot 1000 \qquad (1)$$

There are several impact devices that can be used. The basic type is type D, which gives an impact energy of 11 Nmm. Type C gives an impact of 3 Nmm; type G of 90 Nmm.

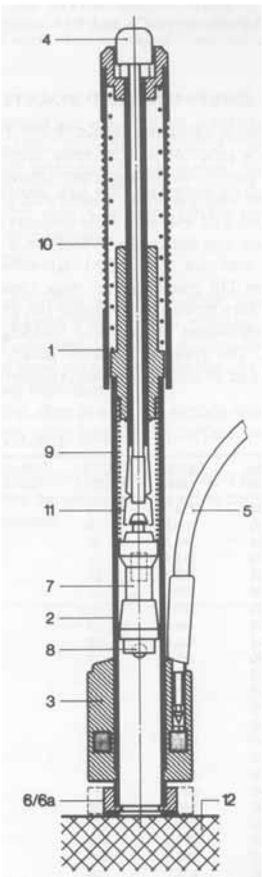

Fig. 1 Equotip Hardness Tester (Proseq SA, Switzerland)

(1) Loading tube, (2) guide tube, (3) coil with coil holder, (4) release button, (5) connection cable to processor, (6) support rings, (7) impact body, (8) spherical test tip, (9) impact spring, (10) loading spring, (11) catch chuck, (12) test material

The handy format of the Equotip Hardness Tester, which is similar to the Pocket Penetrometer used for estimating the compressive strength of cohesive soils, makes it a potential companion in site investigation studies involving rock. A useful application is while logging drilled rock cores, where at regular intervals the rebound hardness can be determined on the side of the cores.

The tester is normally used for quality control checks on steel, for example to monitor corrosion. When used on metal surfaces it is prescribed that EHT measurements should be performed on a flat surface with the device in vertical position. This is a potential drawback for the use of the tester on rock surfaces. While this testing situation can be obtained in a laboratory, in the field the rock surface will have micro roughness and core surfaces are curved. Verwaal & Mulder (1993) found however that variation in roughness of a rock surface did not have a significant effect on the measured L-value. This has encouraged the use of the tester on untreated rock surfaces. Very porous rock types are not suitable for the tester, since no proper rebound can be obtained.

The tester has been used for many years to measure rebound hardness of rock by the Engineering Geology Group of Delft University and ITC Delft, for the purpose of making estimates of rock strength. A large database was built up of laboratory test results on rocks that were sampled by students in a fieldwork area in Spain, where a range of rock types is present. Studies of the relationship of the Equotip L-value with rock properties have been published. In the order of increasing accuracy of prediction of Unconfined Compressive Strength (UCS), these are a simple linear regression relation with UCS (Verwaal & Mulder, 1993; Alvarez Grima & Babuska, 1999), a fuzzy model to derive UCS from L-value, rock density and rock porosity (Meulenkamp & Alvarez Grima, 1999), and a neural network model to derive UCS from L-value, rock type, density, porosity and grain size (Alvarez Grima & Babuska, 1999).

For a first estimate of UCS, the linear regression equations from the Delft rock database can be used. This database involves a range of rock types that have been sampled in the Falset area near Tarragona, Spain. The geological formations are of Carboniferous to Cretaceous age and include sandstones, limestones, dolomitic limestones, dolomites, granites and granodiorites.

For the type D impact device (11 Nmm) and 28 homogeneous samples the following correlation can be derived (Verwaal & Mulder, 1993):

$$UCS = 4.906 \cdot 10^{-7} \cdot L^{2.974} \tag{2}$$

And for the type C impact device (3 Nmm) and 226 samples (Alvarez Grima & Babuska, 1999):

$$UCS = 1.75 \cdot 10^{-9} \cdot L^{3.8} \tag{3}$$

When these equations are used one should realise that:

- The UCS estimate relates to *dry* rock strength; the wet or saturated rock strength may be lower. Without more information, the following can be applied: $UCS_{wet} \sim 0.7 \cdot UCS_{dry}$ (ratio varies from 0.3 to 1.0).
- The database has a large amount of stronger rocks (average UCS=120 MPa, range 5-280 MPa) (Alvarez Grima & Babuska, 1999).
- The regression equation predicts average values only, which implies that low UCS values are *overestimated* and high UCS values are *underestimated* (Alvarez Grima & Babuska, 1999).

The Equotip gives the rebound hardness of a very small volume of rock, whereas the UCS is determined on a much larger volume of intact rock. The rock database consists of mainly homogeneous rock, without flaws in the rock that could affect the UCS. When using the tester to measure rebound hardness on a rock specimen or core it is advised to measure using a regular grid or spacing. The average L-value determined in this way should cover a volume that is representative for the volume of the UCS core.

L-values measured on rock should be calibrated against UCS test results if available. The EHT test results can then be used to inform us on the *variation in strength* of the rock. This is the main advantage of the use of the EHT. The following example illustrates the potential of the tester.

Strength variations in Simsima Limestone (Ras Laffan, Qatar)

Simsima Limestone consists of several components. Main component is a primary yellow crystalline dolomitic component which represents a limestone layer that apparently has been partially dissolved in the past. The karstic holes have been filled-in with a silt size carbonate that has recrystallised to a grey crystalline dolomitic limestone. This secondary crystalline limestone may contain breccia fragments (*clasts*) of the primary yellow limestone.

Fig. 2 Core of Simsima Limestone. Clasts of yellow crystalline dolomitic limestone within grey crystalline dolomitic limestone

The rock is locally weathered, especially along fracture zones. Dissolution and weathering have mainly affected the grey component of the Simsima Limestone, which can completely disintegrate into grey carbonate silt.

During core logging as part of a Boskalis drilling campaign, the EHT was used to measure rebound hardness normal to the core axis at 2 cm intervals. In the laboratory the Equotip Hardness was measured on the flat end faces of UCS test cores.

While measuring in the laboratory, the yellow and grey components were distinguished and it was also noted when the measurement was taken from a weathered part of the rock. The following table summarises the results from the cores that were later used for UCS testing:

Table 1 Equotip Hardness of laboratory samples

L-value (D-tip)	Yellow limestone	Weathered yellow limestone (red stained - other)		Grey limestone	Weathered grey limestone
Max	816	701	791	771	601
Min	337	184	139	157	117
Median	684	592	471	600	378
Average	664	543	461	578	373
St. dev.	87	145	140	115	115
COV %	13	27	30	20	31
n	277	74	177	146	102
n > 600	230	36	35	72	1
%n > 600	83	49	20	49	1

The Equotip results show that the yellow limestone component is harder than the grey component and is also less affected by weathering.

Using the regression equation for the type D impact device (2), the estimated UCS_{dry} values are:

Table 2 Equotip UCS-L values of laboratory samples

UCS-L (MPa)	Yellow limestone	Weathered yellow limestone (red stained - other)		Grey limestone	Weathered grey limestone
Max	224	143	204	189	90
Min	16	3	1	2	1
Median	132	86	44	90	23
Average	121	67	41	80	22
St. dev.	16	9	5	10	3
COV %	13	27	30	20	31
n	277	74	177	146	102
n > 90	230	36	35	72	1
%n > 90	83	49	20	49	1

These estimated UCS-L values are high compared with the UCS values found in the laboratory (Figure 3: UCS~0.3·UCS-L). The following points explain this discrepancy:

- The correlation equation of Verwaal & Mulder (1993) is based on UCS tests carried out on dry, homogeneous rock. The Simsima Limestone rock samples on the contrary were tested saturated.
- It is known that saturated rocks have a UCS which is lower than the dry value (UCS_{wet} is commonly of the order of $0.7 \cdot UCS_{dry}$).
- Simsima Limestone is a rock that consists of components of different strength, the result of UCS tests on rock cores in the laboratory depend on the distribution of these components in the rock. Other factors that influence the outcome of UCS tests are the amount and position of voids and micro-fractures in the rock.

The Equotip measurements that were done on the cores during logging in the field show a similar wide variation. However, in this case hardness measurements were not performed on the flat end surfaces of test cores, as was done in the laboratory, but on the rough sides of the freshly drilled cores. A bias of the measurements towards low values is expected in this case.

The results of Equotip measurements on the cores are illustrated by the example of borehole Boka 2 in Figure 4. For illustrative purposes the plot shows the Equotip results as UCS-L value derived from the Verwaal & Mulder (1993) regression equation (2). This was done because the UCS-L values tend to be 3 to 4 times higher than the measured UCS values and therefore do not overlap with the UCS values on graphic plots. The values vary from extremely low to extremely high, reflecting measurements on weathered or relatively fresh spots of yellow or grey dolomitic limestone. For each 0.5 m core length the average UCS-L value over this

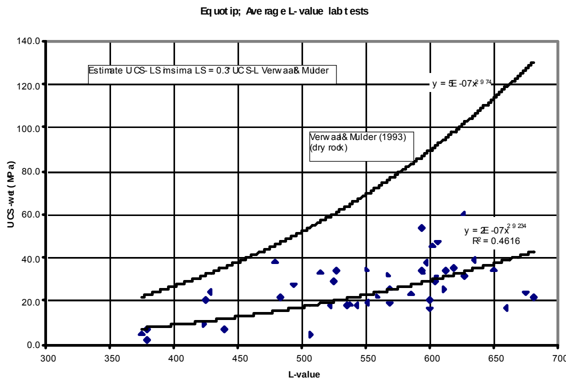

Fig. 3 Plot of UCS_{wet} against average L-value. The Equotip L-values from the end faces of UCS cores do not relate well to UCS values

length is plotted as well. The variation in average UCS-L relates to the UCS test results on the cores, as shown by the parallel fluctuation of the lines in Figure 4. Apart from the fact that the Equotip measurements give an idea of the spreading in strength and hardness on the scale of 2 cm, the average value over 50 cm core length is apparently related to the UCS of test specimens (which have a length of 15 cm). Using the regression equation for the tested rocks given in Figure 3, Boka 2 Equotip data gives a range of predicted UCS values from 0-52 MPa with an average of 22 MPa (Figure 4). The nine UCS tests give values ranging from 19.3-58.5 MPa and have an average of 36.6 MPa.

In the other boreholes of the same drilling campaign, the Equotip Hardness showed a similar close variation with the UCS value of tested samples and gave therefore a reliable insight in the strength variation within the Simsima Limestone unit.

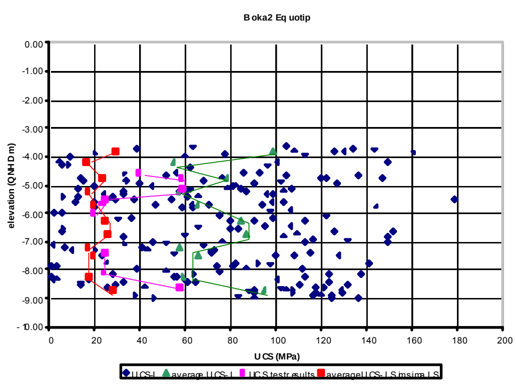

Fig. 4 Borehole Boka 2: results of Equotip and UCS tests

Conclusions on the use of the Equotip hardness tester in this type of rock:

- The *average* Equotip L-value over a certain core length (here 0.5 m) relates well to the UCS.
- Variation in strength is recorded well by the average Equotip L-value.

Despite the large variation in hardness measured in Simsima Limestone, the variation in average scaled to a certain volume of rock gives useful information on rockstrength.

Note that the *spread* (range) of Equotip L-values gives useful information as well. It gives an indication of the degree of homogeneity of the rock on the scale of measurement (in this case every 2 cm in the direction of the core axis). The range of values is very large: within the rock volume a large variation of strength around the average value occurs, from weak to very strong. In other words, Simsima Limestone is a rock type with both soft and hard components. This information can be interpreted in terms of expected cutting behaviour of the rock:

- The presence of weak material deviating much from the average strength of the rock points to possible ductile cutting behaviour.
- The presence of very strong material deviating much from the average strength of the rock may have implications for the rate of tool wear.

Use of the Equotip Hardness Tester

The tester can be used during drilled rock core inspections, to obtain an impression of the strength variation of the rocks, both on the scale of UCS samples and the vertical variation along the core length.

If the L-value is calibrated on rock specimens which have been tested (UCS samples or samples used for Point Load tests or Brazilian Tensile Strength tests), the Equotip can be used to measure the variation of strength on existing cores. In this way the amount of data on strength variation of the rock can be increased, which can be valuable for projects with lack of data.

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Engineering geologist abroad

Michiel Zandbergen *(Geotechnical Engineer, Klohn Crippen Berger, Canada)*

December 4, 2008 was the day that my partner and I arrived in Calgary, Canada. It was a nice evening, with few clouds and a temperature of -4 °C. We were picked up by the HR manager of my new company, Klohn Crippen Berger. She was kind enough to help us get to our temporary apartment and even arranged two bags of groceries for us to start our Canadian adventure.

Calgary is situated in the province of Alberta at an elevation of approximately 1,050 m above sea level, about 80 km east of the Canadian Rockies and about 230 km north of the USA-Canada border. The weather is always interesting: except for July and August, every month of the year has seen snow (as we also experienced last June). There is actually a saying which captures Calgary's weather perfectly: "If you don't like the weather, wait 5 minutes". A major advantage compared to The Netherlands is the average of 2,400 hours of sunshine per year (The Netherlands: about 1,600 hours) and the average precipitation of 410 mm per year is also easy to live with (The Netherlands gets around 800 mm).

Downtown Calgary with the Saddledome and the Calgary Tower

On the other hand, heated seats in a car are quite useful at -35 °C (not to mention the winter tires). One phenomenon is worth looking up: Chinook winds. They can cause the temperature to increase to levels well above freezing within several hours during mid winter. Especially the occasionally accompanying Chinook arc can be a very spectacular sight when the sun is going down at the end of the day (for those interested, please type in 'Chinook wind' at Wikipedia).

As an engineering geologist I have always been interested in mountainous areas, and since my studies at Delft I wanted to live and work near or in a mountainous area at least once in my life. Calgary therefore is a perfect location to live, as with a mere 60 minute drive you can be climbing peaks 2,500 m and higher, go skiing, snowboarding, cross country skiing and/or snowshoeing. As you can imagine, I'm quite happy here in Alberta.

Winter wonderland

Other main attractions in Calgary are the 'Calgary Stampede' or 'The greatest outdoor show on earth' as Calgarians like to call it. This 10 day long spectacle features bull riding, horseback riding, chuck wagon races, a big fair, exhibits, numerous concerts and great fun. We are definitely going again next year!

As mentioned previously, my new employer is Klohn Crippen Berger Ltd. 'Klohn' is an international consulting company with about 350 employees and offices in Vancouver, Calgary, Edmonton, Lima (Peru) and Brisbane (Australia). The majority of the employees are either engineers or (hydro-) geologists. Here in Alberta we mainly support the oil and gas industry with designs for, and monitoring of, their tailings operations. One of our largest clients is Shell Albian Sands. Shell is developing and operating two mines in Alberta's oil sands region. One mine (Jackpine Mine) is still being developed; the other mine (Muskeg River Mine) is already producing oil for some time.

Oil sand mining

My work is concentrated on the Jackpine Mine project. As the design of the initial starter dykes is virtually complete, I'm mainly focussing on the construction management of these starter dykes. Starter dykes are constructed to facilitate the start-up of the Jackpine Mine. After start-up, sand is used to build up the tailing dams to full height (a process called *celling*). 'Cell' sand becomes available after separation of the bitumen from the oil sands. The sands are hydraulically placed on the various tailings dams or dykes, similar to reclamation works in the dredging industry. The clay and silt size particles that are also part of the oil sands are obviously also separated from the bitumen and thickened in cyclones. The product that is left is referred to as Thickened Tailings. These materials are stored in the tailings pond.

Since tailings dams can be up to 80 m high, a very detailed soil investigation together with hydrological, geotechnical and geological modelling and staged slopes stability analyses are required. Similar to The Netherlands, peat and clay are found commonly in the subsurface. The surface is often covered with a peat layer called *Muskeg*, which is difficult to build and travel on. It is therefore that soil investigations are preferably done in the winter season, as the subsurface will freeze up and become trafficable. Obviously, constructing on a frozen subsurface is much easier than on a 'wet sponge'.

The main difference with regards to the clays found in northern Alberta compared to those in the upper surface layers in The Netherlands is that they are heavily over-consolidated due to past glaciers. These (often fissured) over-consolidated clays form a major challenge during construction. This because these clays commonly form shear zones and pose significant stability issues for tailings dams. Shear keys are often used as soil improvement to increase slope stability safety factors.

The challenge in the coming months will be widening and completing the starter dykes to full width and operating an oil sands mine at the same time.

The Aquares Resistivity System, a marine geological exploration method

Case study: Tema pipeline route survey

Peteralv Brabers *(Demco NV)*

Koen de Grave *(PMI Ltd.)*

Abstract

An Aquares resistivity survey was carried out for a pipeline landfall near a refinery in Tema, Ghana. The entire area from the beach through the surf zone and further out was surveyed. A complete 3D model of the subsurface was constructed showing volumes of mud, sand and rockheads. Based on the resistivity results, pipeline trenching operations could be planned and optimised.

Introduction

In the past, acoustical geophysical methods and more in particular sub-bottom profiling methods have traditionally been used for geophysical applications in nearshore environments. In ideal conditions these methods have often managed to provide reasonable quality information about the subsurface geology. The Aquares resistivity system provides high resolution results in a range of conditions that would be beyond the capabilities of the traditional sub-bottom profiling systems. This paper reviews the limitations of traditional seismic methods and describes the performance of the Aquares resistivity system during a recent near-shore resistivity survey in Tema, Ghana.

Survey setting

The survey area consisted of a corridor of 1 km wide and 2 km long, parallel to the beach. Rock exposures were visible on the beach and were known to extend into the surf zone. Water depths varied from 0 to 11 m. As acoustical systems were not expected to be successful in the shallower areas, PMI Ltd. had opted for the Aquares resistivity survey. The survey operations were carried out in close cooperation with PMI Ltd.

Acoustic systems

Sub-bottom profiling systems are currently the mainstay of shallow water seismics and include boomer-, sparker-, pinger-, chirp-, and parametric echosounder systems. The limitations of these acoustic methods are well-known. In shallow water, seismic methods often provide poor quality results because of *multiple reflections*. This happens when the acoustic signal keeps bouncing up and down between a strong reflecting seabed and the water surface and causes the relevant seismic information to be masked by these repeated seabed reflections.

Seismic methods tend to have problems penetrating the seabed when a hard crust is present at the seabed surface. Such structures are often described in boreholes as *caprock* or calcarenite. These are such strong reflectors that they reflect all seismic energy and mask everything below them. In organic rich mud, decomposition of organic matter results in tiny methane gas bubbles. Each individual gas bubble shows up as a diffraction hyperbole on the seismic records, effectively masking everything at deeper levels. Gravel in the subsurface also tends to generate similar diffraction hyperboles, limiting the penetration of seismic

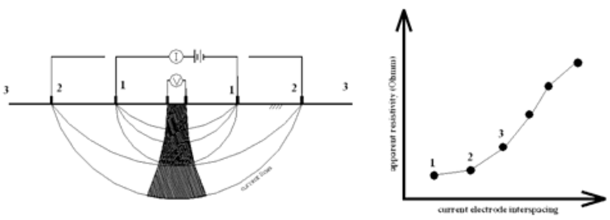

Fig. 1a Land based set-up for a resistivity survey (left) and a resistivity curve showing apparent resistivity values as a function of current electrode interspacing (right)

methods. Seismic depths are determined based on the travel time of the seismic signal and assumptions about the speed of sound in the subsurface. As generally no information is available on the exact speed of sound, assumptions can be inaccurate and therefore seismic depths as well. Surface towed seismic systems are sensitive to sea conditions, rough seas tend to generate poor seismic data quality.

Aquares Resistiv ity System

The Aquares resistivity system as developed by Demco NV is based on the traditional principles of land based resistivity methods but with unique data acquisition and processing specifications. An electrical current is injected into the subsurface by means of two electrodes. The voltage gradient associated with the electrical field of this current is measured between two voltage electrodes placed in between the current electrodes (see Figure 1a). Based on the measured values of current and voltage the average resistivity of the subsurface is calculated for a subsurface volume down to a certain penetration depth. The penetration depth depends on the distance between the current electrodes. Larger electrode distances are associated with increasing penetration depths.

If the measurements are repeated with progressively increasing current electrode distances, information is obtained from progressively deeper geological structures (Figure 1a). As such, a field curve is obtained showing the resistivity as a function of the (horizontal) distance between the current electrodes. After modelling, this field curve is

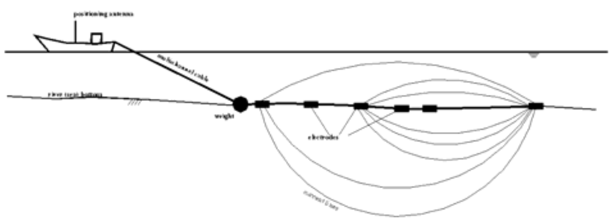

Fig. 1b Aquares set-up utilising a bottom towed cable

transformed into a real geophysical subsurface section showing the resistivity as a function of depth. The resistivity of a geological structure depends on its porosity, water saturation and water resistivity. Gravel usually has a lower porosity than sand and its resistivity is thus higher. Clay with generally very high porosities shows very low resistivities. Solid rock, on the other hand, has a low porosity and shows very high resistivities. Every geological structure therefore has its own specific resistivity.

For water based applications the electrodes are placed on a multi-channel cable trailing behind the survey vessel (Figure 1b). According to the circumstances the cable may be floating or towed on the seafloor. A floating cable may be more efficient in shallow water or if obstacles on the seafloor hamper the use of a bottom towed cable. The electrode geometry is chosen in such a way that good quality data may be obtained even for shallower targets. While the survey vessel is sailing, measurements are carried out and stored automatically without any intervention from the operator. As such, an entire electrical sounding may be obtained every 2.5 seconds. At a boat speed of 2 m/s this corresponds to a horizontal resolution of one sounding at every 5 m.

Fig. 2 Preparation of survey vessel

A complicated sequence of mathematical operations has to be followed before any interpretable results can be obtained. First, the resistivity field data is edited and filtered to improve the signal/noise ratio. The bathymetric and positioning data is edited as well. Then, resistivity data, positioning data and bathymetric data is combined. Geometrical corrections are applied to correct for the fact that the sailed line (and the cable as well) may show more or less significant curvatures. Measurements made with a strongly curved cable are rejected. In case of a bottom towed cable other corrections are made to account for the water depth.

After interpolation of the resistivity information into a regular grid, vertical cross sections or 3D representations of the subsurface are obtained. The results are visualised in colour on cross sections showing the different geological structures

Fig. 3 Shallow draft survey vessel

as a function of depth and geographical position. Eventually, the results may be calibrated with information from a limited number of boreholes in order to verify and sample each geological structure. If sufficient lines have been sailed in the same zone or river section, a 3D model of the subsurface may be constructed. Across such a model vertical and horizontal cross sections may be traced in all possible directions and levels. The processing procedure described above is an interactive process. In order to extract a maximum of information out of the raw survey data the processing sequence has to be repeated several times to find the optimum processing parameters.

Survey results

In order to cope with the shallow waters of the surf zone a shallow draft polyester survey vessel was hired and prepared for the survey (Figures 2 and 3). Lines were sailed perpendicular to the beach in the deeper areas beyond the surf zone and parallel to the beach in the surf zone. A maximum of 50 m interspacing was maintained between the survey lines. The very shallow water next to the beach was surveyed using a four-wheel drive on the beach and an African field crew pulling the cable through the water parallel to the beach (Figure 4). This resulted in 100 % coverage of the survey area. After processing a complete 3D model of the survey was generated. Survey results include horizontal as well as vertical resistivity sections (Figure 5) showing the vertical and horizontal extent of silt, sand, gravel and rock.

Conclusions

Even in the surf zone and extreme shallow water the Aquares resistivity system managed to provide full coverage defining accurate depths and thicknesses and clearly distinguishing between silt, sand, gravel and rock. Based on the resistivity results the pipe trenching operations were de-

Fig. 4 Very shallow resistivity survey

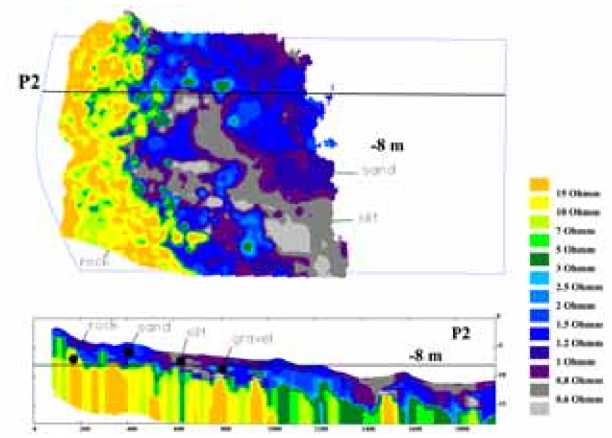

Fig. 5 Horizontal and vertical resistivity sections showing vertical and horizontal extension of silt, sand, gravel and rock

Further information

The Aquares resistivity system is currently used for sand searches, exploration of alluvial minerals (gold, diamonds), pre-lay cable surveys and dredging reconnaissance surveys to assist in dredge and port design. Main advantages in comparison to traditional acoustic geophysical methods are the capability to distinguish between different sediment and rock types, flexibility in application of the method, and accuracy of depths and thicknesses provided.

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Koen de Grave is the co-founder and director of Project Management International (PMI) Ltd., a company specialising in project management, engineering and site investigations. PMI is active worldwide for marine-, civil-, mining-, and oil- & gas related industries (www.pmi-ltd.co.za).

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Peteralv Brabers is the founder and managing director of Demco NV, a geophysical survey company involved with the development and application of the Aquares resistivity survey method. Demco NV is active worldwide, applying marine resistivity surveys on dredging markets, in port design, pipeline and cable route surveys and alluvial mining (www.demco-surveys.com).

A trilogy of papers on site investigation and its tribulations in The Hague

Foreword

Michiel Maurenbrecher *(Retired lecturer, Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

The following three papers are all dealing with a site investigation which was carried out for a proposed housing development in the Willemspark area near the centre of The Hague, The Netherlands.

The first paper describes the site investigation and the type of analyses that could (and should) be done (besides the ubiquitous Koppejan bearing capacity determination) on the site investigation data, especially the CPT profiles. The paper emphasises the need for improved codes of practice with regard to site investigation in the dense urban areas of Dutch cities and towns on 'infill' sites.

The second paper expands on a method to estimate potential settlement that may result from the effects of pile driving causing 'dynamic-type compaction' of the subsoil.

The third paper exists in a poster format (made for ICOF 2008) and shows that problematic site conditions correlate well with the local geology of The Hague. Coincidently an underground car park being constructed for the new Hilton Hotel at the other more 'wealthy' end of the Willemspark neighbourhood of The Hague (best known for its monumental Plein 1813 and the stately homes around it now occupied by embassies) caused excessive settlement of the Panorama Mesdag Museum containing the unique and famous 360° mural view of Scheveningen at the end of the 19th century. This happened whilst writing the first two papers, but coincidence goes further: the linear beach front *(strandwallen)* prominent on both the excellent historical and recent geology maps of The Hague connect the sites mentioned in the paper to that of the Hilton and Panorama Mesdag! Yet, despite the relatively favourable setting of The Hague (with the best property locations situated on its present day and ancient beachfronts and dunes), the properties remain prone to settlements. These can be both natural but also induced, as has transpired recently, by pile driving. The knowledge for good practice is available and present amongst the engineers and, for that matter, the geologists in The Netherlands. Municipalities have to make use of this knowledge and avoid unnecessary hassles. In the end the only people who benefit are the lawyers. Probably this is the policy of The Hague: to make it the world capital for law and international courts. Trouble is lawyers like to live in solid houses and work in solid buildings reflecting the solid fees they receive (often from disputes over settlements caused by pile driving!).

Versions of the first two papers were published in the proceedings of 'Engineering geology for tomorrow's cities', the 10th IAEG International Congress held in Nottingham, UK, on September 6-10, 2006, and of the 2nd BGA International Conference on Foundations (ICOF), Dundee, Scotland, June 24-27, 2008.

A trilogy of papers on site investigation and its tribulations in The Hague

First paper

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

Michiel Maurenbrecher *(Retired lecturer, Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Abstract

The standard site investigation 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 for all foundation designs owing to manoeuvrability and size of pile rigs. In a vacant lot of 5 x 10 m, only a hollow steel tube-piling rig could access the site where a house once stood. The contractor chose this in favour of the design-recommended screw 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 he give them due consideration. Such a situation of municipal involvement happened in Dubai 15 years earlier. The municipality questioned the results of a site investigation report as a result of comparing the report with an earlier investigation on a neighbouring site. Possibly such a precedent can not be a function of all municipalities. The site investigation standard should be extended to include information within a 100 m radius of the site relating to site investigation and foundations.

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 ground subsurface. Regarded as an environmental hazard it is strange that other hazards are not adequately addressed typically for Dutch environments, namely disturbances and possible damage that may result from construction works, possibly the most critical stage being installation of pile foundations.

Site investigation requirements for a small plot of 5 x 20 m are two cone penetration tests of which one includes both end bearing (q_c) and sleeve friction (f) measurements (so as to identify the soils). These are used to calculate the foundation bearing capacity of the piles. In addition it is common practice to examine neighbouring structures for existing damage such as differential settlement and cracks in the plaster work, often done in combination with a photographic record and installation of simple tell-tales over existing cracks to measure any possible movement during installation of piles.

Often though, one resorts to 'vibration-free' piles, the most common being screw (augered) piles and another advertised as vibration-free consisting of hollow cylindrical steel tubes which are, however, driven into the soil 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 annoyance to adjoining residents, structural damage, compaction of sub-soil causing subsidence, and liquefaction resulting in loss of foundation support.
- Erosion of sublayers due to extraction of soil to insert open ended tubular piles.

Methods exist to predict both movement and disturbance, though they seldomly seem to be included in site investigation reports. Methods with regard to compaction due to vibrations are not considered, despite such techniques being used to improve the density of soils (e.g. vibroflotation). This is in spite of a few (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. Soils under bailing operations in open ended piles cause subsurface erosion if water levels inside the pile tube are lowered beneath the water table level.

A recent combined study by the foremost institutes in The Netherlands on the uncertainties in prediction methods based on expert experience, simple prediction methods and more complex (computer driven) methods 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.

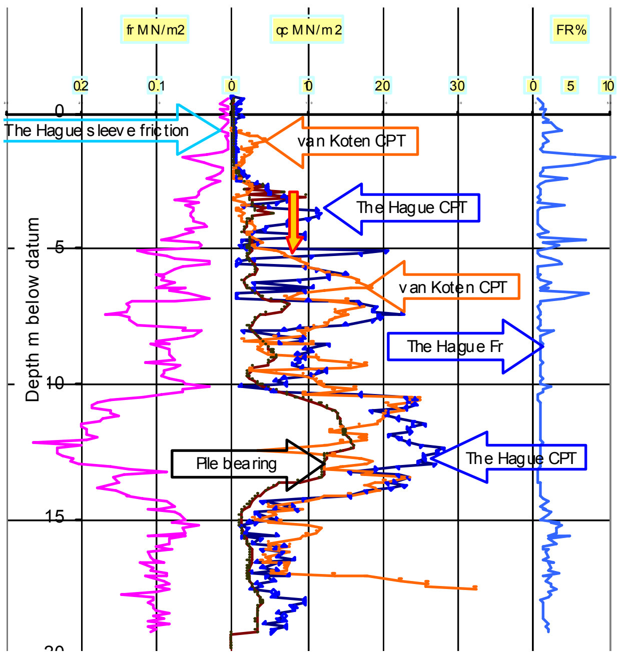


Fig. 1 CPT profiles from The Hague and q_c profile from Van Koten (1992) with position of pile driving

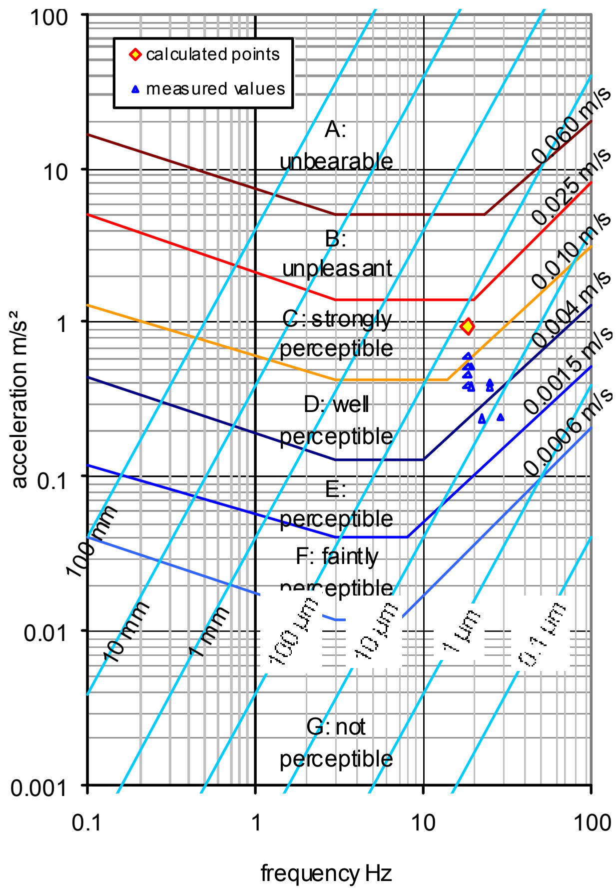


Fig. 2 Chart from Van Koten (1992) with superimposed the predicted values and those measured at the site

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 a CPT profile. A typical profile for bearing capacity of a 20 cm pile is shown in Figure 1 for the site in The Hague. A second method is used by the pile 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 of analysis is originally described in Van Koten (1972). In this instance use is made of a subsequent publication by Van Koten (1992). A worked example

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

Symbol	Description	Formula/source	Value
q_c	Cone tip resistance	CPT profile	8 MPa
D	Equivalent pile diameter		0.32 m
E	Elastic modulus soil		200 MPa
ρ	Density soil		1500 kg/m^3
R	Distance pile foot-observation point		14 m
l	Minimum building dimension		7 m
L	Wave length	$2tc_s=(2\pi D/0.1)/[2(1+\nu)]^{½}$	12.5 m
t	Half wave period	π/ω	0.028 s
ω	Circle frequency	$(0.1/D).[E/\rho]^{½}$	114.11 Hz
f	Frequency	$\omega/2\pi$	18.16 Hz
c_s	Shear wave velocity soil	$[E/(2(1+\nu)\rho)]^{½}$	226.46 m/s
ν	Poisson's ratio		0.3
c_c	Compressional wave velocity	$[E/\rho]^{½}$	365.15 m/s
R_{afs}	Reduction due to wave decay	$e^{(-0.5.R)}$	0.0009
R_b	Reduction due to λ	$e^{(-0.5.l/L)}$	0.76
λ	Influence factor: minimum building dimension/wave length	l/L	0.56
V	Maximum vertical velocity observation point	$0.03(q_cD/ER).c_cR_{afs}R_b$	0.0000069 m/s

has been reproduced here (and translated from Dutch) as the example corresponds to the situation in The Hague. The results can also be compared with velocity measurements made by The Hague municipal engineers. In 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 7 x 10 m. The CPT profile is given in Figure 1.

The cone resistance (q_c) of the layer in which driving requires extra effort is 8 MPa. 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 ranges from 150 to 200 MPa. Dimensions of the piles are 0.32 x 0.32 m. The maximum vertical velocity of the house at a distance of 14 m from the pile foot is given in Table 1.

Measurements taken on site to measure vibrations on applying an impact to a partially inserted pile showed values 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 forming of fractures
E: Little influence for 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 as confirmed by on-site measurements.

Layering can be of influence on wave propagation and attenuation with respect to resonance resulting from reflection of the seismic waves within a layer. A recent report in the web 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 to buildings in the immediate neighbourhood. Rijkswaterstaat (state public works department) will carry out tests to determine what damage vibrations from pile driving could result in. On the basis of these tests Rijkswaterstaat will be able to contain nuisance and damage to a minimum. Additionally all buildings within 100 m 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 related to the effect of pile driving on compaction of soil have been made more to examine the increased hammer energy required to install subsequent driven piles and/or reduce the depth of their end bearing level. On the The Hague site the possibility of compaction 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

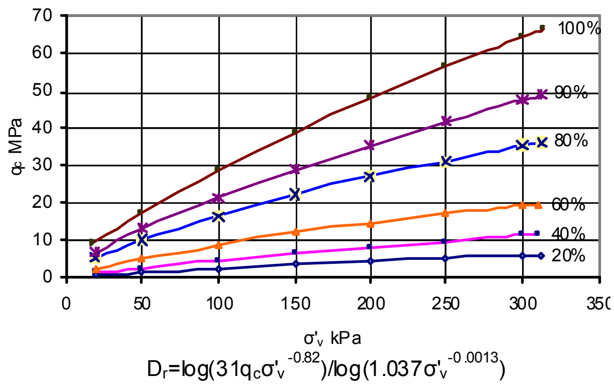


Fig. 3 Relative density based on Lunne & Christoffersen (1985), modified from Schmertmann (1978)

correlate the cone penetration test end bearing (q_c) and friction ratio (R_f) with relative density. Such graphs have been produced by Schmertmann (1978), Searle (1979) and inferred from Robertson (1990). The correlation with Schmertmann (1978) appears to produce values exceeding 100 % D_r (relative density). The Schmertmann chart has been modified, the example shown in Figure 3 is from Lunne & Christoffersen (1985) for electrical q_c values. To obtain densities, a further chart in Figure 4 relates unit weight with relative density D_r.

Relative densities could be derived from cone tests using the q_c-R_f chart by Searle (1979). 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 coarser soil type. This may be because Searle's chart is based on the mechanical cone. Robertson's chart, which is superimposed, is based on the electrical cone and corre-

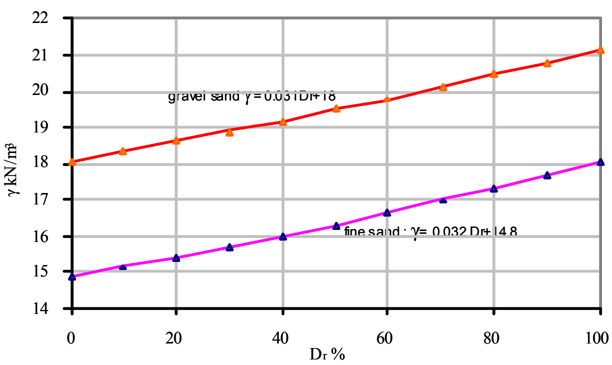


Fig. 4 Relative density versus unit weight (CROW, 2004)

sponds well with the type of soil fraction one would expect in The Hague. Robertson's more recent 'normalised' chart allowing for vertical effective stress is shown in Figure 6 with the CPT data once again plotted depicting similar clusters for the 5-10 m depth zone and the 10-15 m depth zone. The 5-10 m 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 from literature were examined, 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 in 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, the chart in 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 ver-

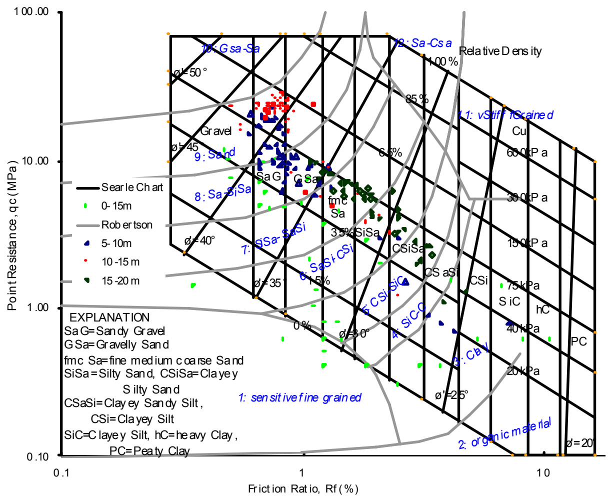
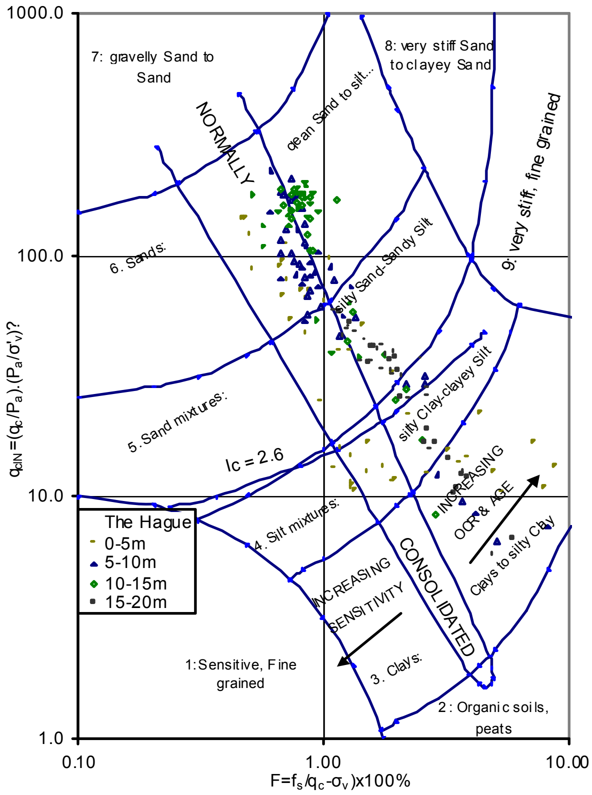


Fig. 5 CPT chart from Searle (1979) with Robertson (1990) superimposed, q_c and R_f values from the The Hague CPT

tical 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 up the process. Unit weights were obtained for every 0.1 m interval of soil depth corresponding with the resolution obtained from CPT profiles. In Figures 9 and 10, rough correlations were made to get an indication as to which density increase was obtained as a result of pile driving. This was used to estimate potential density increases for the sands at the The Hague site. For the 'normally consolidated' sands between 5 and 10 m, up to 140 mm settlement was estimated. The procedure described above indicates that the order of settlement predicted is similar to what occurred during piling operations.

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



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

Fig. 6 Normalised CPT end bearing and friction ratio chart from Wride et al. (2000) with superimposed CPT values from the The Hague site

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 groundwater would not be able to drain. Figure 6 was used in a major research project (CANLEX: Canadian Liquefaction Experiment) to examine susceptibility of soil layers to liquefaction. The sand layers at dam sites in Canada believed to be prone to liquefaction plotted within zones 5 and 6 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 overconsolidation' zone. Unfortunately, presumably because problems were not expected, no careful record of either pile calendars or arising problems was kept during pile installation. 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 weight 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 away (about 1.5 to 2 m) would possibly have avoided the worst influences on adjoining buildings.

Déjà vu: an urgent need for urban site investigation improvements

In the early 1980's 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 4-5 m depth were 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

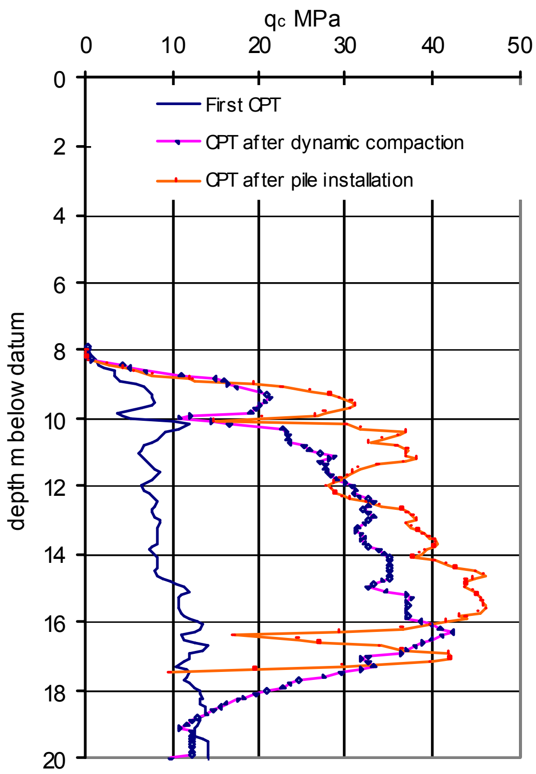


Fig. 7 CPT profiles for IJmeer railway bridge foundations (Van Rossum, 1985)

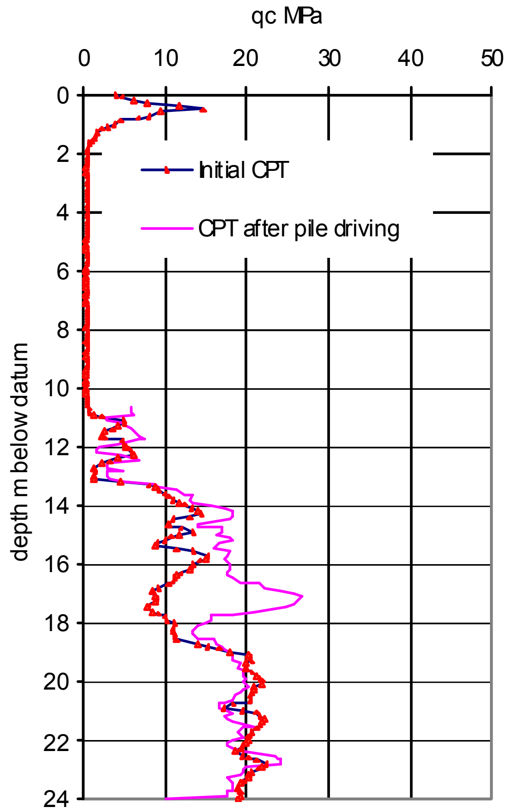


Fig. 8 CPT increase as a result of installation of two piles at a distance of 1.65 m in Rotterdam (Van Weele & Schellingerhout, 1991)

that, indeed, from the earlier investigation it showed that piles were needed and the subsequent investigation showed 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 CPT's with similar profiles. The next obvious step is to examine the type of piles used. It soon became apparent that for an adjoining old building the foundations were reinforced by underpinning with piles, presumably to ensure no adverse settlement would result when installing piles for the new structure. The small size of the plot and neighbouring building did not preclude these drastic measures. Conversely the bane of property developers in Dutch towns and cities, the assessors for planning permits seem overly concerned with the aesthetics of the proposed buildings and are seemingly incapable of critically examining foundation proposals unless they show an absence on the present day fixation of piled foundations. Why does the building have to conform to existing historical facades whereas the foundations demand the latest technology? Shallow piles or even spread footing foundations existing under neighbouring buildings would solve the tight spot. Indeed, the smaller the plot the bigger the problems.

The simplest approach would be that when submitting an application for construction in an urban environment the applicant must show that such records have 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 and allow the structural engineer to propose foundations similar to those of neighbouring buildings.

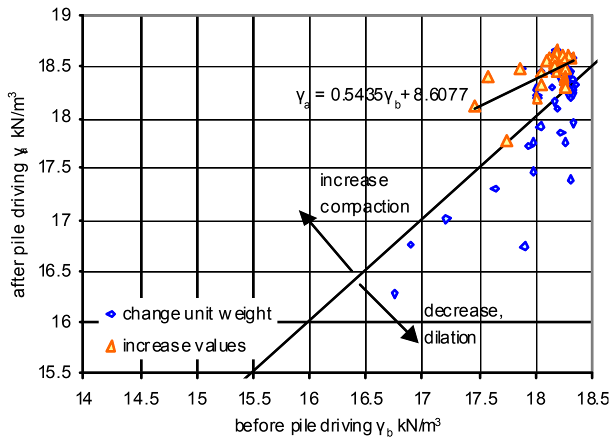


Fig. 9 Before and after pile driving unit weight correlation for IJmeer bridge site

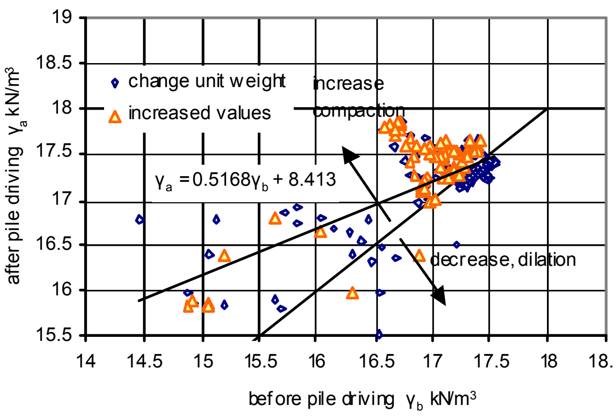


Fig. 10 Before and after pile driving unit weight correlation for Rotterdam site

Acknowledgement

Much of the data in the neighbourhood of the The Hague site was assembled in 1998 by Chewe Kambole for his MSc thesis in Engineering Geology at ITC (The Netherlands) in cooperation with TU Delft.

Editorial note: the original paper has been presented at 'Engineering geology for tomorrow's cities', the 10th IAEG International Congress, Nottingham, UK, September 6-10, 2006.

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A trilogy of papers on site investigation and its tribulations in The Hague
Second paper
Estimating settlements due to pile driving activities

Michiel Maurenbrecher *(Retired lecturer, Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Abstract

Pile driving predominantly causes an increase in density in the normally consolidated sands of the western part of The Netherlands. In the urban environment severe settlements of about 100 to 250 mm can be induced in adjoining older buildings which are usually founded on shallow foundations. From cone penetration test (CPT) records taken from a few examples of increases in cone resistance after pile driving, correlations are made between increases in cone resistance and density. The settlement estimate is done using CPT data based on these correlations. The analysis is made using a spreadsheet and involves a simple iterative process to calculate the vertical effective stress increase.

Introduction

Ten years ago on a small infill site in The Hague, piling operations induced unacceptable settlement of an adjoining house. Though analyses exist for estimating possible damage and disturbance by piling operations, until recently (Meijers, 2007) there appears to be no predictive method for estimating potential settlement due to pile vibrations. Vibrating the soil by dynamic compaction or vibroflotation are well established methods for densification of subsurface layers. To estimate possible settlements assumed to be caused by pile driving densification, a spreadsheet calculation method using CPT profiles has been found to give results that reflect the settlements which occur in this case. Few CPT profiles exist which show a profile before and after pile installation. Those that were found in the literature were correlated with existing relative density relationships. These were in turn related to actual densities of fine sands found predominantly in the western Netherlands.

Procedure for analysis

CPT profile

Though nearly all CPT profiles are digitised recordings of the actual test, the profiles used in this analysis are digitised from the paper graphs simply by tracing a scanned version ensuring a value for every 0.1 m depth interval. The interval produces a resolution which results in an almost identical reproduction of the original profile. This approach was required to obtain the published profiles of the Almere (Van Rossum, 1985), Rotterdam (Van Weele & Schellingerhout, 1991), Amsterdam (Van Seters & Verweij, 2004), and The Hague (acquired from The Hague municipality archives) CPT's. These profiles were analysed to get before and after pile installation cone resistance (q_c) values. The profiles used for the analysis are shown in Figure 1. The four pairs of CPT's

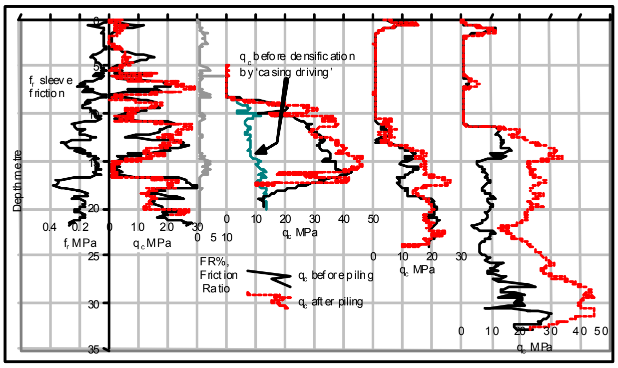

Fig. 1 CPT profiles of The Hague, Almere, Rotterdam, and Amsterdam (see text for literature references)

are located at the railway viaduct outside Almere, Rotterdam centre (infill plot for a terraced house), The Hague (redevelopment area at the Frederikstraat in the Willemspark neighbourhood), and Amsterdam (Oosterdok dockland redevelopment just east of Amsterdam Central Station). It is not certain if the The Hague pair of CPT's are taken before and after piling operations. Of the four pairs, it is the only one in which friction ratios are provided.

Relationship relative density and cone resistance/vertical effective stress

Figure 2 shows two charts combining values of relative density with cone resistance q_c and vertical effective stress σ'_v. The two charts differ, probably as the second chart (Schmertmann, 1978) is meant for the mechanical cone. The first chart (Lunne & Christoffersen, 1983) was used for the analysis as the second chart showed relative densities over 100 % for high cone resistance values. The correlation equations shown in the charts are used for determining relative density. The equations show that subsurface densities would be required to determine the vertical effective stress assuming this is equal to the product of the submerged unit

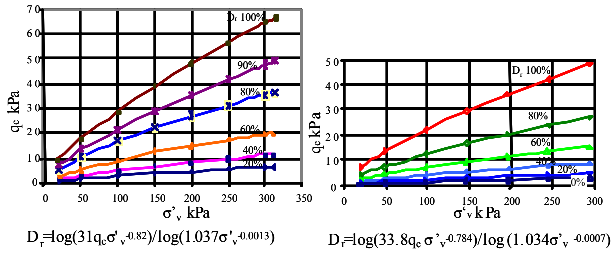

Fig. 2 Relative density charts (Lunne & Christoffersen, 1983; Schmertmann,1978)

weight of the soil and depth (for most situations in the western Netherlands the water table is within 1-2 m of the surface). A situation arises where relative density can only be determined through an iterative process.

Relationship relative density and unit weight

The relationship between relative density and actual unit weight (CROW, 2004) shown in Figure 3 is linear for both fine sands and gravelly sands. Most sands in the western Netherlands can be classified as fine (dune and beach sands deposited in combination with wind and long shore drift). The correlation equations are given in the charts.

Estimating density increase due to pile driving

The CPT profiles in Figure 1 were processed to provide estimates of the unit weight of the subsurface prior to and after pile driving had taken place. This was easily achieved by use of spreadsheets and was done for every 0.1 m depth intervals. Figure 4a shows the initial small portion of the spreadsheet with the relevant columns and associated equations used for determining the densities. Figure 4b outlines the steps required to set up and perform the spreadsheet calculations.

The resulting data from the unit weight estimations is separated into those values that increase in density and those that decrease. The results are presented in Figure 5. In two CPT's the data tended to cluster, indicating that the sand

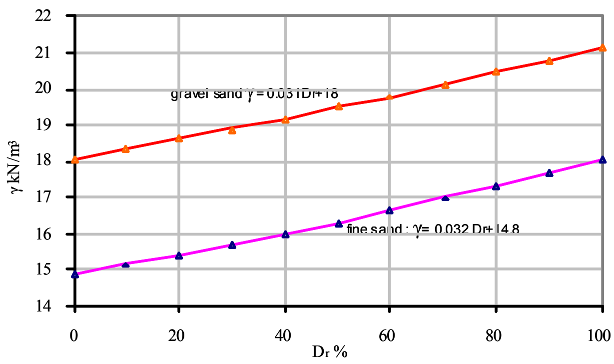

Fig. 3 Relative density versus unit weight (CROW, 2004)

already had a high relative density. The data was interpreted by considering an upper boundary envelope line capturing those data points reaching the highest densities, a median line which lies midway between the upper boundary line and the no change in unit weight line. A 'best fit' trend line was generated through the values of increases in unit weight. The correlation equations are summarised in Table 1.

Choosing a suitable relationship to predict potential settlement

The median lines for all four pairs of CPT's compare favourably. In two instances the 'best fit' trendlines give similar values for The Hague CPT's and for Almere CPT's. The Amsterdam CPT is not dissimilar in profile to that of the Rotterdam CPT but produced a much higher range of increase values. Unfortunately, with the exception of The Hague no sleeve friction data was provided. This would help define sand layers where possible settlement may occur. The close clustering of the 'decrease unit weight' range near to the 'no change line' suggests these points are in clays. The value chosen for estimating potential unit weight increase and the settlement that could result is:

$$\gamma_a = 0.7.\gamma_b + 5 \tag{1}$$

Predicting settlement

The method used is similar to that for analysing the CPT's for unit weight before and after piling. The unit weights are estimated for one CPT using columns A to F in the spreadsheet in Figure 4. The new unit weight is then calculated from equation 1. Figure 6 shows the results of the analysis for an infill plot in The Hague; the calculation being limited to sandy layers (for values of q_c where the friction ratio is less than 1 %). The potential vertical settlement is then found from:

$$\delta H = \Delta H(1 - \gamma_b / \gamma_a) \tag{2}$$

where ΔH is the sampling interval of 0.1 m and δH is the change in the sampling interval due to a change in unit weight. All the δH values are summed to give the total settlement.

The total of all settlement was 270 mm which could be regarded as severe for any structure. The settlement that took place during the piling was about half (140 mm). This may be because the correlation is for situations less distant from the pile driving. The Almere study (Van Rossum, 1985) shows

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	Spreadsheet showing initial rows for 'Amsterdam' pair of CPTs												
2				anchor		γ_b=	anchor		γ_a=	columns sort data for correlations			
3	depth	q_{cb}	q_{ca}	γ_b	σ'_v	f(q_{cb},σ'_v)	γ_a	σ'_v	f(q_{ca},σ'_v)	γ_b< γ_a		γ_b> γ_a,	
4	metre	MPa	MPa	kN/m^3	kPa	kN/m^3	kN/m^3	kPa	kN/m^3	kN/m^3	kN/m^3	kN/m^3	kN/m^3
5					0			0		0	0	0	0
6	0	0.01	0.01	14.357	0.436	14.357	14.357	0.436	14.357				
7	0.1	0.66	0.13	17.325	1.168	17.325	15.961	1.032	15.961			17.325	15.961
8	0.2	0.79	0.13	17.176	1.886	17.176	15.659	1.598	15.659			17.176	15.659
9	0.3	0.82	0.16	17.005	2.586	17.005	15.646	2.162	15.646			17.005	15.646
10	0.4	0.85	0.48	16.882	3.275	16.882	16.461	2.808	16.461			16.882	16.461
11	0.5	0.86	1.13	16.767	3.951	16.767	17.093	3.518	17.093	16.767	17.093		
12													
13	Cell D11: =F11 iteration loop γ_{b11} =γ_{b11}												
14	notes: to start initial calculations first insert dummy value in D6 then later equate to F6 (also for G6 with I6)												
15	Cell E11 : =E10+0.1*(D11-10)												
16	equat^n: σ'_{vb11}=σ'_{vb10}+ (0.1metre) x (γ_{b11}-10), where (γ_{b11}-10) =submerged unit weight												
17	Cell F11: '=14.775+0.0319*LOG(31*B11*((E11)^-0.82))/LOG(1.037*((E11)^-0.0013)1												
18	equat^n: γ_{b11}=14.775 + 0.0319 log(31x q_{cb11} x $\sigma'_{vb11}{}^{-0.82}$)/log(1.03 x $\sigma'_{vb11}{}^{-0.0013}$) (combining equations in Fig. 2 & 3)												
19	Cell G11: =I11 iteration loop γ_{a11} =γ_{a11}												
20	notes: may need a dummy value in G6 to start iteration; later equate with I6												
21	Cell H11 =H10+0.1*(G11-10)												
22	equat^n: σ'_{va11}= σ'_{va10} + (0.1metre) x (γ_{a11}-10), where (γ_{a11}- 10) = submerged unit weight)												
23	Cell I 11 '=14.775+0.0319*LOG(31*C11*((H11)^-0.82))/LOG(1.037*((H11)^-0.0013))												
24	equat^n: γ_{a11}=14.775+ 0.0319log(31xq_{ca11} x $\sigma'_{va11}{}^{-0.82}$)/log(1.03 x $\sigma'_{va11}{}^{-0.0013}$)												
25	Cell J11 IF(F11<I11,F11,0)... algorithm to obtain only values of γ_{b11} when γ_{a11}>γ_{b11}												
26	Cell K11 IF(F11<I11,I11,0)... algorithm to obtain only values of γ_{a11} when γ_{a11}>γ_{b11}												
27	notes: Column J11 becomes the x-coordinate and column K the y-coordinate to plot γ_{a11} = f (γ_{b11}) for density increasing after pile driving												
28	Cell L11 IF(F11>I11,F11,0)... algorithm to obtain only values of γ_{b11} when γ_{a11}<γ_{b11}												
29	Cell M11 IF(F11>I11,I11,0)... algorithm to obtain only values of γ_{a11} when γ_{a11}<γ_{b11}												
30	notes: Column K becomes the x-coordinate and column M the y-coordinate to plot γ_{a11} = f(γ_{b11}) for density decreasing after pile driving												

Fig. 4a Spreadsheet format for correlating γ_b and γ_a before and after piling respectively

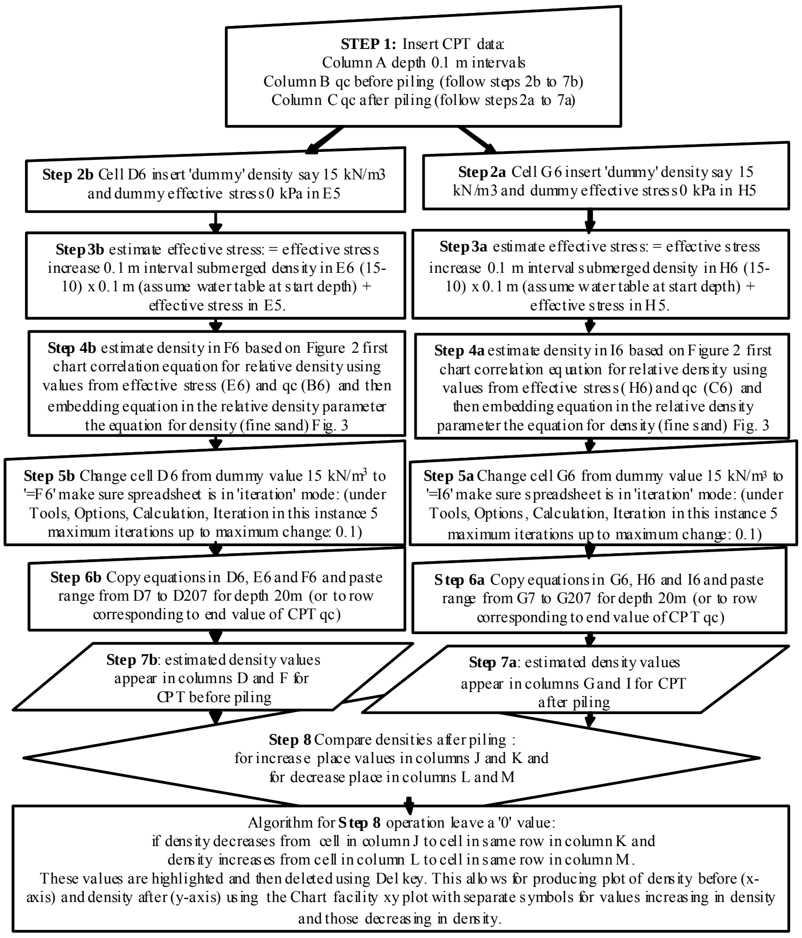

Fig. 4b Flow diagram showing steps for setting up and performing spreadsheet calculations

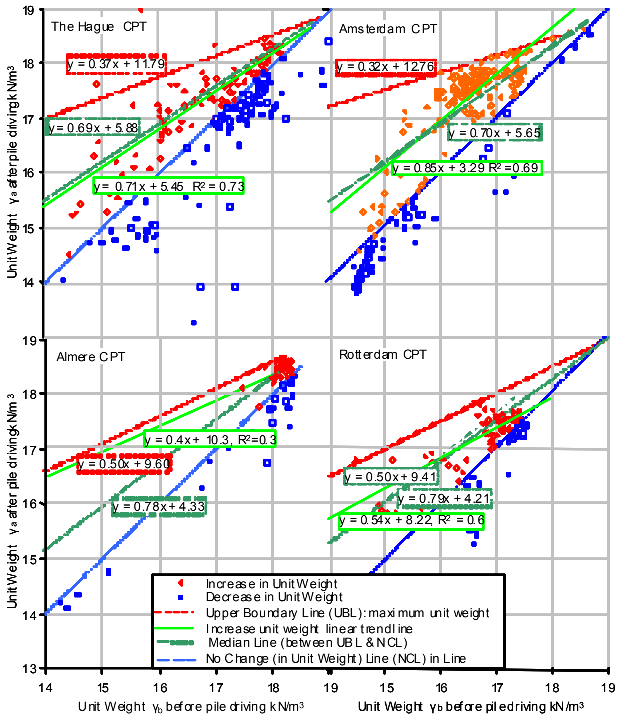

Fig. 5 Unit weights before and after densification derived from four pairs of CPT's

2.10 m distance. Part of the compaction could also take place laterally as the pile displaces the soil on penetration. Figure 5 shows that unit weight also decreases for many values though the decrease could also be a loss of strength in the more cohesive layers rather than a loss of density except in instances where sands dilate. Hence, there is a need to have more CPT's with sleeve friction measurements to examine this aspect.

The estimation should be regarded as preliminary to assess roughly the settlement that may be expected in vulnerable sites such as that of the The Hague infill plot where older buildings resting on shallow foundations are susceptible to piling operations. Equally, these structures are also vulnerable to vibrations from piles. The CPT is used, for example, to estimate potential disturbance due to vibrations on buildings (Van Koten, 1992; Maurenbrecher, 2009). The estimates of shear wave velocities were found to correspond well with those measured by the building inspectorate from The Hague municipality. The prediction does not examine the extent of the settlement with distance from the pile driving. In Figure 1 the CPT profiles for the Almere railway viaduct foundations show initially an increase of cone resistance due to densification induced by 'casing driving' which suggests driving a hollow steel cylinder into the ground. The steel cylinder was presumably then extracted after driving. Figure 7 from the same publication (Van Rossum, 1985) shows that the influence of an increase of q_c values is still significant up to 2.10 m from the original CPT at the point of densification. Determination of the average density ratios (density after/ density before) showed that at 0.85 m, 1.25 m and 2.10 m distance, ratios of 1.043, 1.036 and 1.024 were estimated respectively. When plotted in a straight line, these values intercept the no change in density line (ratio=1) at 3.5 m distance and when plotted as a best fit curve, densification can still be expected at 4.5 m or 1 % (ratio 1.01). The average values include both increases and decreases in density.

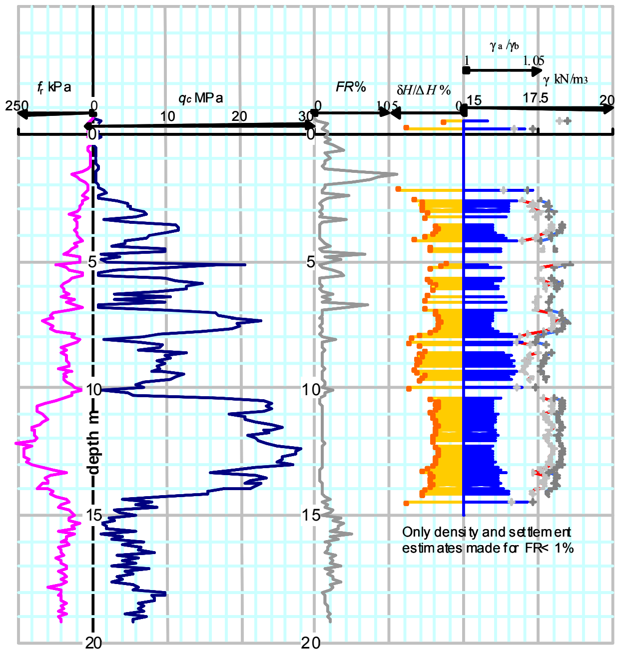

Fig. 6 CPT for foundation design infill plot The Hague, unit weight and percentage settlement are also shown

Table 1 Summary correlation data unit weights from CPT before (γ_b) after (γ_a) pile driving

CPT	$\gamma_a = m\gamma_b + \gamma_c$						
	Upper boundary line		**Median line**		**Data generated trendline**		
	m	γ_c	m	γ_c	m	γ_c	R^2
Almere	0.5	9.6	0.78	4.33	0.4	10.3	0.3
Rotterdam	0.5	9.41	0.79	4.21	0.54	8.22	0.6
Amsterdam	0.32	12.76	0.7	5.65	0.85	3.29	0.69
The Hague	0.37	11.79	0.69	5.88	0.71	5.45	0.73
Average	0.42	10.89	0.74	5.02	0.63	6.82	0.58

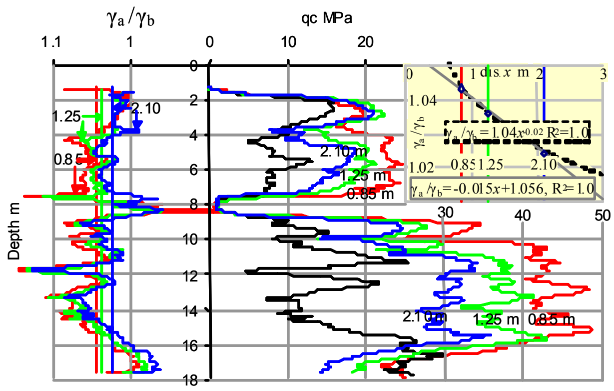

Fig. 7 CPT cone resistance profiles with corresponding density ratios at distances up to 2.10 m from point of densification at the Almere railway viaduct site. Approximate change in average density ratio with distance (x, in meter) given in top right hand corner of figure for best fit line and curve

Editorial note: the original paper has been presented at the 2nd BGA International Conference on Foundations (ICOF), Dundee, Scotland, June 24-27, 2008.

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A trilogy of papers on site investigation and its tribulations in The Hague
Third paper

Added value: archival site investigation

Michiel Maurenbrecher *(Retired lecturer, Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Abstract

In urban environments, municipalities are a large storehouse of geotechnical information. Some municipalities have developed databases for such information, but most keep these records together with the documents needed to obtain permits for construction. Data from municipal records was obtained by an MSc student for his thesis on the dynamic effects of pile installation. The data helped explain problems that had occurred at two existing sites where piling had taken place. This experience demonstrated such investigative work should be part of standard urban site investigations, preferably as a necessary part of the initial site investigation.

Introduction

This paper essentially covers the work done by ITC student Chewe Kambole from Zambia for his MSc thesis (Kambole, 1998) especially with regard to data obtained from the municipality of The Hague. Kambole wanted an opportunity to learn more about pile foundations during his stay in The Netherlands. A building site, where the effects of piling caused damage to an adjacent house and a good deal of disturbance to nearby houses, afforded such an opportunity. In retrospect, one realises, that the investigative approach discussed in this paper should be added to a standard site investigation. It may also influence the way the site investigation is carried out and reported. If such an approach had been used, for example, for the construction of the underground car park for the Hilton Hotel development next door to the Panorama Mesdag Museum the problems the developer now faces may not have arisen (reported in the national newspapers at the beginning of 2008).

This paper is the third of a trilogy of papers about a small urban building plot intended for construction of a four storey dwelling in the Willemspark area of The Hague. Construction was prematurely halted due to vibrations from the pile installation which caused alarming settlement of the adjoining house. The first two papers addressed the issue 'could this have been predicted' on basis of analysis of CPT data obtained from a site investigation for the foundation design. They show that analytical procedures using only the CPT data could both predict the settlement and the disturbance caused by vibrations from pile driving. The disturbance was such that a lawyer residing in a nearby property threatened to obtain a court injunction to stop pile installation operations. A few days later the municipal inspector suspended pile installation operations as a result of the alarming damage to the adjoining house.

Kambole (1998) examined the existing literature on the subject of dynamic loading and ground vibrations causing settlement due to compaction and potential liquefaction. One realised as his review progressed that indeed these types of phenomena would be the cause of settlement by compaction of the subsurface induced by ground vibrations from pile driving. The question then arose if this site was unique in suffering settlement caused by piling operations. Kambole's investigative work took on a new course, namely to see if the soil profile at the site corresponded to others in the vicinity and to examine what happened to foundations from recent nearby construction.

Municipal archives

The obvious source for obtaining data that would answer the question was the municipality. An appointment was made with an employee of the municipal records office in The Hague. Municipalities vary as to how they archive their data. The archiving of site investigation data (submitted to the planning department for obtaining permits for construction) was rather informal. The employee knew in which cabinets relevant files could be found from which CPT profiles could be extracted within the vicinity of the neighbourhood of the Willemspark and adjoining neighbourhoods, namely The Hague centre (area which formed the town in the 17th century) and the Archipel district. The files were appropriately labelled with the districts they covered. The CPT profiles were obligingly photocopied for Mr. Kambole.

The next approach was to learn about foundations for recently constructed buildings. Unlike the site investigations, the foundation records are kept in a large room containing storage cabinets for up to A1 size design drawings. A batch of drawings for a recent apartment building on the Frederikstraat, about 100 m from the building site, could be in-

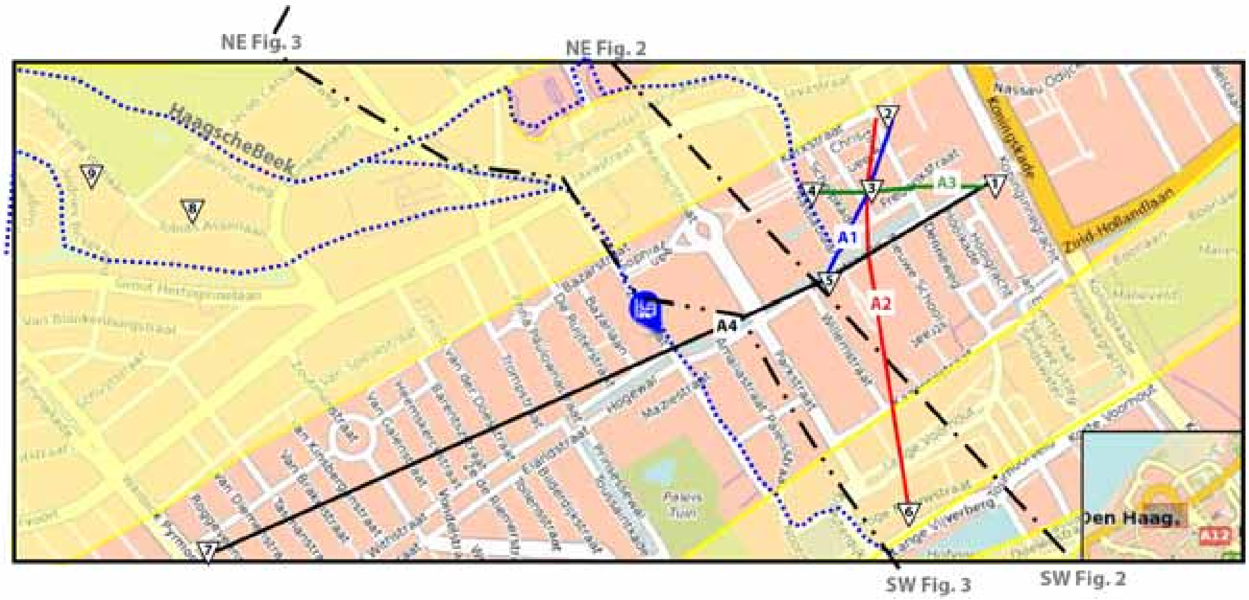

Fig. 1 Street map of the investigated The Hague area with geology superimposed (yellow shading: old dune sands (Stratum of Voorburg) with a cover layer of the sediments of the Stratum of The Hague (less than 2 m), consisting of old beach sands). Non shaded zones: Hollandveen (Holland peat) overlying Stratum of Voorburg (triangles: CPT locations; blue sign: Panorama Mesdag)

spected. Copies could be made but they were relatively expensive. The drawings, as with all engineering drawings, contain sufficient information on pile size and position. The design firm information is also given. It became immediately obvious from these records that a drawing for the installation of piles beneath the adjoining structure was amongst the inspected batch of drawings for the new apartment building. This suggested that prior to or as a consequence of foundation works for the new apartment building measures were taken to strengthen the foundations of a neighbouring old structure. The visit, which took at most two hours, proved to be worth the effort and time.

Applying the data

Profiles were made of the CPT data showing soil types along various cross sections (Figures 4 to 7). The locations of the sections are plotted on a street map on which the geology is superimposed (Figure 1). The geological information is obtained from the geology maps of The Hague (Anon., 1982 & 2007). The geological profile from the 1982 geology map (Anon., 1982) is shown in Figure 2, with the CPT profiles from the building plot (location 3) superimposed. Similarly, Figure 3 shows the CPT profiles superimposed on the geological section obtained from the more recent 2007 geological map (Anon., 2007). Additional CPT profiles, obtained from Kam-

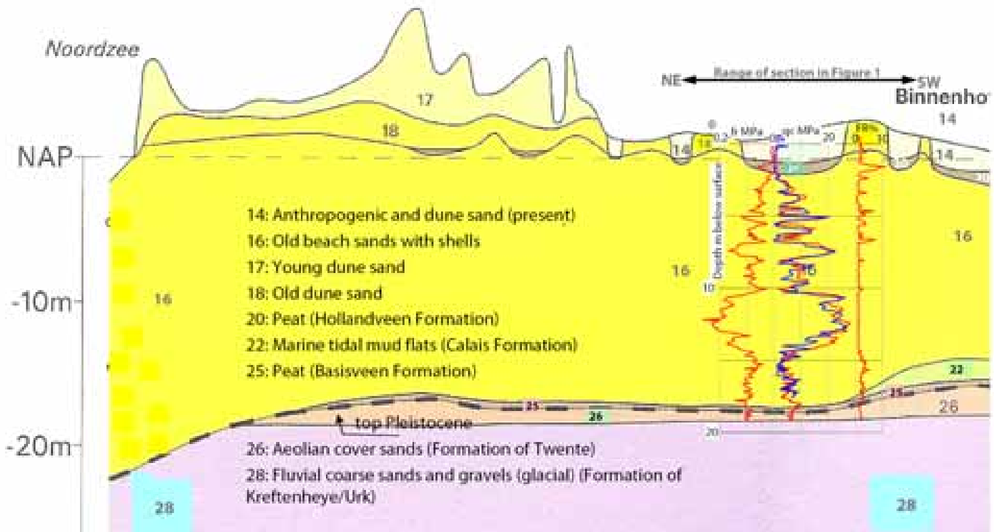

Fig. 2 The geological profile from the 1982 geological map with the CPT profiles (location 3 in Figure 1) superimposed

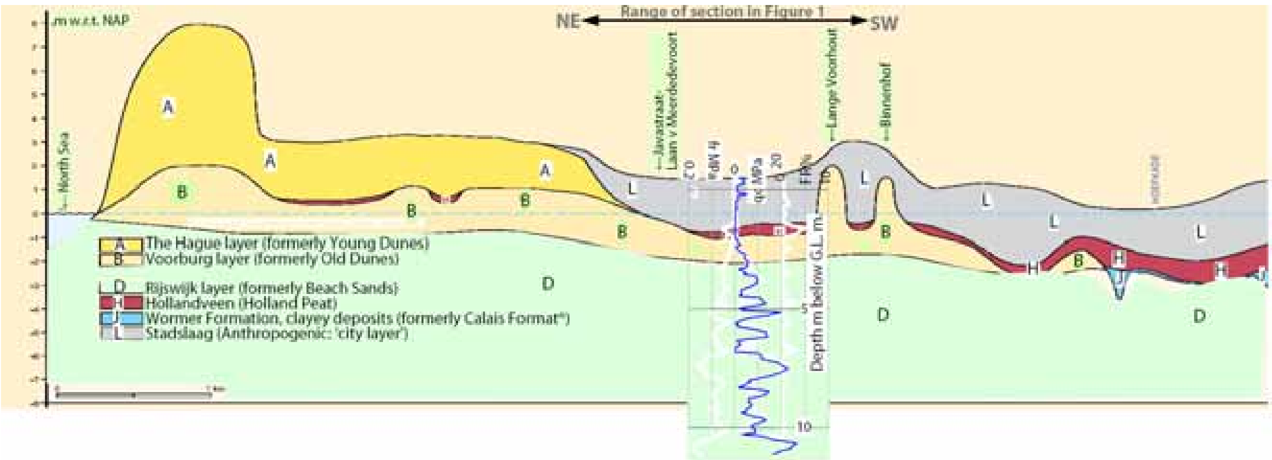

Fig. 3 Geological section obtained from the more recent (2007) geological map with two CPT profiles from location 3 superimposed (pale yellow: with sleeve friction; blue: end resistance only)

bole (1998), are shown in Figures 4 to 7 showing correlations made between the various profiles. There is little variation between the profiles, even in the longest section from Dr. Kuypersstraat to Waldeck Pyrmontkade (Figure 7). The revised edition of the geology map of The Hague and Rijswijk shows more lithostratigraphic subdivisions but is restricted to more recent Holocene stratigraphy and hence for the Willemspark section only sediments down to the top boundary of the Stratum of Rijswijk (old beach sands) are shown (between 3 and 4 m below ground level). The Hollandveen (Holland peat) layer is clearly shown on the CPT sleeve friction profile. The shallow section of the 2007 geological map may not be as useful for indicating soil profiles for foundations.

Déjà vu: Panorama Mesdag

The Panorama Mesdag Museum's 360° view of Scheveningen shows extensive sand deposits of the stratum known as the Young Dune Stratum on previous geological maps of The Hague. These start within ten minutes walking from the museum at the boundary of the Archipel district and Scheveningen woods. Had the museum been located there or slightly closer to the sea at Madurodam, the situation which arose in 2008 would not have happened. Kambole's geotechnical sections clearly show that conditions at the building site (location 3) probably exist at Panorama Mesdag and the Hilton Hotel building. Sheet piles are usually inserted by hammer blows or by a heavy vibrating weight placed on top of the pile. These vibrations will compact the relatively loose sand layers. In the first paper of this trilogy, CPT data classified according to Wride et al. (2000) clearly showed that the sands are still in a 'normally consolidated' phase of consolidation despite some of the layers lying there for more than 1,000 years.

Conclusion

The general observation for pile driving in The Netherlands is that in the east, piles are subject to much higher horizontal ground stresses so that much of the bearing load is taken by shaft friction whereas in the west, pile support is by end bearing. Possibly a corollary should be added to this general observation: pile driving in the west will cause the sands to compact leading to settlement of the ground.

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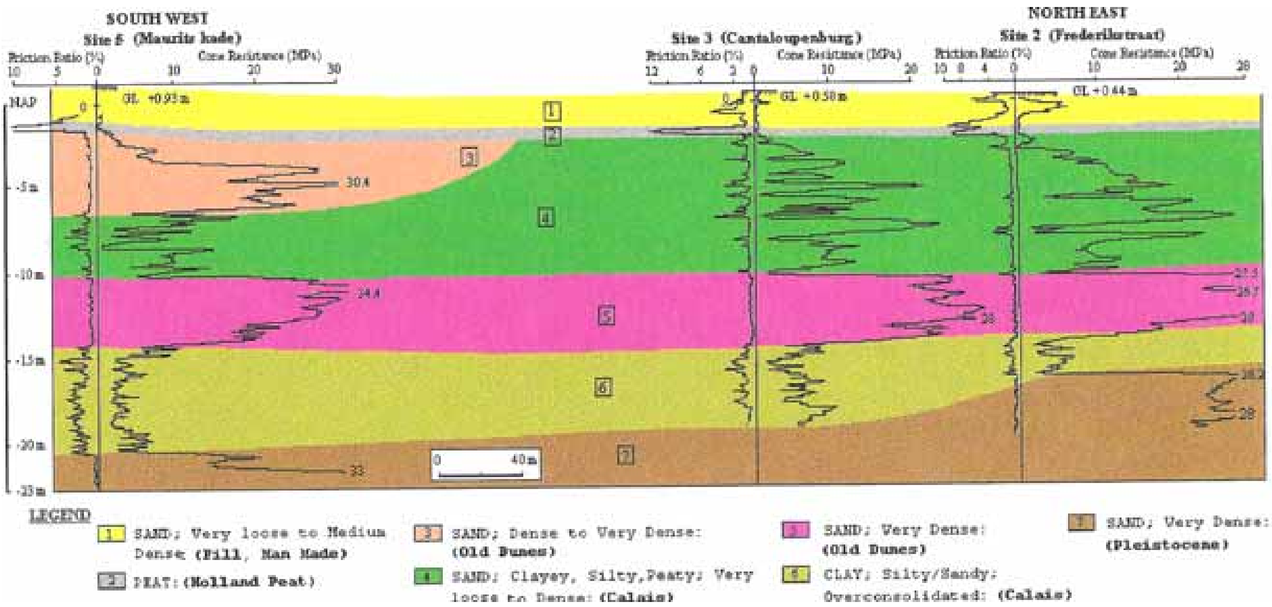

Fig. 4 Geotechnical profile A1, connecting CPT's 5, 3 and 2 (Mauritskade, Cantaloupenburg and Frederikstraat) (Kambole, 1998)

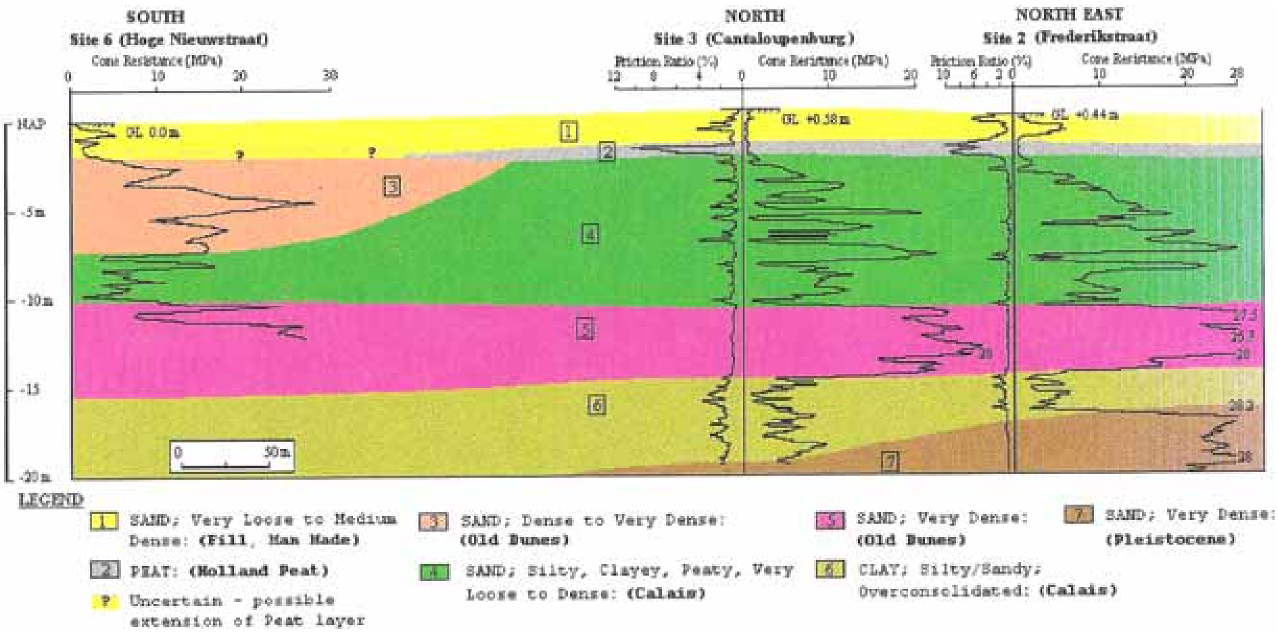

Fig. 5 Geotechnical profile A2, connecting CPT's 6, 3 and 2 (from Hoge Nieuwstraat (parallel to Lange Vijverberg) through Cantaloupenburg to Frederikstraat) (Kambole, 1998)

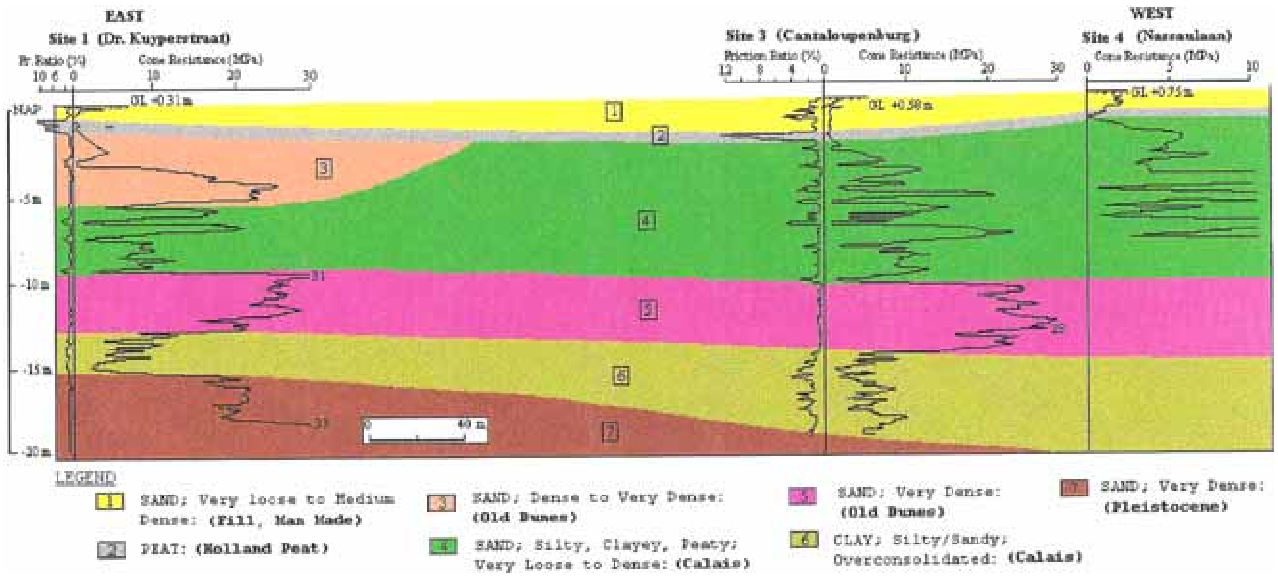

Fig. 6 Geotechnical profile A3, connecting CPT's 1, 3 and 4 (Dr. Kuyperstraat/Mauritskade, Cantaloupenburg and Nassaulaan) (Kambole, 1998)

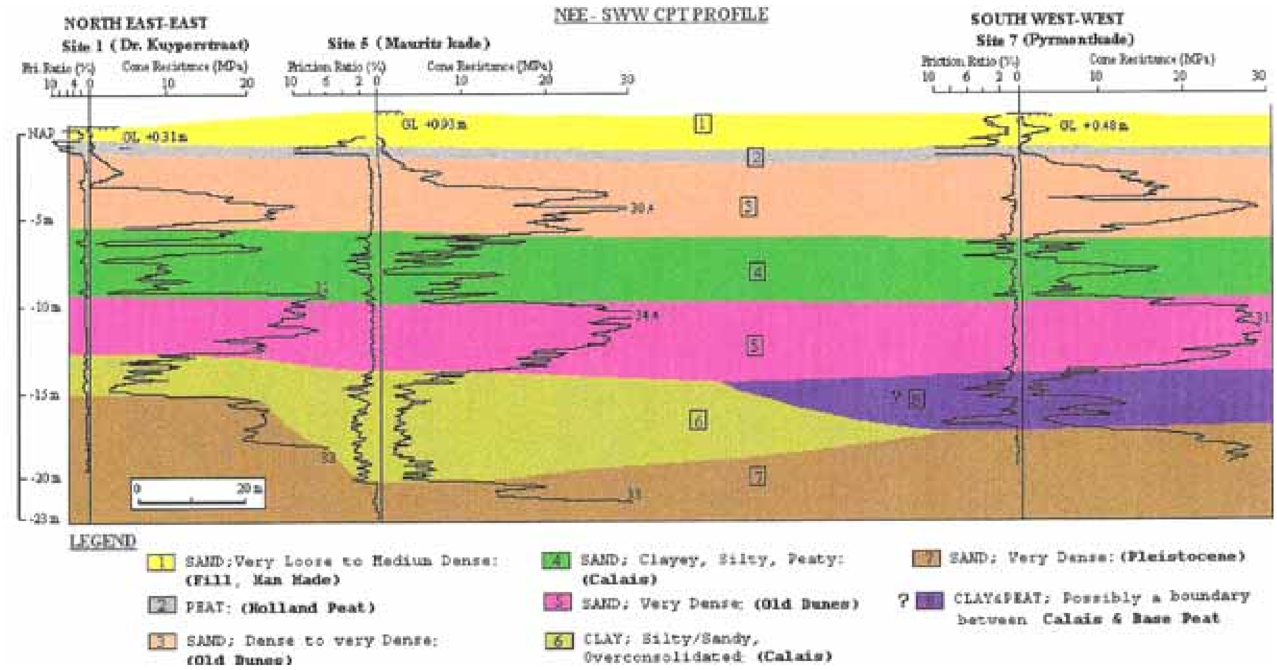

Fig. 7 Geotechnical profile A4, connecting CPT's 1, 5 and 7 (Dr. Kuyperstraat/Mauritskade to Waldeck Pyrmontkade) (Kambole, 1998)

Marine Sampling Holland: a geotechnical company, specialised in drilling and testing in a wide range of environments

Gerrit de Vries *(General Manager, Marine Sampling Holland B.V.)*

Introduction

Marine Sampling Holland B.V. (MSH) is a relatively new company which was established on November 1, 2007 from the privatised marine drilling- and coring department of TNO. The company owns advanced drilling- and coring equipment and has a very experienced staff, specialised in soil sampling of ocean floors, river- and lake beds and harbours and canals.

MSH and its predecessors have a long history that started with the Geological Foundation *(Geologische Stichting)* in the mid-60's of the last century. In this period exploration for oil and gas started in the North Sea, the Port of Amsterdam was extended and large quantities of marine infill sand were needed, and the search for industrial aggregates in the North Sea started as well. For all these infrastructural works, geological knowledge of the North Sea was required. During these years, a reconnaissance study of the superficial seabed layers of the North Sea was carried out based on samples which were taken with vibrocorers that were developed and built in-house.

Oil and gas in the North Sea subsurface

In 1968, the Ministry of Economic Affairs decided to take part in the exploitation of oil and gas in the Dutch sector of the North Sea, and the Geological Foundation became part of the ministry. It changed into the Geological Survey of The Netherlands *(Rijks Geologische Dienst)*, with one of the tasks being to advise the ministry in the field of oil and gas exploitation. The marine department started a reconnaissance mapping of the Dutch sector of the North Sea. Since that time, research of the North Sea geology was intensified, which was made possible by the deployment of ships owned by the North Sea Directorate of *Rijkswaterstaat* (Directorate-General for Public Works and Water Management of The Netherlands). The hydrographical division of the North Sea Directorate is responsible for routine monitoring of shipping lanes in the Dutch part of the North Sea and, in relation to marine incidents, surveying of wrecks and lost cargo. Two special survey vessels, *Arca* and *Zirfaea*, are equipped with sophisticated sensors and oil spill equipment, and for all those tasks knowledge of the seabed geology is necessary.

Part of TNO

In close cooperation with private companies, advanced sampling systems have been developed. In January 1997, the Geological Survey of The Netherlands merged into TNO (Netherlands Organisation for Applied Scientific Research). The group, responsible for marine sampling, became part of TNO Built Environment and Geosciences until November 2007. Around that time, management of TNO Built Environment and Geosciences allowed the group to privatise and Marine Sampling Holland B.V. was founded, as a subsidiary of Wiertsema & Partners B.V., a company specialised in geotechnical- and environmental site investigation and geotechnical advice. Management of MSH is carried out by dr. Cees Laban and ir. Gerrit de Vries.

Fig. 1 Hydraulic vibrocorer

Fig. 2 Mini piston corer

Sampling and testing in a wide range of environments

MSH has a broad range of equipment available. Superficial sampling of the seabed for a first reconnaissance of sedimentological or chemical composition can be done with Van Veen grabs, box corers or hamon grabs. For geochemical sampling of the superficial layers of the seabed, special self operating corers are available that take undisturbed cores with a penetration of up to 80 cm in sandy bottoms. A mini gravity corer makes it possible to sample using very small vessels in shallow waters as lakes and rivers.

Information on deeper layers, up to 5.5 m below seabed, can be obtained by deploying electrical or hydraulic vibrocorers (see Figure 1). The hydraulic vibrocorers make it possible to collect undisturbed samples of sandy, gravelly or clayey seabeds. A core can be taken within 30 seconds only.

Fig. 3 Mini CPT equipment

For deeper information in shelf seas, MSH has systems available for counterflush- or airlift drilling. With these systems, disturbed samples can be taken in sandy, gravelly and clayey soils up to 12-20 m below seabed. These systems are lowered to the seabed after which drilling can take place. Sampling to depths specified above typically takes 30 to 90 minutes. To lower equipment to the seabed, MSH has its own winches and davits available with cable lengths between 1,000 and 5,000 m. A containerised mobile workshop allows the drilling technicians to perform on-board repairs when needed. For sampling of the deep-seafloor, piston- and gravity corers are available with which undisturbed soil samples can be taken up to depths of approximately 18 m below seabed (see Figure 2 for an example of a mini piston corer in use on an inshore project). For CPT testing up to depths of maximum 10 m, a mini CPT system is available (see Figure 3).

Fig. 4 Drilling activities using a jack -up platform

Equipment mentioned before is mainly operated from ships. For nearshore projects from jack-up platforms, in inland waters and on land, land-based drilling- and CPT testing equipment of Wiertsema & Partners B.V. is used (see Figure 4 for an example of a jack-up platform).

Fields of application

Sampling, drilling and geotechnical testing is carried out for a broad field of applications, for example:

- Chemical composition of seabed sediments, environmental quality of river-, lake-, and canal bottom sediments.
- Investigations for quantity and quality of building and infill aggregates (sand, gravel).
- Investigations for quantity and quality of mineral deposits.

- Route survey and geotechnical investigation for laying of pipelines and submarine cables.
- Geotechnical investigation for (offshore) constructions.
- Dredgeability analysis.
- Scientific research.

Hence, MSH can play a role throughout the complete cycle of many offshore engineering works, for example planning, realisation (and sometimes decommissioning) of offshore windfarms, oil- and gas pipelines and beach landings.

Worldwide experience

Projects carried out recently comprise amongst others sand searches for the infill of harbour extensions, coastal extensions, artificial islands, beach nourishments, deepening of shipping routes, stability of tidal bed forms, and sampling for geochemical studies of the seabed.

Worldwide population growth in coastal zones increases the need for coastal protection systems which asks for better insight in offshore sedimentary processes, availability of sand for beach nourishments and infill. The intensification of marine infrastructure implies deepening and maintenance of shipping routes, new harbours and extension of existing harbours. For the exploration of oil and gas in increasing water depths information about the stability of slopes and the ocean floor is required.

The combination of geotechnical and geological knowledge of MSH in close cooperation with Wiertsema & Partners, together with state-of-the-art marine- and land CPT testing, drilling and coring equipment and a well equipped geotechnical laboratory makes MSH the right partner for investigations in a wide range of on- and offshore projects.

Inverse analysis of a road embankment using the Ensemble Kalman Filter including heterogeneity of the soft soil

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Abstract

Geomechanical models are indispensable for reliable design of engineering structures and processes, and hazard and risk evaluation. Model predictions based on site investigation are however far from perfect. Errors are introduced by fluctuations in the input or by poorly known parameters in the model. To overcome these problems an inverse modelling technique that incorporates measurements into the deterministic model to improve the model results can be implemented. This allows for observations of ongoing processes to be used for enhancing the quality of subsequent model predictions. In geomechanics several examples of inverse modelling exist where the improved model of the system is obtained by minimising the discrepancy between the observed values in the system and the modelled state of the system within a time interval. This requires the implementation of the adjoint model. Even with the use of the adjoint compilers that have become available recently, this is a tremendous programming effort for the existing geomechanical model system. The Ensemble Kalman Filter has been implemented to overcome this problem. The Ensemble Kalman Filter analyses the state of the subsurface each time data becomes available. The Random Finite Element Method is used to simulate the heterogeneity of the subsurface. Very promising results of a conceptual example, based on the construction of a road embankment on soft clay, are presented. The Ensemble Kalman Filter is not only used for a straightforward identification of the elastic Young's modulus E of the foundation below the embankment, but also incorporates the determination of several critical parameters of the inverse modelling process.

Introduction

The purpose of site investigation is to determine the behaviour of ground response to the construction of the engineering work (Price, 2009). Calculations during site investigation are uncertain, mainly due to soil heterogeneity. Soil heterogeneity can be classified into two main categories. The first is lithological heterogeneity, which can be manifested in the form of thin soft or stiff layers embedded in a stiffer or softer media (layered cake model) or the inclusion of pockets of different lithology within a more or less uniform soil mass. The second source of heterogeneity can be attributed to inherent spatial soil variability, which is the variation of soil properties from one point to another in space due to different deposition conditions and different loading histories (Elkateb et al., 2003). Since in geostatistics the variable is considered to be a realisation of a random function, it is possible to simulate an infinite number of realisations to represent the variability and spatial characteristics in the experimental data (Bastante et al., 2008). The Random Finite Element Method (RFEM) combines the Finite Element Method (FEM) with the random field theory. A random field is generated using the Local Average Subdivision technique (LAS), which describes the spatial variability of geotechnical parameters throughout a soil layer (Fenton & Vanmarcke, 1991).

During the construction of the engineering work a lot of extra data becomes available. This data can be used to optimise the model and parameter uncertainties, this process is called inverse modelling. There are several techniques, each with their advantages and disadvantages. For this research the Ensemble Kalman Filter has been chosen because it can easily deal with non-linear models and does not need the adjoint model for the optimisation process. The performance of the Ensemble Kalman Filter in a normal field has been proven in Hommels et al. (2005).

The general formulation of the Random Finite Element Method (RFEM) as well as the theory behind the Ensemble Kalman Filter (EnKF) will be explained in the next sections. The effects of the spatial variability on the inverse modelling process using the Ensemble Kalman Filter is shown in a case study based on the construction of a road embankment.

Ensemble Kalman Filter

Evensen (1994 & 2003) introduced the Ensemble Kalman Filter (EnKF). The EnKF was designed to resolve two major problems related to the use of the Extended Kalman Filter (EKF). The first problem relates to the use of an approximate closure scheme in the EKF, and the other one to the huge computational requirements associated with the storage and forward integration of the error covariance matrix in the EKF.

In the EnKF, an ensemble of N possible state vectors, which are randomly generated using a Monte Carlo approach, represents the statistical properties of the state vector. The algorithm does not require a tangent linear model, which is required for the EKF, and is very easy to implement. In Figure 1 a flow sheet of the EnKF is given. At initialisation, an en-

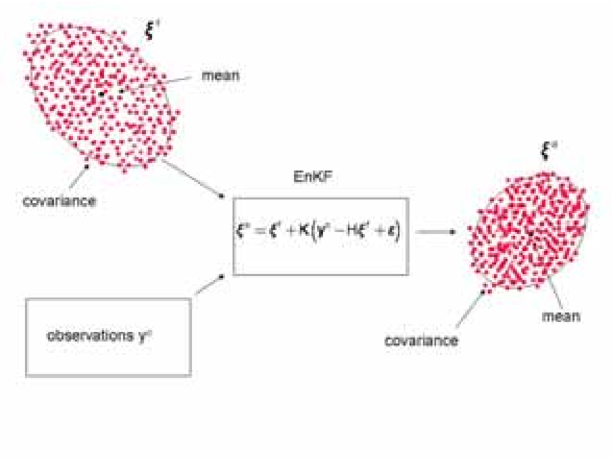

Fig. 1 Flow sheet of the Ensemble Kalman Filter for one assimilation step

semble of N initial states $(\xi_N)_0$ are generated to represent the uncertainty at time step $k=0$. The matrix E^f_{k+1} defines an approximation of the covariance matrix P_{k+1}. The time update equations for the Ensemble Kalman Filter for each ensemble ξ are:

$$\xi^f_{i,k+1} = f(\xi^a_{i,k+1}) + G_k w_k \tag{1}$$

$$x^f_{k+1} = \frac{1}{N}\sum_1^N \xi^f_{i,k+1} \tag{2}$$

$$E^f_{k+1} = \left[\xi^f_{i,k+1} - x^f_{k+1}, \ldots, \xi^f_{N,k+1} - x^f_{k+1}\right] \tag{3}$$

in which G_k is the noise input matrix and w_k is the process noise. The measurement update step equation is:

$$x^a_{i,k+1} = x^f_{i,k+1} + K\left\{y^o_{k+1} - Hx^f_{i,k+1} + \varepsilon\right\} \tag{4}$$

in which y^o are the measurements, H is the observational operator, ε is a randomly added measurement noise, because the measurements are treated as random variables, and K is the Kalman gain, which is defined as:

$$K_{k+1} = \left[\frac{1}{N+1}E^f((E^f)^T H^T)\right]_{k+1}\left[\frac{HE^f((E^f)^T H^T)}{N+1} + R\right]^{-1}_{k+1} \tag{5}$$

The performance of the EnKF is dependent on several input parameters, which are amongst others the model- and measurement noise, the number of ensemble members, the amount of observations and the initial parameter uncertainty.

Random Finite Element Method

The Random Finite Element Method (RFEM) combines the finite element analysis with the random field theory generated using the local average subdivision method (Fenton & Vanmarcke, 1991).

In the Finite Element Method the uncertainty of a material is defined by its mean μ and its standard deviation σ. For more spatial variability, the introduction of an additional statistical parameter (spatial correlation length θ) is required. The spatial correlation length defines the distance beyond which there is minimal correlation and can be determined from e.g. CPT data. A large value of θ indicates a strongly correlated material, while a small value indicates a weakly correlated material. After determination of θ the random field can be generated and in the Random Finite Element Method (RFEM) this is done using the local average subdivision (LAS) technique based on a standard normal distribution (mean μ is zero and the standard deviation σ equals one) and a spatial correlation function ρ. LAS generates a square random field by uniformly subdividing a square domain into smaller square cells (Figure 2), where each cell has a unique local average, which is correlated with its surrounding cells.

Construction of a road embankment

For this case study, construction of a four meter high road embankment is considered, where the Young's modulus E of the foundation is uncertain and modelled using LAS, see Figure 3. The horizontal spatial correlation length is considered to be infinite, where the vertical spatial correlation length is considered to be 0.3 times the depth of the foundation.

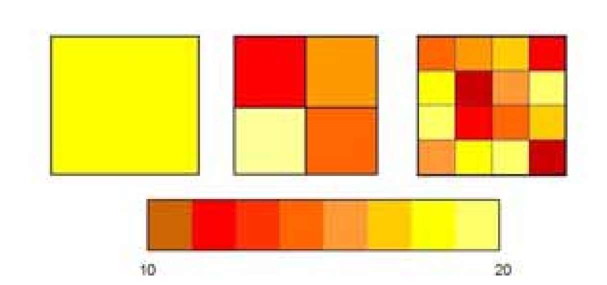

Fig. 2 Basic principles of the Local Average Subdivision theory

In Figure 3, the foundation below the embankment as modelled using LAS is shown. Some of the observation points at which the vertical displacement is measured, and which are used for the input of the EnKF, are indicated with a blue rectangle. The foundation consists of 208 elements (683 nodes). The embankment is constructed in four phases (Figure 3), where at the right axis the vertical displacement at one of the observation points is shown. The influence of the amount of measurement noise, the number of ensemble members and the number of observation points are considered in the case study.

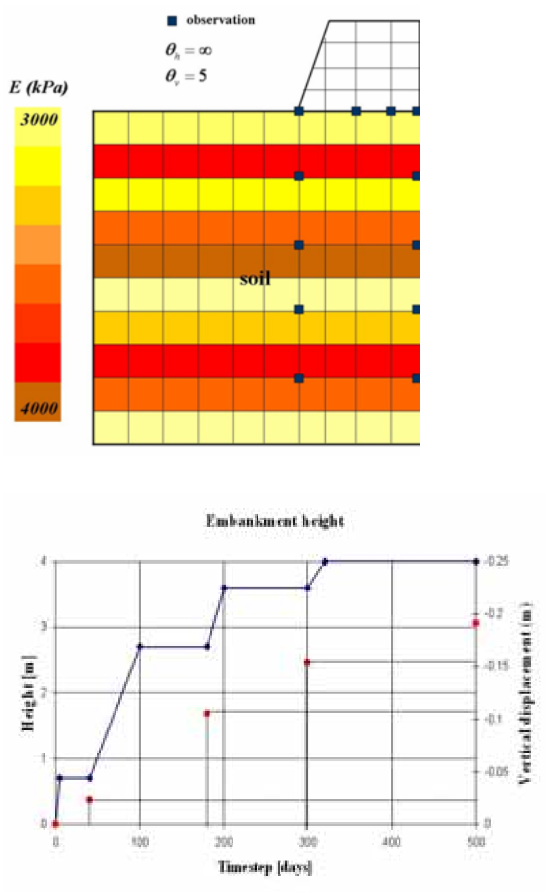

Fig. 3 Top: road embankment with foundation modelled using LAS, in which several observation locations are indicated with blue rectangles. Bottom: embankment height constructed in four phases (left axis); vertical displacement at one of the observation locations (right axis)

Results

First the influence of the amount of measurement noise is considered. In Figure 4, the difference between a measurement noise with a standard deviation of 10^{-4} (top) and 10^{-6} (bottom) using 100 ensemble members in each case is shown.

The influence of the amount of ensemble members is shown in Figure 5. In the top figure, 50 ensemble members are used and in the bottom figure 150 ensemble members are shown using a measurement noise with a standard deviation of 10^{-4}.

In Figure 6 the influence of the amount of observation points on the inverse modelling process is shown. In the top figure, 10 observation points for each time step are used and in the bottom figure 36 observation points for each time step are shown. For both calculations a measurement noise with a standard deviation of 10^{-4} is used.

Conclusions

In Figure 4 the difference between measurement noise with a standard deviation of 10^{-4} (top) and 10^{-6} (bottom) using 100 ensemble members is shown. From these figures it can be concluded that if the measurement noise decreases, the parameter uncertainty decreases and the speed of the assimilation process increases. In the bottom figure it is clearly shown that both the mean and the standard deviation of the Young's modulus are improved.

In Figure 5 the difference between 50 and 150 ensemble members using a measurement noise with a standard deviation of 10^{-4} is shown. From the top figure it can be concluded that 50 ensemble members are not enough for a correct assimilation process of the Young's modulus *E*. In the bottom figure, 150 ensemble members are used and the parameter uncertainty has decreased significantly. In contrary to the bottom figure of Figure 4, only the mean has improved. This can be explained by the lower standard deviation of 10^{-4}.

Figure 6 shows that there is hardly any difference between 10 and 36 observation points on the inverse modelling process. This implies that the EnKF doesn't need a lot of information to perform well.

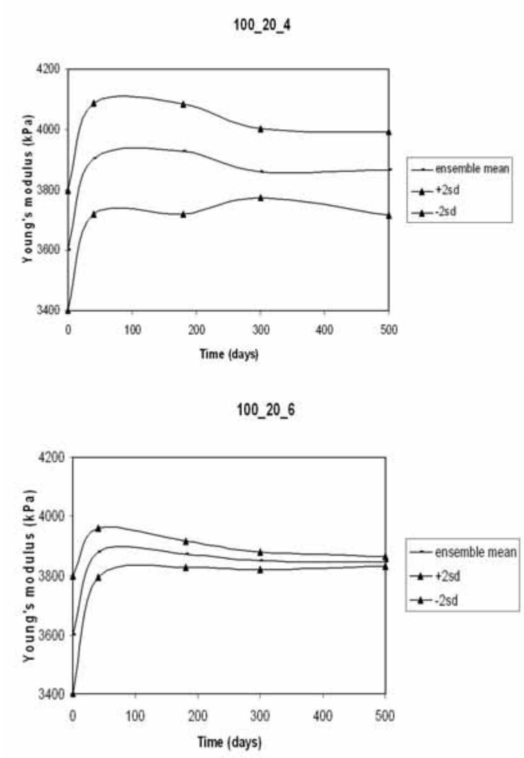

Fig. 4 Difference between a measurement noise with a standard deviation of 10^{-4} (top) and 10^{-6} (bottom) using 100 ensemble members

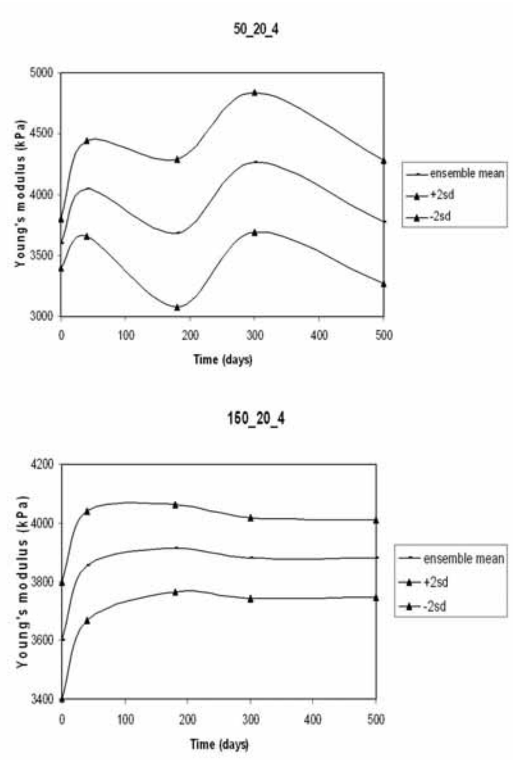

Fig. 5 Difference between 50 ensemble members (top) and 150 ensemble members (bottom) using a measurement noise with a standard deviation of 10^{-4}

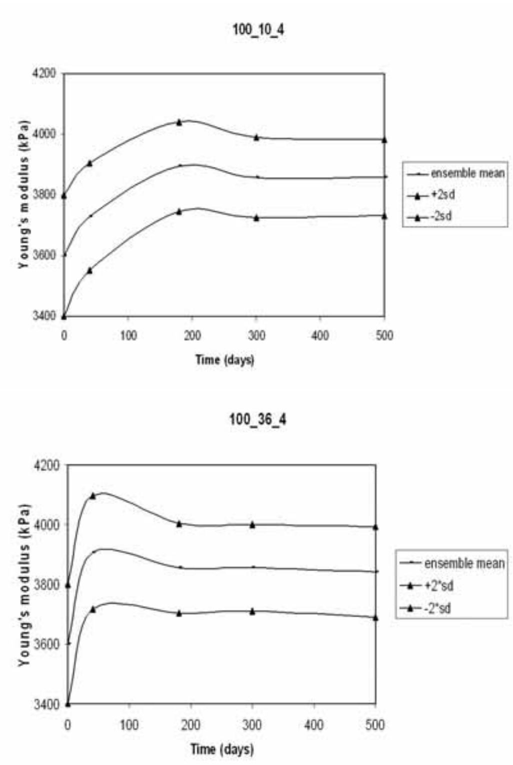

Fig. 6 Difference between 10 observations points (top) and 36 observation points (bottom) using a measurement noise with a standard deviation of 10^{-4}

In general, the Ensemble Kalman Filter is a very promising additional tool after site investigation using the observations which are made during the construction works. It is well able to deal with geological uncertainties and doesn't need much information to show good results.

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Book review

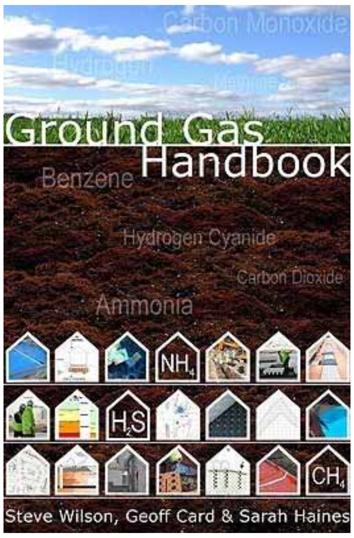
Steve Wilson, Geoff Card & Sarah Haines: *Ground Gas Handbook*

Timo Heimovaara *(Associate Professor Environmental Geo-Engineering, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

The Ground Gas Handbook is an excellent resource for a professional working in the field of contaminated land and who is often confronted with situations where the risks of ground gas have to be considered. After a short introduction where a number of ground gas incidents are highlighted, it becomes immediately clear that this book is for ground gas assessments and ground gas abatement measures within the context of the UK regulations. An overview is given of the different sources of ground gas followed by a very short and non-scientific summary of the physics and biogeochemistry involved, making it accessible to people without much of a science background (although I feel that they still will find it a complicated subject).

As it is a book for the practitioner, a lot of emphasis is put on site investigation, monitoring and assessment. The complete process is described, starting with defining the investigation strategy, through desktop study approaches, into detailed field investigations. Different sampling and analysis approaches are given with their pros and cons. A strong point of the described assessment approach is the paragraph on justification of parameters and sensitivity analysis, clearly illustrating the fact that uncertainties are always present when performing soil investigation and that these uncertainties must be taken into account. In the concluding chapters, the book focuses on the methods and regulations available for implementing protection measures. As may be expected, much emphasis is put on protection measures in relation to building.

This book gives an excellent overview of issues related to ground gas. As it is meant to be a handbook, one does not have to read through the complete book; when using the table of contents and index one can easily find the relevant information. On the other hand, the introductory chapters 1 to 3 are highly recommended for someone just starting in the field or as a brush up of old knowledge. A minor critique could be the fact that this book only focuses on the situation in the UK. A separate chapter illustrating the relationship with regulations in the US and on mainland Europe would definitely increase the potential readership.



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Engineering geological site investigation for linear infrastructure on soft soil

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Abstract

The Geo-Impulse project Reliable Subsurface Model aims at a 50 % reduction in failure costs due to poor subsurface modelling in civil engineering works. The current Dutch approach towards site investigation for linear infrastructure almost completely ignores geological factors. The Total Engineering Geology Approach is shown to be a suitable basis for an improved approach; integrating geology, remote sensing and geophysics. Expert knowledge can be made available to non-expert end users by two methods available at TNO-NITG and Deltares.

Introduction

Following an increase in the number of subsoil related failures in civil engineering works, the Centre for Infrastructure of the Dutch Ministry of Transport, Public Works and Water Management has initiated the program Geo-Impulse. Geo-Impulse aims at a permanent reduction of failure costs by 50 %, approximately €150 million annually. With a budget of only €7 million, the success of Geo-Impulse will depend on realising a radical change in the cooperation between principals, contractors, consultants and researchers.

One of the projects in Geo-Impulse is 'Reliable Subsurface Model' *(Betrouwbaar Ondergrond Model)*. The aim of this project is to promote the application of a balanced combination of maps, archived data and models, remote sensing, geophysics, field and laboratory testing and inverse modelling for management of subsoil related risks. The subsurface model is a set of different 3D geotechnical models with spatial dimensions and properties that are relevant to geotechnical design, with known reliability. The geotechnical models are derived from the geological model in a systematic procedure. The subsurface model will gain in reliability from project conception (using maps, archived data and models) to operation and maintenance of the completed work, integrating monitoring data in the model. In its final state the model is made available for design of future engineering works.

The project Reliable Subsurface Model is closely linked to another Geo-Impulse project: 'Spotlights on the subsurface' *(De ondergrond naar de voorgrond in projecten)*. The latter project concerns the implementation of management of subsoil related risks in civil engineering works. Rules for decision-making in risk management are to be linked to the reliability of the subsurface model.

After a description of the current approach to subsurface modelling, alternatives will be briefly reviewed. Some missing links that are critical for making a big step forward are identified, defining the core of the project Reliable Subsurface Model.

Current approach to subsurface modelling

CUR publication 2003-7 (CUR, 2003) describes the current approach, which relies heavily on borings and CPT's. CPT's are placed at distances of 100 m, with a boring for undisturbed sampling at every four CPT's. In this approach, the probability of missing subsurface heterogeneities with dimensions of 20 m is approximately 80 %. Such heterogeneities are common in the Dutch delta, consisting of buried channels of anastomosing river systems and crevasse splays. A strong stiffness contrast exists between sand channels and compressible flood plain- and residual channel deposits. This contrast may result in unacceptable differential settlements of a road embankment if not accounted for in site investigation and design. Knowledge of the local geology and geomorphology, remote sensing, and geophysical surveys is necessary to guide and complement the campaign of borings and CPT's. Unfortunately, this expert knowledge is not easily condensed in a standard, nor is it provided by the usual training of geotechnical engineers. The obvious solution is to involve external experts, but in practice the geotechnical engineers and experts rarely meet. The knowledge is there, but human, procedural and institutional factors prevent its effective application.

In the current absence of clear and complete standards and guidelines, the experience and attitude of the geotechnical engineer makes the difference between success and failure. Two examples are given that contain keys to improving the current practice.

CUR publication 162 (CUR, 1992) is the first publication that provides guidelines for site investigation for linear infrastructure on soft soil. In the preparation of the guidelines, consultants of three renowned companies estimated the required number of CPT's, borings and laboratory tests for geotechnical design. The difference between the lowest and the highest estimate was a staggering factor three. The experience of the consultants was compatible, the difference was found to be related to the varying degrees of competition which the consultants were used to when tendering for a site investigation. The guidelines in CUR publication 162 (CUR, 1992) and later 2003-7 (CUR, 2003) were based on the highest estimate.

Koelewijn (2002) describes a case where five experienced consultants were asked to perform a stability analysis of a river embankment on the basis of two CPT's only, supplemented by their expert knowledge. A that time, this procedure was typical for the assessment of river dikes. The difference between the lowest and highest factor of safety in the analyses was 0.3, and the different geotechnical models differed significantly (Figures 1 and 2). Obviously, the impact of this variation on potential failure costs is enormous. To simulate the best practice, the participants were given an additional boring log and strength- and pore pressure data and were asked to repeat the analysis. The spread in safety factors was still 0.06. This case lead to a modification of the guidelines for design of river dikes, rewarding a better site investigation with lower safety margins. These examples illustrate the crucial role of the human factor in the current practice, and the need for improvement. The improvement should primarily be directed towards application of existing knowledge by end users rather than development of new knowledge.

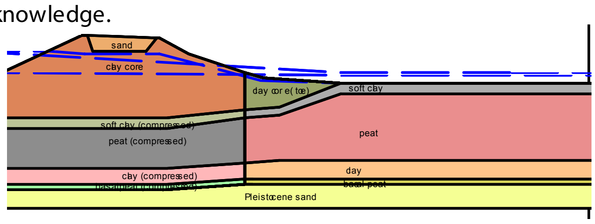

Fig. 1 Soil profile based on two cone penetration tests, consultant I (Koelewijn, 2002)

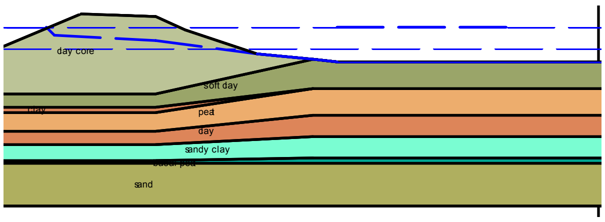

Fig. 2 Soil profile based on two cone penetration tests, consultant II (Koelewijn, 2002)

Suggested approach to subsurface modelling

As indicated in the discussion on the current approach, a great advance can be made by integration of existing geological and geomorphologic knowledge and surveys with existing remote sensing and geophysical methods. This concept is also prominent in the Total Engineering Geology Approach as introduced by Baynes et al. (2005). This approach consists of acquiring a sound understanding of the geological and geomorphological history of the construction site at an early stage of the project. Site investigation strategies, geotechnical risk management and monitoring programs are designed to suit the ground conditions and project requirements. The approach also contains a method to break down the geological model into smaller units which helps in communicating the results of the site investigation to the design phase. Thus, the Total Engineering Geology Approach is the opposite of the current Dutch approach of 'drill first, ask questions later'.

Ngan-Tillard et al. (2010) demonstrate the benefits of application of the Total Engineering Geology Approach to site investigation for motorway construction. The main steps are:

1) Identify the geological and geomorphological history: for a densely populated country like The Netherlands, also the history of cultivation and present and future geohydrology are important. The required information can be found in textbooks, maps and their legends, and archives of the national geological survey TNO-NITG and local water boards.

2) Study remote sensing data: digital surface elevation maps from airborne laser altimetry surveys will reveal the effects of differential compaction at the site scale. These maps allow the detection of shallow buried channels and other architectural elements of meander belts.

3) Walk-over survey: a walk-over survey by a trained geomorphologist or physical geographer will clarify the site morphology, which is a reflection of the shallow subsurface, and improve the interpretation of remote sensing and geophysical data. Hand borings may verify the presence of heterogeneities.

4) Geophysical surveys: a variety of geophysical techniques is suited to detect heterogeneities in the subsurface, and complement the remote sensing data. In the feasibility stage a multi-frequency electromagnetic survey is probably the most cost-effective option (Ngan-Tillard et al., 2010).

5) Integrate the results of the previous steps: an expert will be able to conceive an initial geological subsurface model, based on generic geological models matched with site geomorphology, interpretation of remote sensing and geophysical data and archived CPT's and borings. The initial geological subsurface model produces an initial geotechnical model by breakdown of the model into smaller units and attributing parameters for geotechnical design to the units. At this stage the parameters are derived from archived tests and subsurface models, or correlations.

6) Qualitative assessment of geohazards associated with the ground conditions and the engineering concerns that are to be dealt with in geotechnical design. For linear infrastructure on soft organic soil the main geohazards are high compressibility and low strength, heterogeneity, susceptibility to weathering and changes in groundwater conditions, and intrusion of brackish water. Tentative calculations with the initial geotechnical model will produce a shortlist of construction options and techniques meeting the project requirements. Analysis of the sensitivity of the construction options for uncertainties in the subsurface model will help to identify critical design parameters, direct the subsequent site and laboratory investigation, and devise a plan for control of subsoil related risks.

7) Verification of the initial subsurface models by CPT's, borings and laboratory tests, and construction of a design stage version of geological and geotechnical models. The initial geological subsurface model guides the location of verticals. The required reliability of the critical design parameters drives the amount of verticals and laboratory tests. Different geotechnical models may be derived from a single geological model to respond to different engineering concerns.

8) The design stage version of the subsurface model allows the selection of the construction method meeting the project requirements. Also, the models support the quantification and allocation of risks between contractor and principal. Baynes et al. (2005) suggest methods such as the Geotechnical Baseline Report and the Observational Method. The aim is to eliminate excessive financial risks for the contractor.

9) A final campaign of field and laboratory tests is ordered to obtain data for final design, to address practical construction aspects, to determine warning and intervention levels and contingency measures for the observational method and for verification of critical elements in the subsurface model. The construction stage version of the subsurface models provides the reference for ground behaviour during construction of the project.

10) During construction, data from monitoring critical parameters is continuously being checked against reference values. Deviant behaviour may lead to the implementation of contingency measures and adaptation of the subsurface model to the observed behaviour.

11) The as-built versions of the subsurface models are stored in databases accessible for later use. The models are relevant for maintenance planning, for selection of maintenance or reconstruction actions, and for construction at nearby sites.

Successful application of the Total Engineering Geology Approach still requires a significant amount of expert knowledge on geology, geomorphology, remote sensing, geophysical techniques, geostatistical methods and risk management. This knowledge is not present in the curriculum of the average geotechnical engineer. The authors believe the remoteness of expert knowledge is one of the major obstacles on the way to a substantial improvement of the current practice. Two methods that will eliminate this obstacle have been currently implemented.

The first, the Risk Approach for Dikes *(Rationele Risicobenadering Dijken (RRD))* (Deltares, 2009), was applied successfully in the past years. All modelling is done by experts of TNO-NITG and Deltares, with only the geotechnical models presented to the end user for certain dedicated tasks. The reliability is expressed by providing multiple geotechnical models for a given location, representing different stratigraphies with known probability of occurrence, rather than a single model. Automated geotechnical calculations for thousands of cross sections are summarised in GIS for the assessment of safety levels and prioritisation of strengthening works.

A second approach is derived from the 3D modelling (Stafleu et al., 2008) of the top 30 m of the Dutch subsurface by experts of TNO-NITG based on 350,000 borings in the national DINO database. The resolution of the model is 100 x 100 x 0.5 m. Smaller voxels may be generated for specific geotechnical analyses with block kriging giving the probability of occurrence of different lithologies in the voxels. Next, Monte Carlo simulation produces a large number of realisations of discrete lithologies, which can all be converted to discrete geotechnical models for analysis.

Missing links

The Geo-Impulse project Reliable Subsurface Model will support the implementation of the suggested approaches in and demonstration of these tools in a number of pilot projects. The Total Engineering Geology Approach will provide the guideline for the improved practice. Experience and knowledge with the Risk Approach for Dikes and the geostatistical 3D modelling will help to disclose expert knowledge to non-expert users. A combination of technological developments, procedures for interactions between parties and knowledge dissemination will be required to bring about a lasting improvement after the end of the Geo-Impulse project.

Technological improvements may include data acquisition and reduction for remote sensing and geophysical techniques to support subsurface modelling. Knowledge development may comprise the generation of geotechnical models dedicated to specific engineering concerns from the geological model, and the allocation of parameters for geotechnical design to the units in the models. Other new knowledge concerns the assessment of the reliability of the models in relation to the project specifications, and the integration in risk management methods such as the Geotechnical Baseline Report and the Observational Method.

An analysis of human, procedural and institutional factors will confirm what actions are most effective in elimination of obstacles to the acceptance of the improved approach. Practical guidelines and training will help geotechnical engineers to use the improved approach in their daily work. Software may support decision-making and disclose expert knowledge to non-expert users.

Conclusions

The current Dutch approach towards site investigation for linear infrastructure almost completely ignores geological factors. Differential settlements induced by unforeseen subsurface heterogeneities thus contribute to the considerable failure costs in the Dutch civil engineering sector. Knowledge of the local geology and geomorphology, remote sensing and geophysical surveys is necessary to guide and complement the campaign of borings and CPT's. This knowledge is available, but human, procedural and institutional factors prevent its effective application. An improved approach can be based on the Total Engineering Geology Approach (Baynes et al., 2005). Expert knowledge can be made available by two methods available at TNO-NITG and Deltares. The Geo-Impulse project Reliable Subsurface Model will generate knowledge and procedures to implement the improved approach, and provide tools to support and guide the end users.

Editorial note: this article has been presented at the First international conference on Frontiers in Shallow Subsurface Technology, Delft, The Netherlands, January 20-22, 2010.

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Laboratory news: nanotom®, a high-resolution nano CT-scanner

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Introduction

The nanotom® forms part of the product line *phoenix|x-ray* of GE Sensing & Inspection Technologies, for high-resolution 2D X-ray inspection and 3D failure analysis, using Computed Tomography.

Computed Tomography (CT) x-ray imaging in 3D plays an important role in material research nowadays. At our department we started with Computed Tomography in 2002, on data collected with a Siemens SOMATOM medical CT-scanner. However, due to the limited resolution of the medical scanner (0.3 x 0.3 x 1 mm), the need was growing to visualise more detailed structures. In 2008, a combined investment of six sections of the departments of Design & Construction and Geotechnology made it possible to buy a high-resolution nano CT-scanner.

Fig. 1 nanotom® from phoenix|x-ray

Operation principles and specifications

The nanotom® (see Figure 1) is using an X-ray cone beam creating two-dimensional X-ray images while progressively rotating the sample through a full 360° rotation (see Figure 2). These projections contain information on the position and density of absorbing object features within the sample. This accumulation of data is used for reconstruction of the volumetric data. The main components of the scanner and the scanning process are:

- **High-power nanofocus X-ray tube** (high voltage (up to 180 kV), high power (up to 15 W), with a minimal focal spot size <0.9 micron).

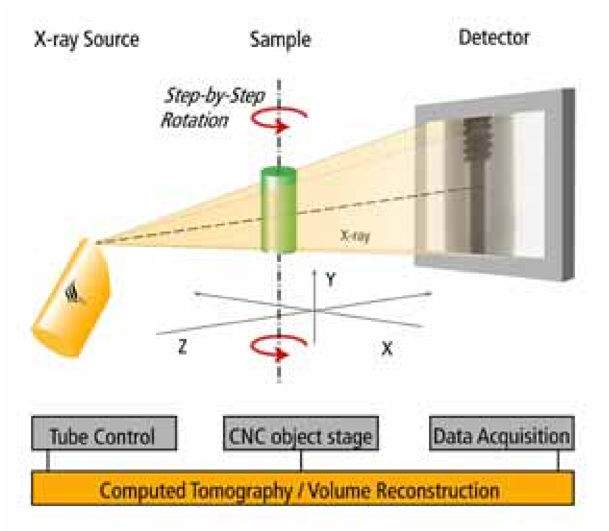


Fig. 2 Operation principle of the nanotom®

- **X-ray digital detector** (12 bits resolution, 2,200 x 2,200 pixels (5 megapixel), field of view appr. 110 x 110 mm, max. resolution (depending on object size) <0.5 micron (in all directions)).
- **Sample** (maximum sample mass about 1 kg, maximum sample size is about 120 mm across).

The reconstructed 3D volume shows object features in grey values based on the differences in material density. It provides three-dimensional images at microscopic resolution of rock samples, binders, cements and cavities and can help identify certain sample characteristics such as size and location of voids in oil-bearing rock. Figure 3 gives a cross-

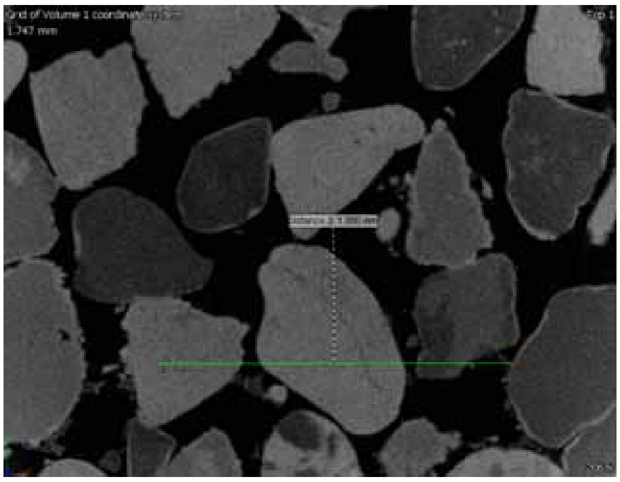


Fig. 3 Cross section through a weakly cemented sandstone

section through a 3D reconstructed body of weakly cemented sandstone. Grains with different grey values are visible: light grains are lime based, dark grains are quartz based. Some of the quartz grains are surrounded by a light coloured layer, which is calcite (this is the cement between the grains).

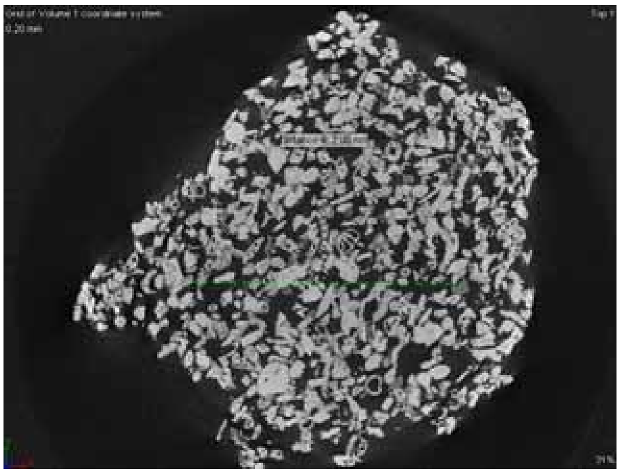


Fig. 6 Cross section of limestone from Valkenburg (picture width is about 4 mm)

Use of the nanotom® in practice

The highest resolution is achieved using samples with a maximum size of about 2 mm. Examples of this quality are given in Figure 7 and 8, showing a 3D view and a cross section of a foraminifera with a diameter of about 1 mm, scanned and processed by Arjan Thijssen. An example of Maastrichtian limestone with a porosity of about 40 % is given in Figure 6.

For more dense material like road pavement and concrete the maximum sample size is, due to the needed X-ray penetration, limited to a maximum of about 45 mm. A cross section of a noise-reducing top layer of pavement is given in Figure 4. Powerful segmentation software makes it possible to separate the different materials inside a sample. Figure 5 gives a 3D view of the connected voids in the pavement

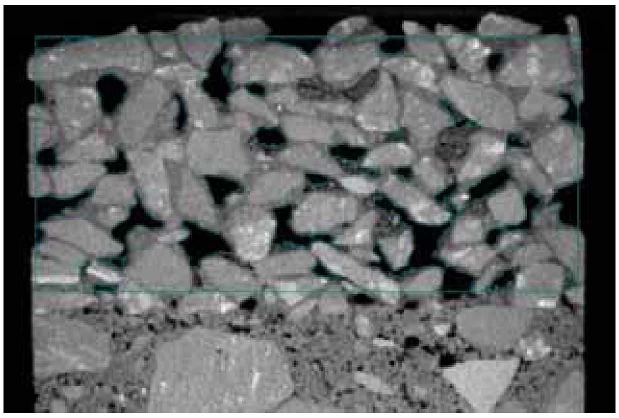


Fig. 4 Cross section through a top layer of noise-reducing pavement

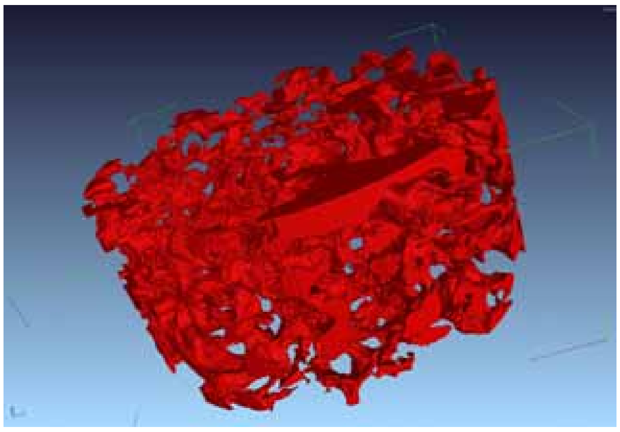


Fig. 5 3D view of the skeleton of connected voids in the pavement sample of Figure 4

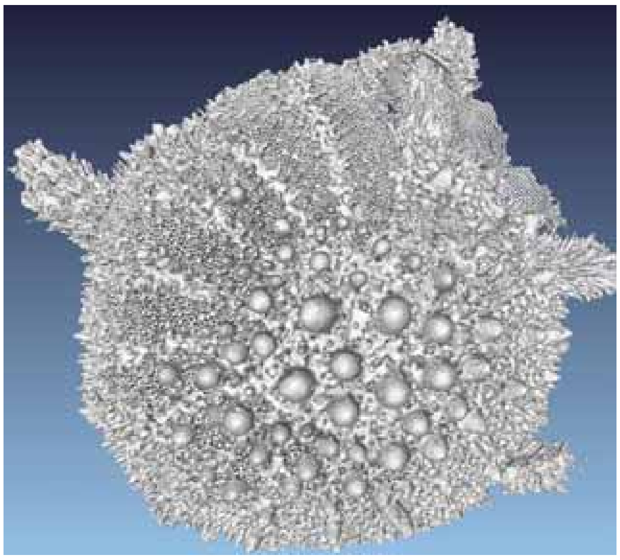


Fig. 7 3D view of a foraminifera with a diameter of about 1 mm

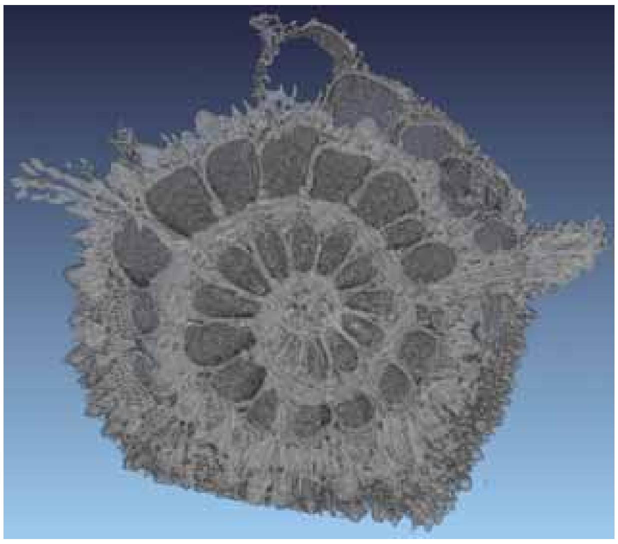


Fig. 8 Cross section of the foraminifera of Figure 7

Book review

P.G. Fookes, E.M. Lee & J.S. Griffiths: *Engineering Geomorphology. Theory and Practice*

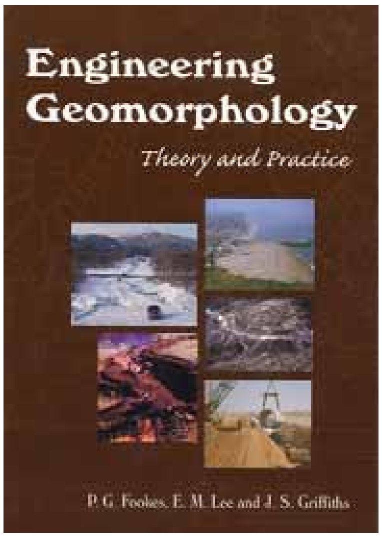
Leon van Paassen

Geomorphology is the study of natural processes which shape Earth's surface and create landforms. As all engineering works are built on (or just underneath) the surface, geomorphology is a very relevant discipline for civil engineers. In this book the authors provide a good overview of the basic concepts of geomorphology, share their vast expertise and provide many examples where knowledge of geomorphology is applied in engineering projects.

The book is divided into five parts. Part 1 covers the basic geomorphological concepts that are used to explain the causes, mechanisms and consequences of landform change. The next three parts focus on slopes, rivers and coasts and provide many empirical relationships and classification methods which are used to estimate hazards, define ground conditions or provide resources for civil engineering works. The final part of the book describes common site investigation techniques that are used to assess the impact of geomorphology on engineering works.

The book is well referenced to the latest publications and nicely illustrated, including coloured sections with the well-known terrain models of professor Fookes. The book is useful for students and practitioners in civil engineering and is well recommended as an introduction to the field of geomorphology or as a reference guide.

Still the question remains whether engineering geomorphology should be considered as a separate discipline or be a fully integrated part in closely related disciplines such as geology, geotechnical engineering, soil mechanics or engineering geology. Although the book provides methods to predict the rate of landform change and associated hazards, it seems to lack the information to turn qualitative information about the different types of soils into quantitative data, which is used by geotechnical engineers to design their constructions, such as soil strength and deformability or the difference between rock mass and rock material properties. As the book focuses mainly on soils in slopes, rivers and coasts, it misses information required for designing foundations, tunnels and slopes in rock or for offshore constructions, while the oceans and mountains make up about 80 % of Earth's surface. Probably this is the domain of the engineering geologist or geotechnical engineer.



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Determination of soil stiffness properties

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Abstract

Good knowledge of soil stiffness properties is very important for geotechnical engineering. Stiffness parameters are required as input for many geotechnical calculations related to settlements or horizontal deformations. Since the latest decennia the available calculation methods are getting more and more sophisticated. Finite element methods like PLAXIS are nowadays widely used, as they have become accessible not only for specialised engineers, but for common practice as well. As a consequence, the need for good methods for the assessment of soil parameters is growing strongly. Soil stiffness parameters can be determined by in situ testing or by laboratory testing. Especially for in situ testing, a wide scope of different tests is available worldwide. However, the choice of test type depends strongly on the geotechnical tradition of a country. The use of foreign methods in a country is now stimulated by the development of international standards for in situ and laboratory testing. This paper deals with a large scope of tests. For each test a short description of the test principle is given, together with an example of a test result and the field of application. Furthermore, the practical advantages of each test are evaluated.

Introduction

When making an assessment of stiffness parameters of a soil, the first stage should be to make a soil model consisting of a number of layers with different soil stiffness properties. This soil model can be generated quickly by using CPT's (Cone Penetration Tests) and/or borings with sampling. The second stage consists of determining the parameters of the different layers by in situ and/or laboratory testing.

On or more of the following test methods may be used:

In situ testing

- Ménard pressuremeter test
- Cone pressuremeter test
- Cone load test
- Marchetti dilatometer test
- Seismic CPT
- Plate bearing test

Laboratory testing

- Oedometer test
- Triaxial test

Assessment by correlation from CPT

Description of different in situ testing methods

Ménard pressuremeter test

Developed in France by Louis Ménard more than 50 years ago, it is nowadays the most common in situ testing technique for all purposes in France: foundation design, settlements, retaining walls, etc. More than 1,000 rigs are carrying out 15,000 tests annually, with a vertical spacing between tests of 1.5 m. Outside of France the test is mostly used for special projects only. The principle of this test is that a measuring cell (height 200 mm, diameter 60 mm; see Figure 1) is inflated with water (allowing measurement of injected volume) so that its diameter is increasing while the soil is pushed away horizontally. Above and below this measuring

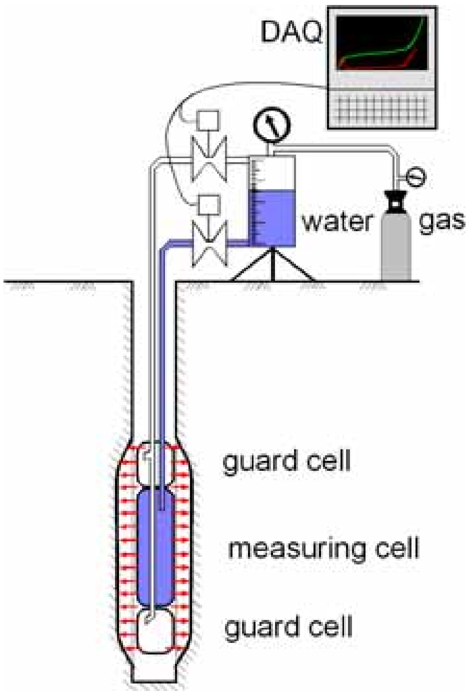

Fig. 1 Ménard pressuremeter equipment

cell guard cells are positioned that expand using gas. This results in a *plain strain* measurement condition, which makes the interpretation of test results easier. Inflation of the three cells is performed stepwise in 10 stages until 'failure', which is defined as an increase in diameter of the measuring cell until twice the initial volume. At each stage the volume of water in the cell is measured, resulting in a curve (see Figure 2).

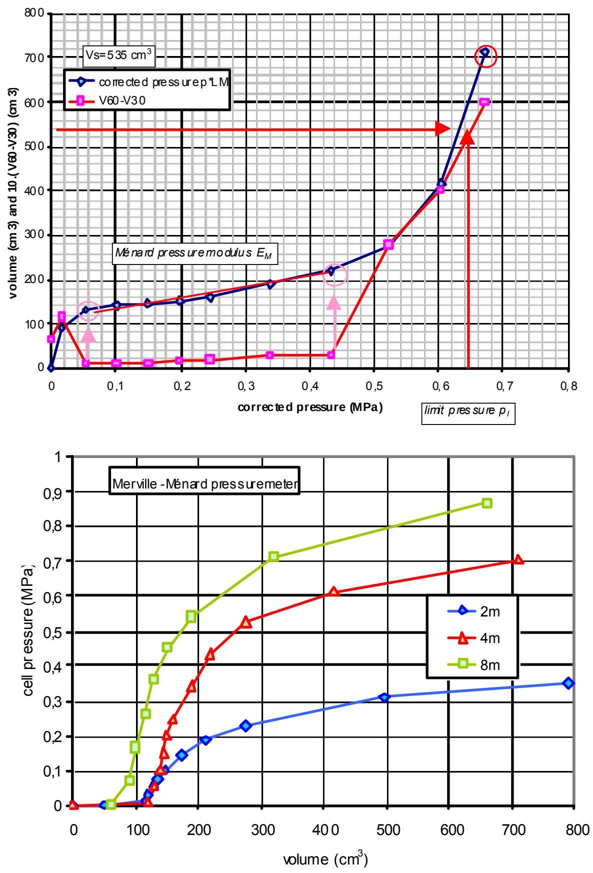

Fig. 2 Ménard pressuremeter curves

From this curve the following parameters can be derived:

- Limit pressure P_l
- Creep pressure P_f
- Pressuremeter modulus E_M

The pressuremeter modulus E_M is derived in an empirical way from the slope of the load-displacement curve. These parameters are usually presented related to depth in a pressuremeter log, see Figure 3.

The preferred installation method is to lower the pressuremeter cell in a pre-drilled open hole. Where necessary, the stability of the hole is assured by using bentonite mud.

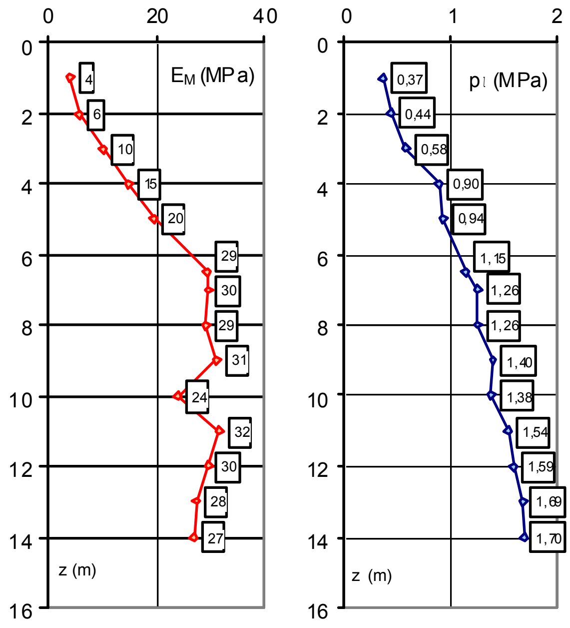

Fig. 3 Ménard pressuremeter logs: pressuremeter modulus (left) and limit pressure (right)

When hole stability problems are expected, the cell is installed in a slotted tube and hammered into the ground. The slotted tube is rather flexible regarding dilatation. As the soil is pushed away when penetrating with the slotted tube, the soil has already been subject to displacement when starting the actual test. This inconvenience can be overcome by using the so-called STAF-method, in which the slotted tube is emptied by inside drilling during its penetration. This is therefore a *self boring* pressuremeter installation technique, but cheaper and with limited risks.

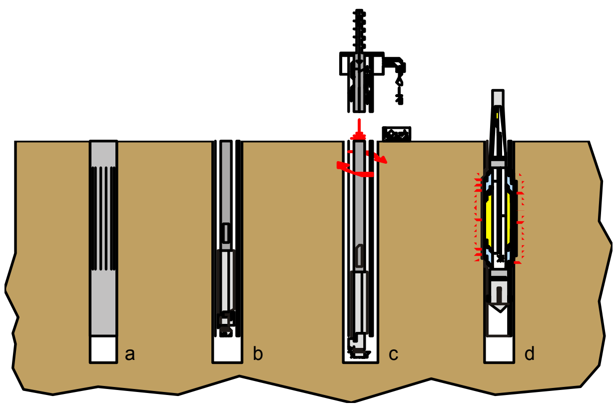

Fig. 4 Using a pressuremeter in a Chinese lantern inserted by self boring: a) Chinese lantern, b) lowering the asymmetric tool, c) rotary percussion drilling, d) positioning the 44 mm probe in the casing and performing the expansion test

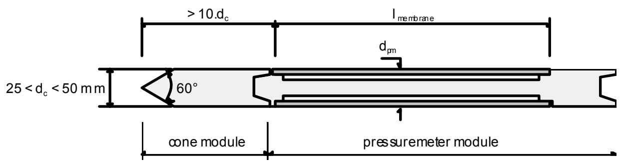

Fig. 5 Cone pressuremeter probe

Cone pressuremeter test

About 20 years ago the first commercially used cone pressuremeter equipment was introduced on the market by Fugro and Cambridge In-Situ Ltd. The major aim was to include a load-displacement testing modulus in a CPT test. The pressuremeter cell is located just above the friction sleeve (see Figure 5). At depth intervals of e.g. 1 m, the CPT test is interrupted for carrying out a pressuremeter test. The loading procedure is not stepwise like the Ménard pressuremeter, but done by a constant flow of water injected into the cell which results in a constant rate of dilatation. An-

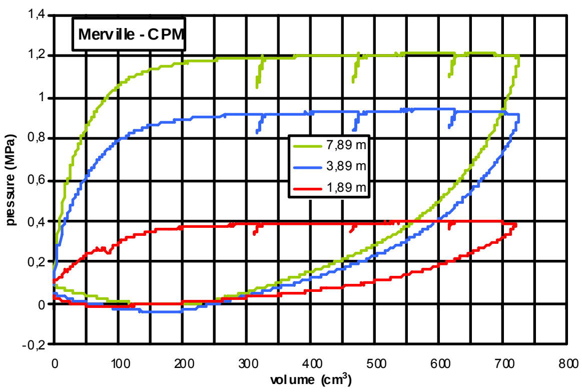

Fig. 6 Cone pressuremeter curve

other difference from the Ménard test is that there is only one cell (no guard cell). Like the normal slotted tube version of the Ménard test (which is also a *full displacement pressuremeter test*), the cone pressuremeter test starts after displacement of the soil by the penetration of the probe, so the first part of the load-displacement curve is missing. Therefore an unloading and reloading loop is included in each expansion test. Different methods are in use to assess the E-modulus from the slope of the curve.

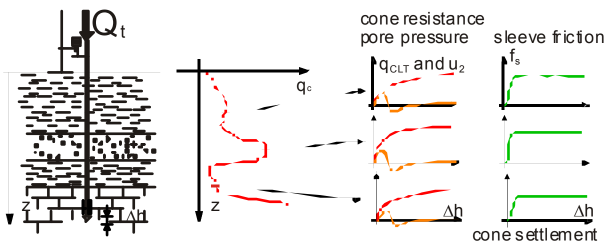

Fig. 7 Principle of the cone load test

Cone load test

The principle of this test is to interrupt a CPT test at a certain depth in order to carry out a load test on the cone while measuring the vertical displacement of the cone. Like with the Ménard pressuremeter test, the load is applied in about 10 stages until failure. The development of this test started 40-50 years ago and was given a new impulse in 2005 by Ponts et Chaussées (Paris), Blaise Pascal University (Clermont-Ferrand), Lankelma and Fondasol, by using an electrical CPT test and a 20 tons CPT rig. From the slope of the different parts of the curve the E_{CLT} and the E_{50CLT} are derived. The test method has been used for different projects in France and in The Netherlands, including validation of the test results by correlating the obtained E-modulus to the E-modulus of triaxial tests and Ménard pressuremeter tests. Further validation will be done at other projects.

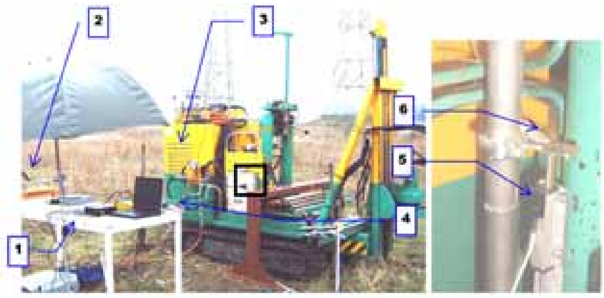

Fig. 8 Cone load test set-up: 1) device used to visualise and record the parameters during the cone load tests and penetration, 2) pump, 3) connection of the hand pump on the hydraulic system of the CPT, 4) rigid structure anchored in the ground, independent of the thrust machine, 5) displacement transducer, 6) plate attached to the push rods for measurement of displacements

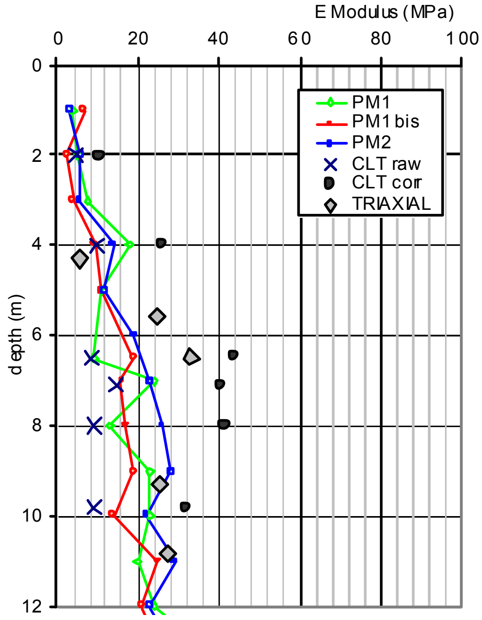

Fig. 9a Cone load test curves

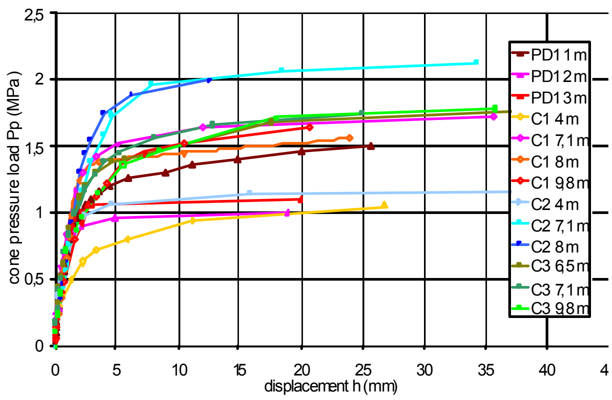

Fig. 9b Cone load test curves

Marchetti dilatometer test

This test was developed in Italy by Sylvano Marchetti, some 40 years ago. The test equipment consists of a flat spear equipped with an inflatable membrane, which is pushed into the ground. Pressure readings are taken before inflation and at 2 mm membrane displacement intervals. From these readings the dilatometer modulus and other parameters are derived. This method is used worldwide on a limited scale in soft soil.

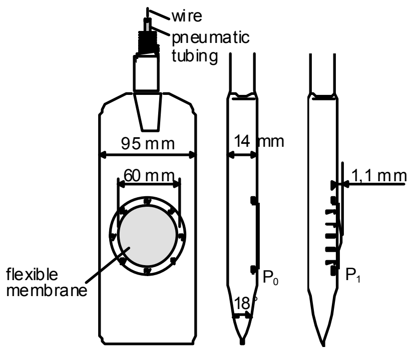

Fig. 10 Marchetti dilatometer probe (top) and pressure control and read-out unit (bottom)

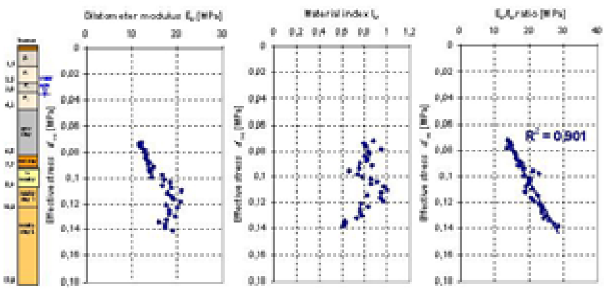

Fig. 11 Dilatometer results

Seismic CPT

This is a standard electrical CPT cone for measuring cone resistance and local friction, but which also includes one or more geophones. At every meter the CPT test is stopped for a seismic measurement. For this measurement a compression wave and/or shear wave is generated at soil surface by a hammer. The time elapsed between the impact of the hammer and the arrival of the wave at the cone is measured. These measurements result in a diagram of wave velocity versus depth. The small strain G-modulus is derived from this velocity:

$$G = \rho . V_s^2$$

The same method can be used by lowering the geophones in a borehole. The method is more and more used worldwide.

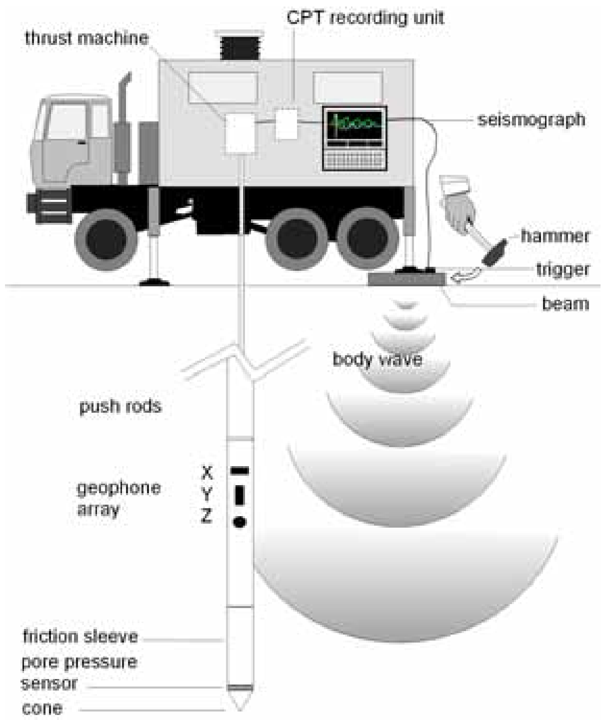

Fig. 12 Principle of the seismic CPT

Fig. 13 Plate bearing test equipment

Plate bearing test

This method consists of stepwise static loading of a plate. The E-modulus is derived from the slope of the load-settlement curve. As settlement under the plate is unknown it's not a Young's modulus. The method is often used for the compaction control of natural soil or filled materials for shallow foundations or road construction.

Description of different laboratory testing methods

Triaxial test

This is a compression test carried out on a soil specimen, while the specimen is confined in a cell. The first version of this testing method was the Dutch *Cell Test*, developed more then 80 years ago by Keverling Buisman. Some decennia later the first triaxial tests were carried out in the United Kingdom. Nowadays, the cell test has disappeared due to international standardisation. The primary aim of the triaxial test is to determine the shear strength or the internal angle of friction ρ and the 'cohesion', or undrained shear strength. Over the latest decennia the test is done more and more to determine also the E-modulus (Young's modulus) from the load-settlement curve.

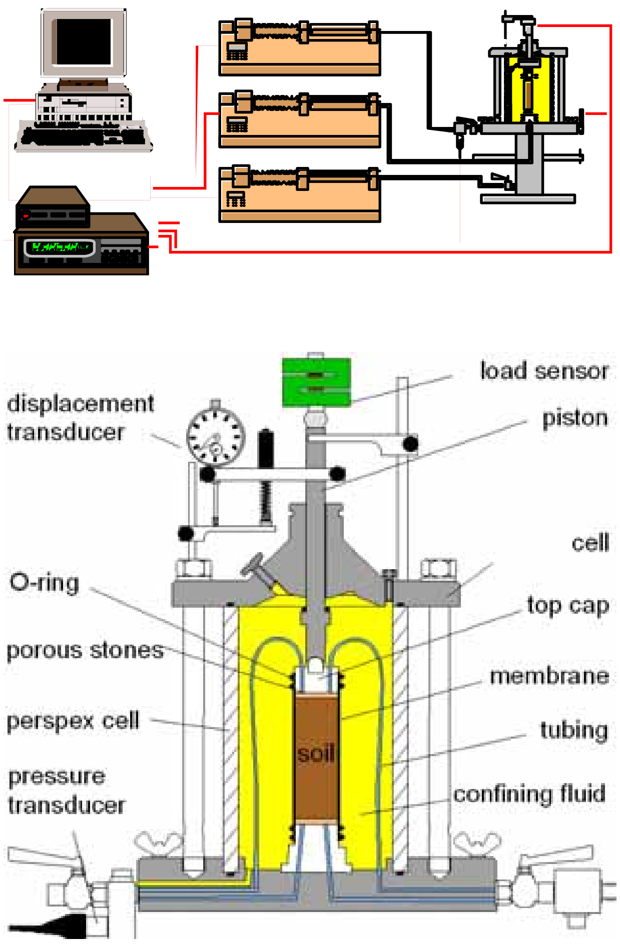

Fig. 14 Principles of triaxial testing equipment

There are different testing procedures available:

- Drained or undrained
- Consolidated or unconsolidated
- Isotropic or anisotropic consolidation
- With or without unloading stage
- Static or cyclical loading
- Small strain testing using dynamic loading such as bender elements or resonant column
- One stage, single stage (three different specimen, consolidated at different stresses, each loaded until failure) or multistage (one specimen, consolidated and loaded at three different stresses, at the first two stages loading is stopped long before failure at small deformation)

In The Netherlands the number of triaxial tests carried out yearly is increasing strongly since 10 years because of the need for input parameters for finite element computations. This test allows reproducing the stress history followed by a soil element close to a structure.

Simulation of a path starting with a K_o-consolidation and then a decrease in deviatoric stress, like during excavation and reloading during construction of a structure.

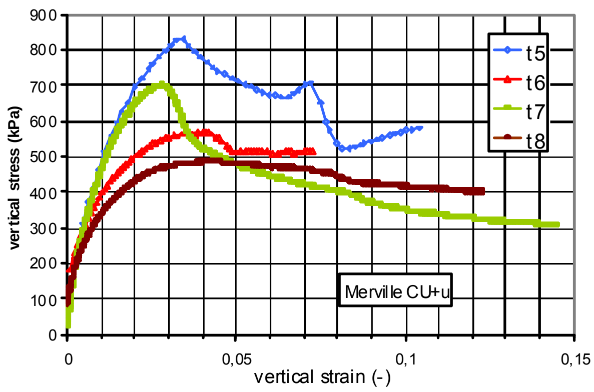

Fig. 15 Deformation of a specimen under triaxial loading

Oedometer test

The first oedometer tests were also carried out 80 years ago in The Netherlands by Keverling Buisman. The purpose of the drained test is to determine the time-settlement behaviour of soil. The test specimen in this test is contained in a stiff ring (see Figure 16), therefore the deformation of the specimen is completely different from triaxial testing. Usually the loading is done stepwise, the resulting stiffness modulus E_{oed} is a constrained modulus, which is different from a Young's modulus. In some countries, like The Nether-

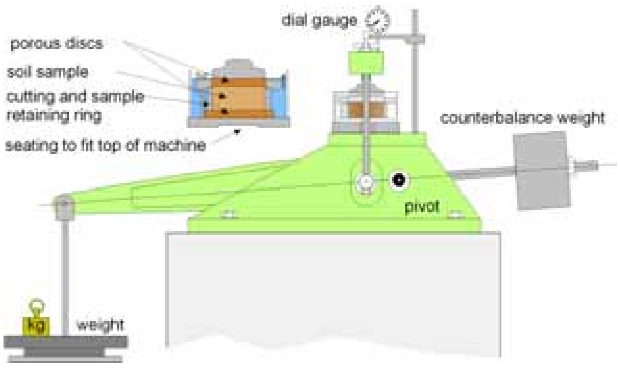

Fig. 16 Oedometer equipment

lands, the stiffness of the specimen is not expressed as an E-modulus (strain vs. stress), but as strain vs. the logarithm of stress (*C*). This is because for soil, *E* strongly depends on the considered strain level. Since a number of years the test is also carried out at a Constant Rate of Strain (CRS test), often combined with the measurement of radial stress.

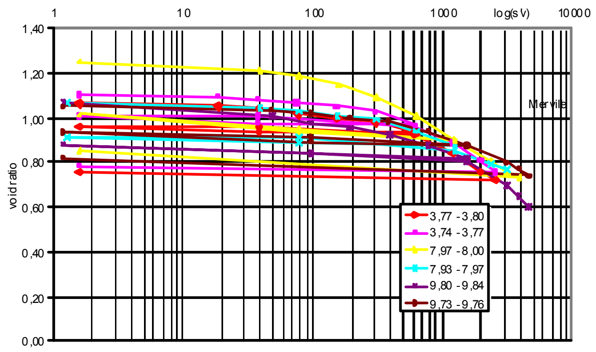

Fig. 17 Deformation of a specimen under oedometer loading

Correlation between soil stiffness and cone resistance

In literature, relationships between cone resistance and soil stiffness can be found (e.g. Baldi et al., 1989), see Figure 18. Table 1 is included in the international standard Eurocode 7 for geotechnical design (BSI, 2007). Please note that such correlations only give a rough indication. When relevant, for each site a local correlation should be established by carrying out specific in situ and/or laboratory testing. The CPT results may be used for the establishment of the soil model and for the selection of the depth of tests. In this way the test results will be more or less representative for the considered soil layer, even when the test is done on a small specimen.

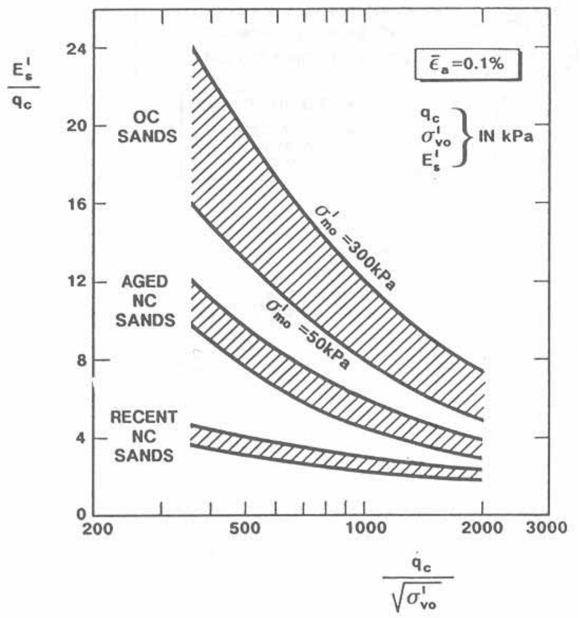

Fig. 18 Example of a relationship between cone resistance and soil stiffness (Baldi et al., 1989)

International standards

Since 20 years a number of European committees are working on international standards, replacing national standards. The most important standard is Eurocode 7 for geotechnical design (BSI, 2007). In this standard it is specified which specific test can be used for a certain type of geotechnical engineering project.

Table 1 Relation between cone resistance and soil stiffness

Density Index	Cone resistance (q_c) from CPT [MPa]	Effective angle of shearing resistance (φ') [°][a]	Drained Young's Modulus (E') [MPa][b]
Very loose	0.0-2.5	29-32	<10
Loose	2.5-5.0	32-35	10-20
Medium dense	5.0-10.0	35-37	20-30
Dense	10.0-20.0	37-40	30-60
Very dense	>20.0	40-42	60-90

a) Values given are valid for sands. For silty soil a reduction of 3 should be made, for gravels 2 should be added

b) E' is an approximation of the stress and time dependent secant modulus. Values given for the drained modulus correspond to settlements for 10 years. They are obtained assuming that the vertical stress distribution follows the 2:1 approximation. Furthermore, some investigations indicate that these values can be 50 % lower in silty soil and 50 % higher in gravelly soil. In over-consolidated coarse soils, the modulus can be considerably higher. When calculating settlements for ground pressures greater than 2/3 of the design bearing pressure in ultimate limit state, the modulus should be set to half of the values given in this table

Other standards are dedicated to a testing method. For the tests mentioned in this article, the following standards have been prepared by different working groups of TC 341:

Table 2 Standards for different test methods

Test type	Document number	Status
CPT (mechanical cone)	CEN-ISO-22476-12	Published (EN ISO 2009)
CPT (electrical cone)	CEN-ISO-22476-1	Final draft (2009)
Drilling and sampling	CEN-ISO-22475-1	Published (EN ISO 2006)
Ménard pressuremeter	CEN-ISO-22476-4	Draft (2007)
Cone pressuremeter	CEN-ISO-22476-8	No draft available
Marchetti dilatometer	CEN-ISO-22476-11	Published (TS 2005)
Seismic CPT	-	-
Plate Loading Test	CEN-ISO-22476-13	Draft (2005)
Oedometer	CEN-ISO-17892-5	Published (TS, EN ISO 2004)
Triaxial test	CEN-ISO-17892-8 CEN-ISO-17892-9	Published (TS, EN ISO 2004) Published (TS, EN ISO 2004)

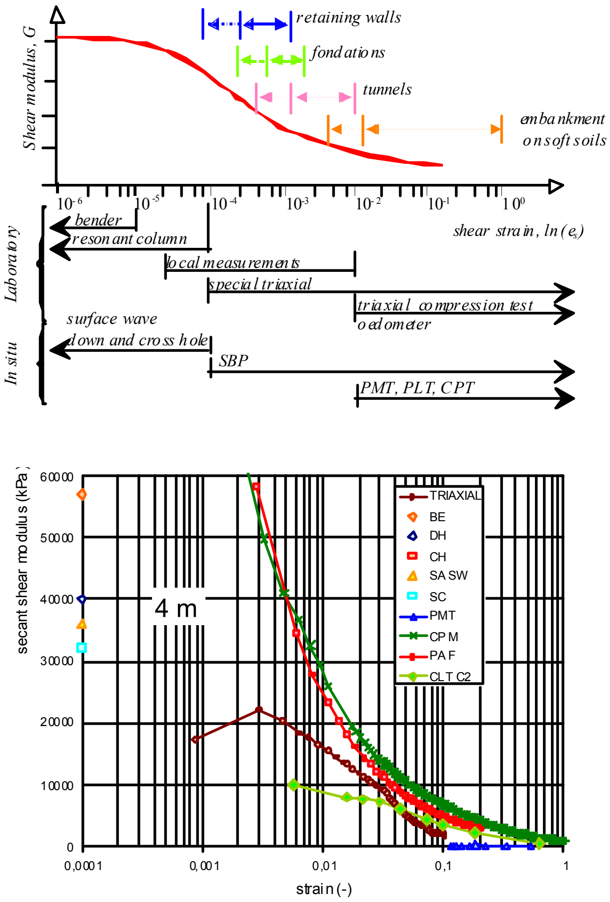

Fig. 19 Typical variation of stiffness with strain for most soils

Evaluation of different methods

When selecting a testing method for obtaining stiffness parameters and when applying the test results in a computation, the following factors should be taken into account:

- The strain level during the test related to the strain level of the calculation of the deformation: the smaller the strain, the higher the stiffness.
- The stress level of a test may be completely different from the stress level in calculations.
- The E-moduli resulting from the different tests are not automatically Young's moduli, so they cannot be directly used as input for all computations.
- Drilling prior to in situ testing or sampling as well as the sampling operation itself may have disturbed the sample, even when the best available techniques are used. This disturbance is even a *certainty* when sampling cohesionless sand below the groundwater table.

In Tables 3 and 4 on the next page, information on field- and laboratory testing as described in this article is summarised.

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Table 3 Summary of field test methods

Type	Test	Stress history/initial state parameter			Deformation characteristics		Strength parameters		
		σ_{ho}	K_o	σ_p	E	G_{max}	c	φ	Drained
Penetration tests	SPT				C		C		
	CPT				C		C		
Plate load test	On surface or embedded in borehole			D	T				
	Cone loading test				C				
Expansion tests	Pressuremeter	D					C	C	
	Self-boring pressuremeter	D	T		C	D	C	C	
	Cone pressuremeter				C		C	C	
Other tests	Seismic cone				T	T			

D: direct measurement; T: theoretically deduced; C: empirical

Table 4 Summary of laboratory test methods

		Stress history/initial state parameter			Deformation characteristics		Strength parameters		
Type	Test	σ_{ho}	K_o	σ_p	E	G_{max}	c	φ	Drained
Consolidation tests	Incremental loading			D	T				
	Constant rate of strain	D		D	T				
	K_o	D	D	D	T				
Triaxial tests	Conventional				T		T	T	T
	Local measurement	D	T		T	T	T	T	T
Cyclic and dynamic triaxial tests	Resonant column	D			T	T			
	Bender elements	D			T	T			

D: direct measurement; T: theoretically deduced; C: empirical

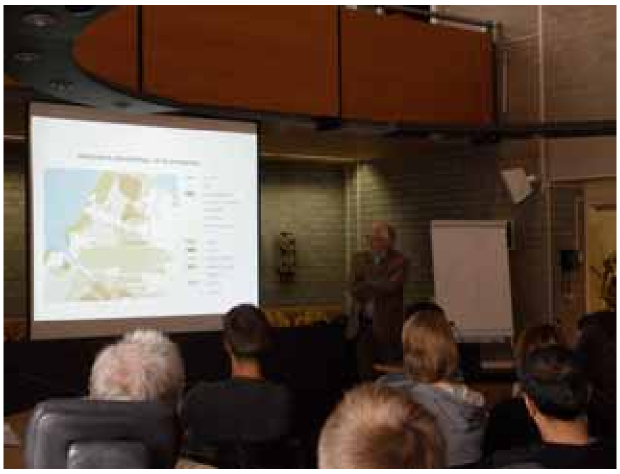
Ingeokring excursion: pity for peaty dikes in Reeuwijk

Dominique Ngan-Tillard & Michiel Maurenbrecher *(Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Comfortably installed in the seats of the Reeuwijk municipal counsel chamber on a sunny Thursday afternoon in October 2009, a group of about 30 participants was introduced to the local environment and the objectives of this Ingeokring excursion: the presentation of test sites selected in the framework of the national 'Flood Control 2015' research program, in which a consortium of various institutions and engineering firms investigates the potential of remote sensing techniques for dike quality assessment in the 'mining fields' north of Reeuwijk in what are now the polders of Middelburg and Tempel.

To describe what the project was all about, Ingeokring chairman Joost van der Schrier gave an introduction and explained how the Dutch built, or rather mined and burned, their country. Joost explained the function of the primary, secondary and tertiary flood protections. He stressed upon the continuous need to dewater Dutch polders as dewatering for agricultural use of the land causes oxidation of peat above the groundwater table, peat shrinkage, land subsidence, and flooding which in turn requires dewatering. He also recalled the famous (or infamous) peat dike failure in Wilnis, which occurred during the very dry summer of 2000 (Van Baars, 2005). In fact, the dry summer was a good year for failures as they had also occurred near Delft, where many of the institutions participating in the research reside or used to reside, this occurring in what is probably also a mining area. Dry summers lead to peat dike failures? Joost showed some simple models of the mechanics of failure that apply to dikes, including those of peat. A dike retains water usually in a canal which may seep through or beneath the dike to the lower levels of the polder area. Seepage may occur at these lower levels causing an increase in pore water pressures at the toe of the dike or beyond the toe of the dike into flat polder area usually in a moat running parallel to the dike. This reduces the shear strength which can cause potential failure. One would expect more adverse situations in a wet summer rather than a dry summer. Yet, in Wilnis the failure happened in a dry summer. Just to emphasise the need for investigation, Joost insisted on the necessity to maintain the 4,000 km of peat dikes that form part of the secondary and tertiary dikes in order to keep our feet dry, disregarding whether or not the weather gets wetter or drier. He illustrated this by showing the doomsday map of The Netherlands if too many dikes would fail causing flooding of all the polders, leaving very little of the country dry. Concluding his presentation, Joost linked cost-effective maintenance to dike quality assessment and wished that airborne remote sensing techniques combined with geological information could provide low cost information for dike stability assessment. His introduction was useful as the public was cosmopolitan and not familiar with the formation and management of peaty lands.

The second speaker, Jan Rupke, emeritus professor from the University of Amsterdam and alderman *(wethouder)* of Reeuwijk municipal council, lectured on land subsidence. He outlined the interplay of marine and fluvial sedimentation from cold glacial to warmer interglacial periods which we find ourselves in at present. The massive load of the ice sheets in Scandinavia caused the land there to depress, just like the ice sheets extending all the way south into The Netherlands (to latitudes as far south as present Reeuwijk).



Jan Rupke during his presentation in the municipal counsel chamber of Reeuwijk

The ice sheets being thinner and of less duration here did at first cause the land to rise until the Scandinavian shield melted. Then, the unloading and rebound of Scandinavia caused the southern areas to sink ('isostatic balance'). Combined with sea level rises this had a major influence on drainage patterns. The ancient Rhine was diverted to drain further south during the big freeze. With the thaw setting in, one of its major tributaries followed the present *Oude Rijn* (Old Rhine) about twenty kilometres to the north of Reeuwijk. So what was inherited from all this geological activity: rise and fall of land levels, diverted rivers and rise in sea levels from meltwater? The delta of today would not be recognised by Neanderthal men who occupied the area until about 10,000 years ago and may have lived here through its many changes of the previous glacial and interglacial periods (a Neanderthal jaw bone was excavated recently by dredgers in the North Sea. Coincidently, the first skeletal remains of this now extinct human species were found in

the Neanderthal valley in Germany. The Neander River is an upper tributary of the Rhine in the Ruhr region and has its confluence with the Rhine south of Düsseldorf). During the Holocene, our own ancestors of the Bronze Age would see the swamps and meandering Rhine causing the creation of peat deposits and, through sedimentation, the levees along the *Oude Rijn*. The levees consisting of sand are much more favoured for urban settlements, with buildings often having basements. The swamps in the lagoons behind the beach barriers became areas of energy resources allowing the more nutrient rich silts to settle and allow growth of luxuriant flora especially in the warmer interglacial reaching its peak about 50,000 years ago.

During his talk, Jan further highlighted the inter-fingering of river and marine deposits in the area. In Reeuwijk, Holland peat is underlain by marine clays of the Calais Formation that overlay intercalations of Calais tidal flat clays and Holland peat or Gorkum sand river channels. Jan focussed on the intrinsic link between geomorphology and geology and pointed on a geological map at old Gorkum sand channels on which ancient roads and farms are founded. He further showed the Reeuwijk lakes on the map, the shallowest lakes being 2-3 m deep. They occupy locations where Holland peat was dredged and land has not been dewatered afterwards. The deepest lakes are more than 30 m deep. They were dredged in fields suffering from chronic seepage and dike failure, in order to provide sand for the construction of the A12 motorway in the mid sixties of the previous century and more recently for the construction of the N11 national road. Today the lakes have a high recreational value, somehow in conflict with measures taken to protect the local fauna and flora. Before concluding, Jan captivated the attention of the public with a curious phenomenon apparently triggered by sand winning: polder salinisation and sand boiling. It is thought that dredging allowed fresh water to circulate through marine clays via buried sand channels, to leach their salt and re-emerge as a sand boil in the low laying polders east of Reeuwijk: the Middelburg and Tempel polder.

When Jan drives back from The Hague with his municipal colleagues (after meeting with the society of municipalities) he likes to surprise them with observations such as "we are now entering the first area of extensive mining in The Netherlands!". They have hardly left The Hague and are in the polder area leading up to Zoetermeer. Here was a lake which was drained allowing first farming and then urbanisation. This is the lagoonal area behind the many beach barriers extending approximately west from Voorburg to possibly a number of kilometres out to sea from Scheveningen. The lagoons filled up with plant debris and fine silts and clays during overflow of the old river Rhine to the north during the spring floods (from snow melts in the Alps) so they became bogs probably similar to the Biesbosch area of the major present day Rhine estuary south of Dordrecht. As the weather became colder and modern Netherlands took shape, the forests were cleared for farming and trees were used for building and firewood. When this resource diminished the next stage was extraction of peat. The process was quite straightforward: skiffs were loaded with excavated peat extending the lake thus formed from what used to be a creek and previously a tidal channel. Pathways connecting hamlets in the bog area remained eventually, due to the 'mining activity' they became tracks on remnants of the original land surface. These were underlain by remnant peat deposits later to be paved probably with brick and its slopes with protective clay-grass (reeds for underwater slopes) cover. In this way the 'peat dikes' were formed. Often the remnants being these dikes of the original land serving as tracks or boundaries between the owners of the peat excavation areas.

The third speaker, Robert Hack, explained how remote sensing can be used for the check of kilometres of peat dikes. He asked the public to imagine the minister of Public Works in a control room, facing an illuminated map of The Netherlands lying under sea level. Time lapse remote sensing data recorded on board of jet fighters flow to the room. Where the data points to a dike on the verge of collapse, the light bulb switches to red and the minister orders evacuation of local population. Before such a scene becomes reality, the suitability of remote sensing data in detecting vulnerable dikes must be investigated. Robert explained briefly the physical principle of most promising techniques: airborne laser scanner surface altimetry, gamma ray, thermo-infrared, visual light, and multi-spectral near-infrared. For example, changes in thermo-infrared signals are correlated to changes in temperature and groundwater content while changes in multi-spectral near-infrared signals reveal changes in ground type, vegetation and groundwater content. To remain healthy plants need enough water, not too much, not too little. If their roots get drowned in water, plants loose their green colour, become yellow and die. During this degradation process due to excess water, their infrared response changes as it would also change due to shortage of water or contamination by vegetation, unfriendly minerals and fluids.

Then, Robert presented preliminary results obtained at different locations in Reeuwijk and showed striking differences between patterns recorded along the same dike: the Tempel dike. On the one hand, the northern extremity of the dike is in good condition and associated to remote sensing data organised parallel to the dike. On the other hand, the southern extremity shows signs of collapse and is characterised by a patchy pattern of remote sensing data. All types of remote sensing data show the same trend in their pattern. To correlate surface observations to subsurface processes, cone penetration testing and a 3D multi-electrodes geo-electric survey were conducted. Not less than 30 CPT's were pushed in a zone of 30 x 45 m and two Begemann boreholes were made for a better interpretation of CPT parameters in terms of soil type. While the CPT's and boreholes show no anomaly, the geo-electric survey highlighted a strange high resistivity saucepan-like structure at 6 to 10 m deep. Robert tentatively interpreted it as a sand boil in formation. In the public, professor Molenkamp asked whether it could be a large gas pocket. It was concluded that further investigation was needed. After a series of questions on correlation between strength and deformation, correlation between strength parameters and geo-electric parameters, and correlation between the dynamics of dike failure and vegetation stress, Peter Verhoef thanked the speakers and insisted on the need to include local knowledge on site history in dike quality assessment and invited the excursion participants to a site visit.

The participants drove in a convoy along Reeuwijk's narrow roads towards Reeuwijk-Dorp. Modern Reeuwijk is a combination of lakes and polders, both remnants from the peat mining period. The lack of windmills in the landscape suggests the polders are peat lagoons that have been drained using steam pumps (which were first used to drain mines in the UK). Coal (both lignite and anthracite) started to become the substitute for peat fuel in The Netherlands when peat production diminished. The coal came from south Limburg until that also ended in 1970 when natural gas from Groningen became the principal energy source.

First, the group stopped at a tertiary dike, the Vreesterdijk, along a small road that is now one-way but was leading in the past to Reeuwijk-Dorp. The dike is a remnant of Holland peat. On both sides, three meters of peat have been excavated. Jan Rupke explained that excavating deeper was not feasible at the time. The peat was dredged from a boat with a spade attached to a wooden stick. It was then loaded on the boat and discharged in a field where it was left to dry before being cut into blocks (*turf* in Dutch). The blocks were then sold as combustibles. Here and there the dike has been raised to win the race against subsidence caused by dewatering. Jan Rupke also explained the step-by-step pumping of water from the many ditches that drain the polder. The Vreesterdijk gave the visitors a good impression of height differences in the lowland country.

At the second stop, the group climbed across a couple of fences to step on the north extremity of the Tempel dike.

Here the dike is in good condition. Remote sensing images show a vegetation cover organised in bands parallel to the dike. Tracks left on the soft soil by tractors used to cut the grass could explain the organised pattern. The group is reminded that a good interpretation of remote sensing data is facilitated by a good knowledge of human use of land over the year.

At a third stop in Tempel the group admired a sand boil. A 15 m wide crater has formed in a channel. The whole group stood at its edge, watching gas bubbles emerging at the water surface…

At the fourth stop in Reeuwijk-Dorp the group faced the south extremity of the Tempel dike observed at stop 2. The relief of the dike is irregular. At its toe, the channel is wider. Cracks run parallel to the edge of the fields along the channel. It is here that the buried saucer was found and remote sensing data was chaotic. To restrain the group to have a closer look, Robert mentioned about cows that ventured too close to the distressed zone and disappeared.

The trip ended at 'Friends', the local bar-restaurant. Kindly, Ingeokring offered a drink to all participants while they exchanged the latest news. Siefko Slob was proud to announce his PhD defence on another remote sensing technique that has been quickly adopted by practitioners: the terrestrial laser scanning of rock masses to extract, without climbing, information on rock discontinuities.

Reference

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Optimisation of site investigations for offshore wind farm developments

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Gulin Yetginer *(Geotechnical Manager, RPS Energy)*

Introduction

The majority of offshore wind farms that have been developed to date are constructed in shallow waters with a relatively simple geology. The wind farms currently under development are getting larger and are subjected to a more complex geology. Just by increasing the wind farm size, there is a potential that the area will cover more diverse geological conditions. At the same time they are constructed in progressively deeper water. To add to the complexity of the development, there is a need to develop these wind farms in a shorter time span. These changing boundary conditions require a more advanced approach in subsurface investigations to meet the objectives and to save cost at the same time. The approach described in this article is based on wind farm developments by Centrica Renewables Energy Ltd. in the Wash area, offshore UK (Figure 1).

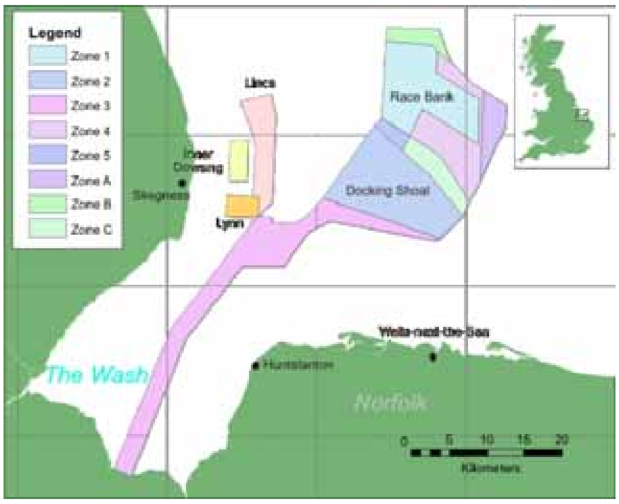

Fig. 1 Wash area developments

To give an understanding of the wind farm sizes it is best to compare Centrica's different rounds of development in the UK. Round 1 consisted of the Lynn and Inner Dowsing fields, each with 27 turbines. Construction of these sites was completed in 2008. The construction of the first of Centrica's Round 2 developments, Lincs wind farm, will commence this year. Lincs will consist of 75 turbines. Depending on the capacity of the chosen turbine, Centrica's proposed Docking Shoal site could consist of up to 177 turbines and its proposed Race Bank project could consist of up to 200 turbines. Round 3 sites have just been released for development. The total Round 3 area covers approximately 10 times the area of Docking Shoal and Race Bank combined (divided into smaller areas).

Common approach

Until now, a generally adopted approach was to start with a desk study, the results of which are used to set up a geophysical investigation. After all data has been gathered and processed, a few sections will be produced to assess the site conditions. Once the turbine layout in the field is known, the boreholes and CPT's are targeted at the turbine locations and the results of the geotechnical investigation are checked against the geophysical cross sections. At some locations, laboratory test results are combined with nearby boreholes. As there is no full coverage of boreholes for each turbine location due to the size of the field, the distances between boreholes can be such that correlation of lab test results is limited.

This approach works fine for smaller fields where borehole coverage of turbine locations can be high without the need to expand the geotechnical investigation over multiple (summer) seasons. Apart from scale issues, the geology needs to be relatively straightforward, where a few sections are indeed sufficient to understand the total area under development. But as the sites are moving towards more complex geological areas, a different approach is required. Furthermore, one can only start the geotechnical investigation once the turbine layout has been defined. Unfortunately, this layout is often not available at an early stage in the development. At the same time, an early assessment of potential foundation types does give a better understanding of the total cost of the development. But to be able to make a proper cost assessment, geotechnical information has to be available.

For the Lynn, Inner Dowsing and Lincs field developments, the above described common approach proved to be adequate. But the size and geological complexity of the Docking Shoal and Race Bank developments required a different approach, the so-called integrated approach.

Integrated approach

With the integrated approach, the investigation phases are basically the same. However, more time is spent in the office to get the most out of the data sets to allow for an optimisation of the next step in the investigation.

Desk study

Desk studies should not be considered as just an inventory and summary of all information available. From the start, the aim should be to create a soil province map. This soil province map can then act as a guideline to set up the geophysical site investigation and at the same time allow for early thoughts on potential foundation types. As the development will include many foundations, the cost implication is considerably higher compared to oil and gas developments. The sooner it is known what type of foundations are to be considered the sooner the cost implication can be entered into the financial model of the wind farm development.

A thorough understanding of the local geology is crucial for the success of the desk study and any further step. This is best achieved by having an (engineering) geologist specialised in the area.

Geophysical investigation

Once the desk study has been completed, the correct equipment can be selected for the geophysical investigation. In case any geotechnical information was already available in the area, it needs to be assured that at least one seismic line is run directly across this information to allow for direct correlation between the data sets.

No automatic processing should be used for the end product. Instead, manually picking each reflector is crucial. The main reason for this is that the selection of the reflectors should be based on foundation properties, not just geological formations. At the same time, based on the soil province map, a basic understanding of the area can be used to make sense out of the various reflectors and improve the quality of the processing. The aim of the processing is not just to present some sections, but to set up a 3D geological model of the subsurface in GIS.

3D geological model

The 3D model should be used to thoroughly understand the geology in the area and appreciate the depositional environment of all formations. This knowledge can be used to set up a stratigraphic column or allow for checking of the stratigraphic column set-up during the desk study. Once the model has been set up it can be used as a tool to design the geotechnical campaign, focussing on the positioning of the boreholes based on the required information of each formation instead of on the turbine foundation locations. This allows the start of the geotechnical campaign early in the wind farm development before the turbine layout is known.

The locations of the boreholes should be positioned to obtain sufficient information of all critical formations. This means that the amount of boreholes needs to be increased in more complex areas, but can be reduced in geologically homogeneous areas. Wherever possible, the boreholes should be positioned on the seismic lines to allow for a direct correlation.

Geotechnical investigation

When conducting the geotechnical investigation, it is crucial to prioritise the boreholes in such a way that the first set is spread throughout the field to allow for an early check of the 3D model. Once it is shown that the general model is correct, one can start to focus more on the details and the critical layers. Detailed logging can further improve the understanding of the geology in the area. For example, specific minerals, fining upward sequences and fissures in more competent material can all give clues on the depositional environment.

At any stage, when different material than expected is encountered, one should go back to the geophysical data to check the need for reinterpretation of the data. When the interpretation and thus the model need to be amended, the borehole layout might have to be changed at the same time (Figure 2). This procedure allows for a more flexible approach compared to the common approach where the results of the first step are merely used for the second step and no amendments to previous processing are considered.

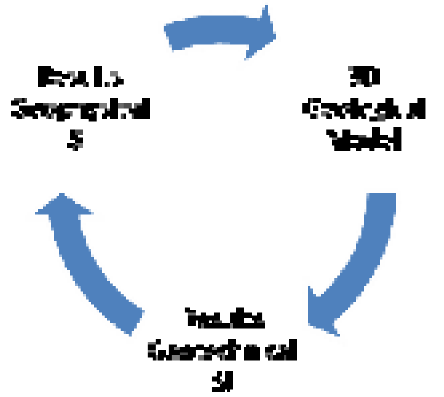

Fig. 2 Schematic representation of the integrated approach

Advantages integrated approach

There are several major advantages to the integrated approach:

- It allows for a more thorough understanding of the geology of the site instead of only knowing the soil conditions at a certain number of turbine locations.
- It ensures that sufficient information is available of all critical layers instead of having some information available in case you have encountered the material at the turbine location.
- It allows for geotechnical information being available early in the project development, which makes it possible to select suitable foundations and assess cost implications.
- Turbine layout can be adjusted to avoid expensive foundations.
- Data integration highlights errors and omissions of individual data sets and adds to the overall confidence.
- It gives good value for money. The total cost of the data integration is approximately similar to the execution of one borehole at the site.

This integrated approach has been adopted for Docking Shoal and Race Bank and has created a better understanding of the sites and as a result has reduced the number of required boreholes. Figures 3 and 4 present the main boundaries in the 3D model for Docking Shoal and Race Bank: the seabed and the top of the underlying chalk formation.

From the 3D model it becomes apparent that considerable channel features that have a potential effect on turbine layout are present in the top of the chalk.

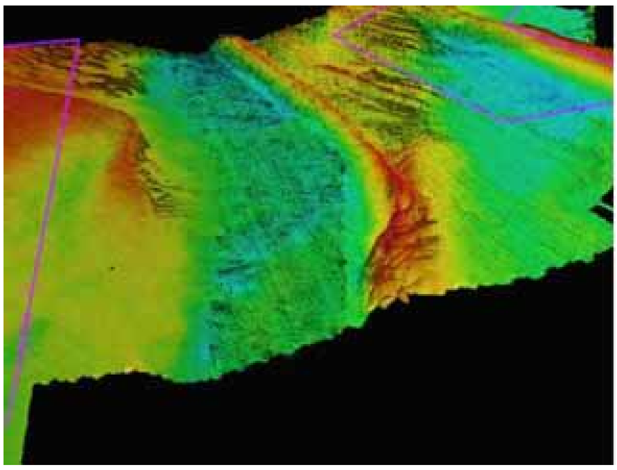

Fig. 3 Bathymetry Docking Shoal and Race Bank

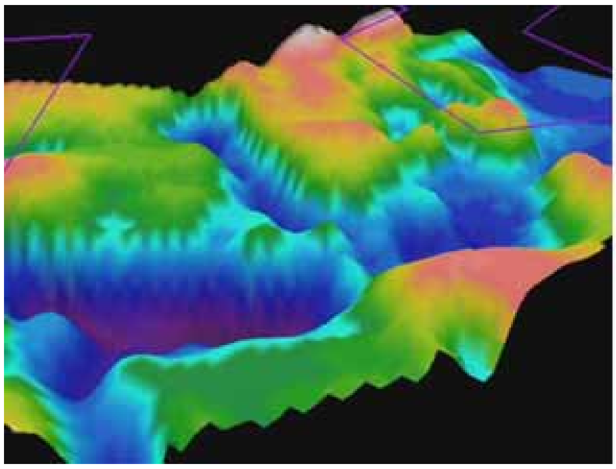

Fig. 4 Top of chalk Docking Shoal and Race Bank

Recommendations

Even though the integrated approach is almost standard practice for geohazard investigations in the oil and gas industry, it is still relatively new in the renewables industry. This means that even though the approach gives a much better understanding of the site, there is a good chance that some participants will not feel confident about not having the boreholes at the turbine locations. It is therefore necessary to discuss the approach with all parties involved. It is even more important to ensure that the certifying authorities are aware of your approach and that they feel confident that it is giving the required results. It is eminent that the certifying authorities are involved in the process from the beginning onwards and that they agree on each next step (i.e. borehole locations, changes in the model, changes in borehole layout, etc.).

Any interpolation of data often requires an extra factor of safety for the foundation design. In other words: it will require more steel for the foundation. It is therefore strongly recommended, despite the integrated approach, to conduct seabed CPT's at the turbine locations once the layout is known. Seabed CPT's are considerably quicker than boreholes and a complete field can be tested in just a few weeks. This also provides useful information regarding the spud can penetration assessment of the installation jack-up. The success of seabed CPT's does depend on soil conditions as very consistent soils might prevent sufficient penetration.

Professor's Column: why bother with statistics?

Prof.dr. Michael A. Hicks *(Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Soil is a variable material!

It exhibits inherent spatial variability of material properties and this influences material behaviour, contained fluid response and global geo-structural performance. Spatial variability may take the form of geological layering, or heterogeneity within layers or so-called uniform deposits. But, whether the soil deposit is naturally occurring or the result of engineered construction, spatial variability means that we are never quite sure what we have in the ground and this leads to uncertainty in the design and assessment of new and existing structures.

So, how should we deal with such a problematic material?

The conventional approach is to identify distinct material layers and then assign single representative values, or *characteristic* values, to the soil properties in each layer. When these values are used in engineering analysis this leads to a single factor of safety. But, of course, factor of safety tells us nothing about the probability of failure of our structure. Is this reasonable when we are dealing with such a variable material?

Eurocode 7 advocates the use of a *cautious estimate* when assigning characteristic property values, but gives little guidance as to how this should be achieved. But it does provide a list of things to be considered when choosing characteristic values and from this list a number of issues may be inferred:

- There is uncertainty about property values, partly because of soil variability, and partly because of limitations in field and laboratory testing.
- The spatial nature of soil variability is important.
- Characteristic values are problem-dependent.

Eurocode 7 also raises the possibility of using statistics for determining reliability-based characteristic values, but it is not a requirement. As such, geotechnical engineers are likely to interpret the *cautious estimate* approach as merely carrying on with what has always been done.

But are we missing out on opportunities by avoiding the use of statistics? My view is: yes!

The determination of characteristic property values is just one illustration of the possible uses of statistics in geotechnical practice: in this instance, statistical methods may be used to supplement engineering judgement, to help guide the decision process and to help make sure that our *cautious estimate* really is safe. But there are also other applications in which the statistical characterisation of soils can have a significant impact. In all cases, however, the starting point is to identify layering and to define the material properties for each layer in terms of their statistics, rather than as single deterministic property values.

So, how do we set about characterising soils statistically?

The cone penetration test for example enables a convenient solution. It provides a means for identifying the transition between layers, and within layers it provides a continuous record of data from which statistical information can be derived. The idea is that each material property is characterised by a probability distribution and by a mean and standard deviation. But more importantly, it should also be characterised by length scales defining the spatial correlation of property values in different directions. It is this feature of soil variability that is particularly challenging and it is a feature that is often overlooked in statistical analysis of soils. Put simply, soils generally exhibit a small correlation length in the vertical direction and a much larger correlation length in the horizontal plane, and each of these correlation lengths needs to be determined when characterising soils.

Once again, the concept of characteristic property values can be used as an illustration. If a reliability-based characteristic value is derived from only the probability distribution and point statistics of a property, the derived value will tend to be, in most cases, over-conservative. This is because no account is taken of the spatial averaging of property values over potential failure surfaces, which reduces the range of *effective* property values. Conversely, if the characteristic value is derived from a probability distribution that has been modified to take account of reduced variation due to local averaging, an unconservative parameter value is likely. This is because no account is taken of the fact that deformations and failure are attracted to weaker zones.

So, reliability-based characteristic values should be based on a modified *effective* property distribution that differs from the actual property distribution in two ways: it should be narrower to account for variance reduction due to averaging of properties; and it should be shifted relative to the actual property mean to account for weaker zones having a greater influence on geostructural performance than stronger zones. The final distribution is a function of the actual property distribution, the spatial characteristics of the variability and the problem being analysed, and it may be derived using stochastic analysis. So, the final result is a reliability-based characteristic value that is problem-dependent.

But, as already mentioned, characteristic values are just one illustration of the relevance of statistics in geotechnical engineering. In broad terms, the advantage of characterising soils statistically is that stochastic methods can then be used to quantify the uncertainty that arises due to spatial variability, due to there being incomplete information about the deposit as a whole.

A question that is often asked is, "don't you need lots of data to carry out a stochastic analysis?". Although it is easy to say "yes", the real answer is not as simple as that. We certainly need good quality test data: but were we to have so much data about a site that the soil profile was known everywhere, there would be no need to carry out a stochastic analysis! Stochastic analysis is needed because we don't have all the data, although, yes, a reasonably extensive data set is obviously desirable if we are to reduce uncertainties to an acceptable level.

So, here are some applications to consider:

- Cone penetration testing will give soil information at discrete locations across a site. By using the statistics derived from CPT data, stochastic analysis can be used to make predictions of the spatial variability over the entire site and to quantify the uncertainty associated with these predictions. The degree of uncertainty may be reduced if the predictions of spatial variability are conditioned to the known soil profiles at the CPT locations. In this case, the results can also be used to highlight most likely problematic areas, or to indicate optimal locations for further testing should a lower degree of uncertainty be required.
- Uncertainty in ground conditions causes uncertainty in the performance of any structure founded on or within the ground. Stochastic analysis can be linked with numerical methods such as finite elements to quantify geostructural performance within a probabilistic framework. Uncertainties may be reduced by conditioning analyses using observations obtained during and after construction.

Of course, stochastic analysis is not limited to quantifying uncertainty associated with soil spatial variability. It can also be used for quantifying uncertainties due to possible model errors, measurement errors, transformation errors, and so on.

The use of statistics in geo-engineering gives us new opportunities. Give them a try!

Site characterisation with the video cone

S. Paul, V. Hopman & T. Peters *(Deltares, unit Geo-engineering, Delft, The Netherlands, simon.paul@deltares.nl)*

Abstract

Geotechnical and geological soil characteristics can be measured by different in situ measuring techniques. For example with standard Cone Penetration Testing (CPT) different layers in the subsoil and bearing capacity can be determined. However, a detailed view of the different layers, grain size distribution, grain shapes or contaminants cannot be established by in situ measurements. To determine these properties, soil or groundwater samples need to be brought to the surface for analysis. To determine these properties in situ, one would like to see into the subsoil. This prompted Deltares to develop a camera probe (video cone). In the past decade, the Deltares video cone has been successfully applied at many site characterisation projects. In this paper, a number of these cases will be described.

Introduction

Site characterisation is an important step in site remediation and construction projects. The collected data is used to describe subsoil conditions or to predict subsoil behaviour. There are two ways of gathering data about subsoil: sampling and in situ measurement Remote sensing and geophysical techniques are used more and more but sometimes miss vertical resolution, have a low horizontal resolution (big areas) or need calibration by sampling and sounding techniques. Remote sensing and geophysical techniques have area coverage, while sounding and sampling are point measurements. The approaches are complementary.

In situ measurements with help of *push-away* equipment (e.g. CPT equipment) are very popular in The Netherlands for geotechnical purposes and have a long tradition due to the soft soil conditions in the western part of The Netherlands.

In situ measurements have the following advantages (Van den Boogaart et al., 2002):

- There is generally less disturbance than during drilling and sampling.
- The method is generally fast and consequently relatively cheap.

Sampling has the advantage that the sample can be investigated and tested. Laboratory testing and analyses of these samples can determine properties like detailed view of the different (thin) layers, grain size distribution, grain shapes and triaxial and direct shears.

For environmental studies, in situ measurements are generally restricted to a small number of parameters measured at a restricted number of depths. Here, direct push tools hold the promise to increase vertical spatial resolution and the number of parameters that can be detected quickly and at relatively low cost (Hopman et al., 2009). Optical sensing and detection methods have shown to be powerful tools in combination with push-in equipment. In the visual wavelength range, many contaminants are visible or can be made visible in the subsoil.

In the late '90s of the 20th century, Deltares developed a video cone in partnership with the Dutch Ministry of Public Works. The probe is designed to collect more information from the subsoil compared with other techniques. For geotechnical purposes, this information can consist of (thin) layers (stratigraphy) and subsoil texture (e.g. grain size distribution), which will normally be obtained by sampling techniques. Porosity can also be derived from this information. For environmental purposes the useful information can consist of (vertical) spatial distribution of contaminants.

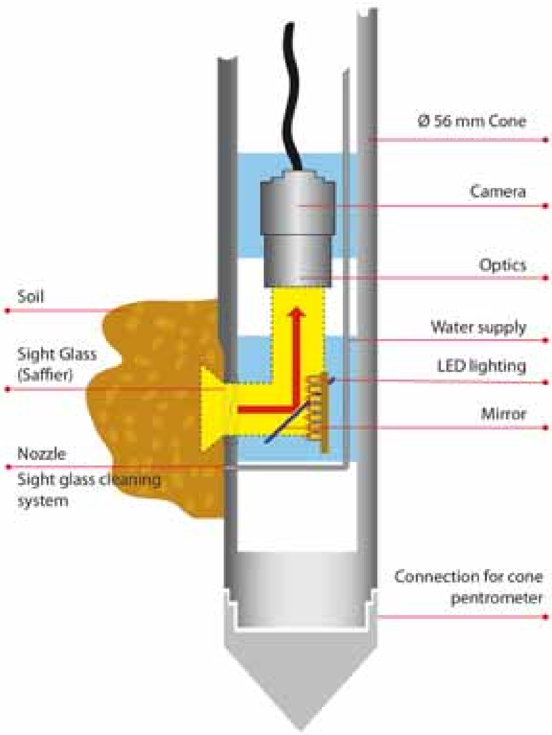

Fig. 1 Deltares video cone

Video cone technology

The Deltares video cone is an innovative soil investigation technique that uses standard penetration material to record images of the subsurface. It is able to reach depths that are also possible using other soil penetration techniques. The probe consists of two parts: a normal cone penetrometer (nose of the probe) and a wider stainless steel tube housing the camera. The diameter of the camera part is 56 mm and the total length is about 1.2 m. A schematic drawing of the probe is shown in Figure 1.

A sapphire sight glass is fitted in the stainless steel housing. The dimensions of the sight glass are 14 x 20 mm. A mirror, LED lights and a digital camera are mounted in the housing behind the sight glass. The camera is connected to a laptop via a special data transmission cable.

By using extension casing pipes and a standard hydraulic push-in unit, the camera probe can be pushed into the subsurface at a preferred speed of 0.2 cm per second. This penetration speed is 1/10 of the standard CPT speed. At the moment, increasing speed will negatively influence the quality of the images taken. Deltares is working at techniques to increase the speed and still get good images. During penetration the light shines through the sight glass on the adjacent soil, from which the camera takes images every 10 mm of descent. At the same time, real time images can be viewed on the laptop. The camera magnifies the images which makes it possible to distinguish grain sizes as small as 30 µm (design range: 63 µm to 2 mm). The resolution of the camera is 1,200 x 1,600 pixels (2 megapixels).

The camera probe is fitted with a cone penetrometer with a diameter of 36 mm. During push-in of a cone penetrometer into the subsurface the force at the tip of the cone, the friction on the sleeves and water tension are measured. The data collected from the cone penetrometer can be used for cross correlation with the recorded images.

Geotechnical applications

In geotechnical applications the video cone has been successfully utilised for mapping of gas loading of sludge depots (case 1), settling properties and contents of landfills (case 2), and charting layers of sand at a dike at Vianen (case 3). These cases are described below.

Case 1: gas loading of sludge depots

Dredged sludge is often strongly polluted and therefore needs to be stored in special depots. Gas production by bacteria in the sludge reduces the storage capacity of sludge disposal sites (Greeuw & Van Ree, 2003). The video cone has been applied, in combination with other techniques, to map the gas loading of the Slufter depot at the Maasvlakte and the IJsseloog depot at the Ketelmeer, both in The Netherlands. The images give a good impression of gas loading in the verticals. Figure 2 shows an image taken at one of the video cone soundings at the IJsseloog depot.

Fig. 2 Gas bubbles in consolidating sludge at the IJsseloog depot

Case 2: geotechnical and environmental site characterisation of landfills

The video cone was used for site characterisation at different landfills in Dordrecht, Maastricht and Toronto (Canada). The objectives were different. In Dordrecht, a general insight of the contents of the landfill was required. In Maastricht, the local government wanted to redevelop the former landfill Steilrand. The goal of the investigation was to determine the geotechnical properties of the landfill, resulting in a geotechnical advice on settlement and foundation possibilities for future construction on the landfill. The video cone was used to map different layers, void spaces, heterogeneous patterns, perched water bodies, the groundwater table and closed-in gas bubbles. In total 14 video cone tests (including CPT's) and 37 separate CPT's were performed. In addition, 10 validation trenches were dug. The combination of techniques gave a good insight in packing and possible settlement of the landfill, from which the geotechnical advice could be derived. The use of the video cone limited the amount of CPT's and field research required for characterisation. The main objective of the Toronto project was the mapping of organic-rich zones in a closed conventional landfill. Identification of the organic-rich zones is essential for the location and placement of aeration wells that are required to convert a conventional landfill into an aerobic bioreactor landfill (ABL). Forced aeration of organic-rich zones will enable accelerated decomposition of waste,

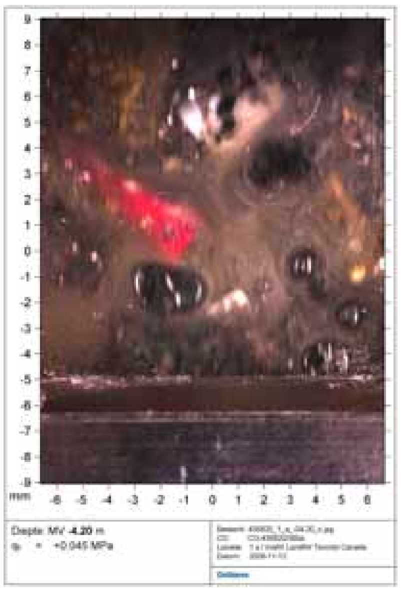

Fig. 3 Image of Inisfill landfill in Toronto, Canada (depth is 4.20 m)

thereby eliminating the generation of methane which is an explosive hazard. Therefore, to implement an ABL system at any closed landfill site it is important to know how much organic matter, perched water and gas are present. Another interest was the porosity and permeability of the fill material in the landfill. Methane gas can migrate through void spaces to the edges of the landfill, which is an unwanted situation. Void spaces, different layers of Municipal Solid Waste (MSW) and sand, perched water and gas bubbles could be detected, estimating porosity and permeability proves to be more difficult. Figure 3 shows an image taken at 4.2 m below surface level. The image shows an unidentified object (red), perched water, gas bubbles in the perched water, and heterogeneous MSW with some sand (grey matter).

Case 3: mapping water bearing sand layers under dikes (sensitivity to piping)

Piping is a phenomenon which can threaten the stability of flood defences. This phenomenon can arise when, in case of a large scale hydraulic head, soil particles in layers that are susceptible to erosion are transported underneath the flood defence by seepage flow, as a consequence of which erosion channels are created (Technical Advisory Committee on Flood Defences, 1999). Currently, a significant part of the Dutch flood defences does not meet the required safety levels concerning piping. Sensitivity to piping is determined by mapping the water bearing layers (e.g. grain size distribution and thin silty layers) under the flood defences and the top layers and putting this data in a computer model (e.g. Sellmeijer). At rivers, the subsoil is very heterogeneous which requires a lot of field research in the form of drilling and sampling. The objective of this research was to demonstrate that the video cone could be faster and more cost effective in mapping the water bearing sand layers under the flood defence. The test conducted at Vianen (The Netherlands) showed:

- The probe causes minor disturbances of thin layers, which slightly effects the observation.
- In case of large seepage flows in the water bearing sand layers with a big load of silt the video cone has difficulties getting a clear picture (muddy images).
- When the images are stitched together, a good overview of the layers was obtained.
- When individual images of sand layers are analysed, the grain size distribution and grain shapes can easily be distinguished.

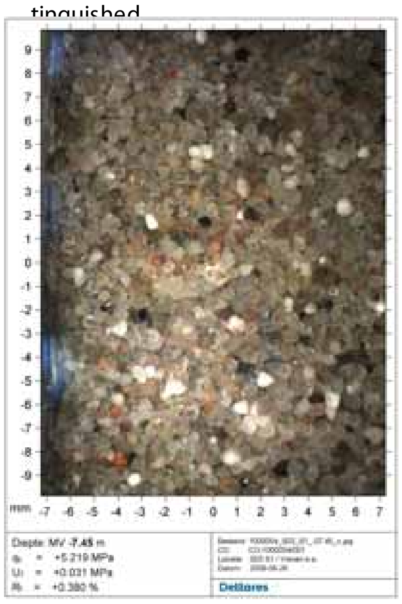

Fig. 4a Image of sandy layer in Vianen (depth is 7.45 m)

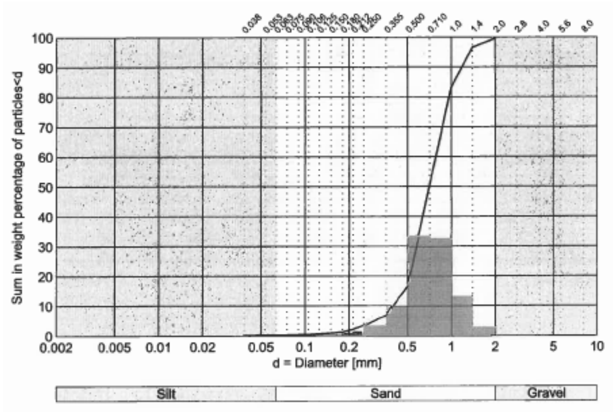

Fig. 4b Grain size distribution of the material in Figure 4a

Apart from the video cone soundings, Begemann continuous sampling and a normal CPT were carried out. These measurements were used to verify the results of the video cone soundings. Figure 4a and 4b show an image and grain size distribution of material from the site in Vianen. Other geotechnical applications such as mapping of gravel layers of horizontally directive drilling paths (prior to drilling) and classification of sand layers for the sand mining industry are currently investigated.

Environmental applications

For environmental applications the video cone has been successfully utilised for the mapping of landfills (see case 2). Many more projects were performed mapping contaminated sites (with mineral oils, LNAPL's or DNAPL's). The mapping of a creosote oil contaminated site (case 4) and the mapping of a metallic mercury contaminated site (case 5) are described below.

Fig. 5 Creosote oil in the subsoil as seen by the video cone (image size is 5 x 7 mm)

Case 4: creosote oil contaminated site

At a former timber processing plant in Rotterdam, creosote oil had leaked into a sandy aquifer and due to its high density had accumulated above impermeable silty and clayey layers at about 12 m below surface. The video cone was applied as the monitoring of wells had proven to be very ineffective in detecting the extent of this type of contamination (Van den Boogaart et al., 2002).

The creosote oil could easily be detected. From about 40 cm above the impermeable layer, small amounts of 'black liquid' appeared in threads and droplets. Going deeper, the voids were more and more filled with creosote oil, until the deepest part where the voids were completely filled. Figure 5 shows creosote oil in the subsoil as seen by the video cone.

Fig. 6 Metallic mercury in the subsoil as seen by the video cone

Case 5: metallic mercury contaminated site

Metallic mercury had leaked into the subsoil at a former processing plant and due to its high density it had accumulated above impermeable silty and clayey layers at about 9 m below surface. The video cone was applied as other methods could not detect the metallic mercury in situ. The metallic mercury could easily be detected. Very small droplets were identified, accumulated at 10 different layers in the subsoil. Figure 6 shows metallic mercury in the subsoil as seen by the video cone. Many other contaminated sites were characterised. For the video cone the contrast between contaminant and subsoil is important. Deltares is currently investigating in situ colouring of colourless contaminants, like chlorinated organic compounds (DNAPL's).

Concluding remarks

The video cone is an effective tool for field screening for geotechnical applications (stratigraphy and subsoil texture, e.g. grain size distribution) and environmental applications (vertical spatial distribution of contaminants). For geotechnical applications direct push tools are generally causing fewer disturbances than drilling and sampling and the method is fast and consequently relatively cheap, which is also the case for the video cone. In addition, the video cone gives real-time images of the subsoil which makes analysis on location or in the office possible. This provides additional information compared to the CPT.

Furthermore, it could be emphasised that for environmental applications, direct push tools hold the promise to increase vertical spatial resolution and increase the number of parameters that can be detected quickly and at relatively low cost. This can be achieved by using a combination of in situ measurements like the video cone and push-away equipment. Samples do not need to be taken to the surface for examination which is a big advantage for contaminated sites.

Further development of the video cone could, in combination with data obtained from the attached cone penetrometer, give the ability to (semi)-automatically classify subsoils, analyse texture and give an estimation of porosity of the subsoil.

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De Ondergrondse: board change and St. Petersburg study trip

From the board

(by Jan-Willem Vink, secretary 2009-2010 board)

During the general members meeting on October 22, 2009 the new board of De Ondergrondse, consisting of Thijs de Blaeij, Hugo Engelen, Jochem van der Meulen and Jan-Willem Vink, was officially installed. Also, the current status of De Ondergrondse and a review of the previous year were presented.

The old board, which consisted of Paul Spruit, Werner van Hemert, Stijn Biemans and Etienne Alderlieste, performed outstanding last year. They organised many interesting excursions, such as the excursion to Belgium. Many activities were well visited and were experienced as very satisfying by their participants. I would like to thank the old board for the good work in the past year, they left the new board with a tremendous situation to start from!

Fig. 1 New board of De Ondergrondse: Jan-Willem Vink (secretary), Hugo Engelen (treasurer), Jochem van der Meulen (chairman), Thijs de Blaeij (commissioner)

During the meeting, the new board presented the goals they have set for the coming year. Main goal this year is to attract more Bachelor students for the MSc Geo-Engineering and the MSc Engineering Geology. There are signals that a lot of Bachelors tend to choose for the Master's, but we have to find out why they eventually decide not to go for it. There seems to be a need for more promotion of the Master's degree. We have also planned to organise the main activities of De Ondergrondse: four excursions, four lunch lectures, four newsletters and of course the study trip.

Since the start, a few months have passed and we can already see the results of our efforts as the organising committee for the study trip consists partly of Bachelor students. Also, the bi-weekly drink in cooperation with the geo-section seems to pay off. These drinks give students and teachers the opportunity to interact in a more informal way with each other. These sessions produce good creative ideas to solve the problem of attracting more Bachelors.

St. Petersburg study trip 2009

In July 2009, De Ondergrondse organised a study trip to St. Petersburg, Russia, which was one of the highlights of the year. Preparations started late November 2008. A group initially consisting of seven enthusiastic Geo-Engineering students volunteered in making the study trip into a great success. In finding an appropriate destination we mainly looked at the number of interesting geotechnical projects in progress and possible cultural activities. Several destinations were found, of which St. Petersburg looked the most appealing. After the location was chosen, the real work started in establishing contacts with contractors, universities and companies. Contact was made with employees of the St. Petersburg Mining Institute, who were very willing to help in arranging the trip. The hospitality has been amazing and we were no less lucky with the weather, projects and impressive cultural activities that were undertaken.

Arrival *(by David Laughton)*

Only a few hours after our arrival in St. Petersburg, a Russian breakfast was waiting for us with the necessary Russian fats and proteins. Since we had arrived late in the night or rather early in the morning, we had agreed to depart to the St. Petersburg Mining Institute at a not too early hour.

They provided us with a charming guide, a PhD student from the institute named Anni who would accompany us and was our translator in time of need. That late morning we were welcomed very officially at the faculty in a magnificent conference room by four representatives of the institute: Mr.

Fig. 2 In front of the Mining Institute

Pashkevich, the Vice-Rector; Mr. Tulin, Head of the International Department; Ms. Shukina; and our official translator Kyril, an English teacher at the faculty. From the speeches it became clear that the institute has a very good reputation in Russia and the rest of the world in the field of mining science.

The institute was founded by Catherine II in 1773 as Russia's first technical college. The institute consists of several departments: Mining, Geological Prospecting, Metallurgy, Mining Electro-mechanics, Underground Space Development, Economics, and Correspondent Studies. About 7,500 students are divided across these departments; the students are mainly Russians as all teaching is in Russian. After a strict selection only the best students from all corners of Russia are welcome to commence with their studies at this prestigious institute. It was a bit disappointing that there were hardly any students to talk to during lunch because of the holiday season.

The tour that followed was in line with the preceding laudatory words. The immense building with its decorated halls, its ornate ceilings and impressive statues looked more like the old elite cadet school that it once used to be than an ordinary functional university building. One could think that the faculty is a museum, but then we had not yet seen their own Mining Museum. This museum contains a stunning 230,000-item collection of minerals, paleontological exhibits, geological and mining models, precious and faux gem stones and ores, displayed in twenty halls. Items such as a 500 kg quartz crystal and a 850 kg copper nugget are just two examples of the astonishing collection. When we asked Kyril how unique this museum is in Europe, he replied proudly that it's unique in the world.

After leaving the faculty we were offered a city tour by bus. Kyril told us everything about the buildings and their history. From time to time we got out of the bus to visit the beautiful monuments by foot. Kyril turned out to be a real connoisseur of history and he knew the answers to almost all our questions. One of the most remarkable phenomena is the reconstruction of the city after WW II. Looking around, it was hard to believe that the city was destroyed for nearly 70 percent after it had been besieged by the Nazis for 550 days. The city has nowadays been nearly completely restored to its original state. After our tour it was time for dinner in the auditorium of our hostel apartments where we could reflect on the impressions of our visits that day. Because of the topographic features of St. Petersburg the days were long and therefore we could once more admire the beautiful centre in daylight. A sunset around midnight was a perfect ending of the day.

Fig. 3 Part of the large collection of minerals and fossils

St. Petersburg Flood Protection Barrier *(by Tim Drummen)*

On the second day of the trip the (geo-)technical highlight of the study trip was visited: the Flood Protection Barrier. The barrier is situated some 30 km west of St. Petersburg and connects Kotlin Island with the northern and southern bank of the Gulf of Finland. It has a length of 25.4 km and consists of a breakwater construction separated by six floodgates and two navigation channels, one for small, recreational boats (C2) and one for large container ships (C1). On top of the breakwater a dual carriage way with three lanes will be situated, which eventually will be part of the St. Petersburg ring road.

Ever since Peter the First founded St. Petersburg in 1703, the city was threatened by more than 300 floods, with the most severe one in 1824 when the highest water table was measured at 4.1 m above average sea level. The floods are caused by low pressure air that moves in from the west and creates

Fig. 4 Overview of the city and the dam that is constructed in the Gulf of Finland (source : NASA World Wind 1.4)

so-called 'long waves' that bring a lot of extra water into the Gulf of Finland. This large amount of water, the shallow depths at the Neva river mouth and strong western winds block the water of the river Neva causing the water to rise to high levels. Design of the current flood protection barrier already started in the seventies but due to environmental issues and a lack of funding, the construction works were stopped in 1990. At that moment the barrier was completed for 60 %. In 2003 the European Bank for Reconstruction and Development decided to fund the completion of the project. Construction works were restarted and the project should be completed in 2010. We entered the barrier from the north side and had the first stop at the 'small' navigation opening (C2). This navigation opening, which is only open during iceless periods, has a width of 110 m and a depth of 7 m. The opening is crossed by a bridge that can be lifted up to 10 m by hydraulic pumps. In case of a flood risk a concrete door, which is 'hidden' at the bottom, is lifted to close the navigation opening. The construction of the actual barrier and the traffic bridge was done in a dry environment using a cofferdam. The concrete floor (slab) of the construction is founded on 500 mm diameter bored piles.

Fig. 5 The northern gate for smaller ships

The next stop was at the Boskalis/Hochtief construction site. Boskalis and Hochtief are contracted to finish the 2.6 km gap in the flood protection barrier that still remains. This construction site is located on the south side of navigation opening C1. This opening has a width of 200 m and a depth of 16 m and is operational during the whole year. In case of a flood the opening can be closed in 30 minutes by two floating curved doors. This is the same principal as the Maeslantkering near Rotterdam. Unlike the Maeslantkering the gates are closed and opened by tractors. Like navigation opening C2 this construction was also created in a dry environment using a cofferdam. First, a traffic tunnel was constructed in this cofferdam, 17 m below the navigation opening. This was already done in the 1980's. Boskalis was contracted to dredge part of the waterways to reclaim the land for the last part of the barrier and to construct the armour layer of the breakwater. Hochtief, a German contractor, was contracted to finish the last part of the tunnel construction and the ramp that leads the traffic in and out of the tunnel.

Fig. 6 Excavation of the ramp

Boskalis first dredged the marine clay out of the Gulf of Finland to 18 m below sea level. After this, land was reclaimed up to 3 m above sea level. The sand used for the reclamation was dredged from a Boskalis sand borrow area at 30 minutes sailing from the construction site. Because of the good properties of the sand, no compaction measures were required. Granite armour stones were obtained from a mine 200 km from the construction site. Boskalis is now excavating the last part of the ramp (near the tunnel) in the embankment that they first reclaimed. A sheet pile wall separates the tunnel entrance, with the foundation and the construction of floating doors on top of it, from the bottom of the ramp. The stability of this sheet pile was calculated and a safety factor of 1.1 was the outcome. Remedial measures were taken by placing large concrete blocks in front of the sheet piles. Also no traffic or other large loads are allowed on top of the sheet pile.

The ramp, constructed by Hochtief, has a length of 730 m, a thickness of 70 cm and is founded on Fundex piles with variable diameters of 650, 800 and 1,200 mm. The piles penetrate into an over-consolidated hard clay layer up to a depth of 15 to 18 m below sea level. According to the (Russian) design the concrete ramp should be watertight. To achieve this, the ramp is sealed with PVC foil.

Because the site investigation was performed a long time ago, new boreholes are drilled for verification of the previously acquired data. The boreholes run to 5 m in the clay and only clay samples are cored. After leaving the construction site (and gaining some valuable party information) the last part of the program was a visit to the office of the supervisor of the project. Here, a scale model of the navigation opening C1 was presented. After a good lunch in the office we headed back for an afternoon off in St. Petersburg.

Fig. 7 Inside the Mining Institute

Ingeokring mini-symposium 'Special Ground Investigation'

Jordy Mollé

On December 3, 2009 the yearly mini-symposium of the Ingeokring was held. The topic of the day, which was attended by about 40 professionals, was 'Special Ground Investigation - Innovation in Site Investigation'. The number of MSc students attending was significantly smaller than normal; apparently they had to hand in an assignment the next day and most of those that came to the symposium were only present at the welcoming coffee but looked definitely stressed! The mini-symposium itself was a success with well-presented lectures by several professionals from a wide range of companies and institutes, and an audience that was interested and participating with questions and discussions.

Peter Verhoef (Boskalis) held the first presentation of the day and he showed the usefulness of the (unfortunately) not so frequently used Equotip Hardness Tester for rock core logging (see also the article in this Newsletter about the Equotip). Then Paulien Kouwenberg presented some of the findings of her recently completed MSc thesis. She discussed not only the site investigation but also the laboratory investigation for the geotechnically challenging A2 tunnel in Maastricht.

Next, Leon van Paassen (TU Delft) showed some of the outcomes of his recently completed PhD thesis in a presentation titled '3D modelling of cemented zones in biogrouted sand'. He also provided some recent results of the still ongoing project. Arjan Venmans' (Deltares) presentation 'Towards more reliable subsurface models: data integration for design of motorways on soft soils' gave details of the Geo-Impulse program that intends to have ground-related failure costs in The Netherlands halved by 2015. Arjan has also provided an article that deals with the Geo-Impulse project for this edition of the Newsletter. The last speaker of the day, Diederick Bouwmeester (Fugro), showed some special or recently developed in situ push-in techniques.

At the end of the meeting, Peter Verhoef thanked a group of active Ingeokring members for their contribution to the Ingeokring over the past few years. Chris Bremmer, Marcel Remijn, Leon van Paassen, Michiel Maurenbrecher, Erik Schoute and Gerhard Wibbens: once again, thank you for your efforts, enthusiasm and good work. The drinks at the Geo-Engineering Laboratory concluded a very interesting and successful mini-symposium!

Thesis abstracts

3D subsurface modelling and geotechnical risk analysis for the tunnel construction of the A2 project, Maastricht

Paulien Kouwenberg

To reduce traffic problems and to improve the social climate in Maastricht a tunnel will be constructed for the A2 motorway. The tunnel will be constructed using the cut-and-cover method. Before construction starts, a ground improvement campaign will be performed. Afterwards sheet pile walls will be installed in cemented slurry trenches and the building pit will be excavated *in the dry* wherever possible. The subsurface of Maastricht consists of a top soil of sand and clay (3 m) and a thick gravel layer (10 m) on top of limestone. The water level in Maastricht is generally high (3 m below surface). The geotechnical and geological risks related to the tunnel construction have been studied based on geological knowledge and data of the preliminary site investigation campaign, in order to assess geohazards and to identify the most promising techniques for further investigation phases. The Formation of Maastricht consists of weak porous limestone with flint and hardground layers. The limestone can be affected by dissolution, weathering by groundwater flow, erosion due to the Meuse River or be faulted and reduced to carbonate sand. Problems with excavation, leakage of slurry, lack of bearing capacity and passive resistance and, last but not least, uplift of the building pit floor or high groundwater inflow are expected during construction. Various databases were exploited to map the fluctuations of the gravel-limestone interface. Characteristic fossil and flint horizons allowed the segmentation of the cores of the limestones into members. Vertical displacement of members between adjacent boreholes revealed the presence of a major fault underneath the Voltastraat. The A2 borehole cores did not only contain rocks but also carbonate sands. A correlation between needle penetrometer and UCS values was established in the laboratory. Needle penetrometer tests performed on cores at close spacing allowed an objective mapping of the spatial distribution of carbonate sands and very weak to weak limestone along the tunnel alignment. The presence of carbonate sands was tentatively related to the presence of faults. At the Voltastraat fault, analyses of groundwater chemistry confirmed the connection between deep and shallow limestone and gravel layers. Many geophysical techniques were performed in a pilot zone, above the Voltastraat fault. Reflection seismics conducted with a low energy airgun source was found to be the best performing technique. It revealed the presence of a complex fault zone rather than a single fault. The north-east orientation of correlated to fractures measured in boreholes with the help of a borehole camera. Estimating in situ permeability and deformability remained difficult. Primary permeability was found to vary over 3 orders of magnitude, up to the in situ permeability measured with the Lugeon test (10 mD to 10 D). In the fault zone a higher permeability is expected. For each member the statistical distribution of material properties was studied and the flint content per member was estimated based on the borehole logs. 3D models of the stratigraphy, lithofacies and some material properties were constructed to image the subsurface along the A2 tunnel. The models highlight weaker zones and can be integrated into groundwater flow models.

Discrete Element Modelling: the influence of high hydrostatic pressure on the cutting processes of hard rock

Nico Parasie

Seafloor Massive Sulfide (SMS) contains high levels of metals such as copper and gold. Water depths of up to 2,000-3,000 m make mining of SMS a challenge. Despite this, the mining industry encourages research into feasible extraction methods of SMS, pushed by the high metal prices nowadays. This research focuses on the influence of hydrostatic pressure on the cutting process of hard rock. A 2D numerical model of the cutting process is created using discrete element modelling (DEM). The software package Particle Flow Code 2D (PFC2D) from Itasca Consulting Group is used. The rock material creation in PFC2D consists of reproducing numerically the physical UCS, biaxial and Brazilian tests executed on the benchmark material. SMS is the rock of interest for deepsea mining, nevertheless Langmeil Sandstone is chosen as benchmark material. This consideration is taken because SMS samples are rare and no strength test results obtained on SMS samples are published. In addition, the high heterogeneity of SMS makes numerical modelling difficult. It was not possible to come up with a singular numerical rock sample matching the mechanical properties of the sandstone for a large range of confining pressures (0 to 40 MPa). Therefore two samples were created: the first sample is valid for unconfined tests, the second sample is valid for confining pressures between 20 and 40 MPa. It appeared that the material's unconfined compressive strength and elastic constants are independent of particle size. The transition point from brittle to ductile failure is simulated successfully for the second rock sample. Three different cutting scenarios are studied: cutting of dry rock without hydrostatic pressure, cutting of dry rock under hydrostatic pressure and cutting of satucapable of simulating hydrostatic pressure. This method applies a force similar to the hydrostatic load on each boundary particle of the sample. The method simulates the transition between brittle and ductile rock cutting successfully, by increasing the effective stress while fluid and pore pressures are absent. The transition from brittle to ductile cutting is reached at a hydrostatic load of 20 MPa for the Langmeil Sandstone. The cutting force (450 kN) obtained by the numerical model for the unconfined calibrated rock is compared with existing semi-empirical models. Goktan's (1995) model (300 kN) provides the best match while Evans' (1961) model (75 kN) underestimates the required cutting force. The horizontal cutting force increases with increasing hydrostatic pressure. The increase of hydrostatic load is mainly transferred into an extra load on the cutting tool in horizontal direction. The specific energy calculated from the cutting force obtained by the numerical model (4.9 MJ/m^3) is in good accordance with the values for sandstone found in literature (5.5 MJ/m^3). The influence of the tool shape, the cutting velocity and depth of cut on the cutting process of dry rock is modelled. The chisel pick tool shows to be more efficient than the pick point tool for shallow water depths. The difference in efficiency between these tools becomes smaller with increasing hydrostatic pressures. Measurement circles in PFC2D are able to register porosity changes during the cutting process. These measurements are used to estimate pressure differences in the crushed zone. This method provides a good qualitative insight in the development of pore pressures during the cutting process. Useful quantitative values were not obtained. The measurement circle method indicates that cavitation will not occur for high hydrostatic loads such as 20 or 30 MPa, which implies that the required cutting force will continuously increase for an increasing cutting velocity. The main recommendations are introducing the Biot poro-elastic equations and heterogeneity into a 3D discrete element model. Quantitative estimations for the specific energy and pore pressures during the cutting processes should be obtained. Execution of laboratory tests of rock cutting under varying hydrostatic pressures is of utmost importance to check the performance of numerical rock cutting models.

Calibrating the TU Delft double wall triaxial cell for investigation of the unsaturated behaviour of Maastricht carbonate sands

Tim Drummen

Unsaturated soil is a relatively new subject in soil mechanics. Most of the models and equations used are based on a *traditional* approach, based on saturated soil. Due to the presence of soil in an unsaturated state, both in nature and artificial applications, this approach is not justified in many situations. Also in the Dutch subsurface unsaturated soil is present. For investigation on this subject TU Delft purchased a double walled triaxial cell for testing unsaturated soil samples. This apparatus, based on the axis translation technique for the application of suction, is developed for accurate measurement of sample volume changes during various loading paths on unsaturated soil samples. A good calibration of the several parts of the double wall triaxial cell is of high importance because of the sensitivity of the volume change measurement. Additional investigations on observed phenomena during these calibrations and tests to get a good insight on how to use the double wall triaxial cell are performed. The first tests with the apparatus are performed on Maastricht carbonate sands that were tested before by TU Delft for the A2 tunnel project in Maastricht, The Netherlands. The earlier test program contained deviatoric loading tests, performed with a *regular* triaxial cell. These tests were done on carbonate sand samples in an unsaturated state, but with an apparatus developed for saturated soil testing. To investigate the behaviour of the unsaturated carbonate sand and for comparison with the earlier observed results the double wall triaxial cell is used. Due to the lack of experience and some unknown soil properties the observed results do not describe the behaviour of the carbonate sands. However, the mistakes that were made should be considered as a lesson for future investigation on unsaturated soils with the TU Delft double wall triaxial cell.

Final safety design of the Cumbidanovu dam in Sardinia: a combined approach of geotechnical and structural engineering

Maren Katterbach

The lack of adequate integration of geology and engineering into the design of engineering structures might have significant undesirable consequences. It appears that all too frequently the interface between the engineering structure and the natural environment, such as a dam foundation, presents a virtual interface between both disciplines of engineering and earth science respectively. This work demonstrates how to integrate both disciplines into the final design of the Cumbidanovu dam in Sardinia. Foundation stability is one of the most important factors influencing the safety of a concrete dam and has been one of the key technical problems in the design of the studied dam project. The central research question to be answered is if the latest final design of the Cumbidanovu dam complies with the standards for safety and if it is economically sound. To do so, the following four central hypotheses and theories have been checked during the thesis work:

1) A critical assessment of sliding safety should be based on a high quality database. Therefore, a further refinement of the already available data of the Cumbidanovu dam site is required. The data acquisition should involve both simple and sophisticated means.
2) The safety factors for the planned Cumbidanovu dam with its complex foundation conditions can be increased by improvements.
3) Cooling of the dam during and after construction is necessary in order to ensure dam stability, which might be affected by crack initiation and temperature discrepancies.
4) The global stability assessment of the Cumbidanovu dam with its encountered complex foundation rock mass requires the combination of structural and geotechnical considerations. Structural and geotechnical aspects should be properly integrated into the overall dam design in order to achieve dam safety with an optimised final design.

The desk study at the beginning provides an insight into the actual site conditions as well as their future development. Adapted to this outcome, a field survey is planned and conducted in order to determine the variations with time and work progress. Special attention is paid to the possibilities and limitations of geomechanical classification systems. It is shown how to obtain a sound geomechanical database and the corresponding data processing. Especially for the studied dam site, which is characterised by complex foundation characteristics, a careful geomechanical evaluation is indicated in order to estimate the influence of the rock mass on the dam structure. The acquired data are subsequently integrated into the structural analyses, which are recommended to evaluate the dam stability. By using different geomechanical and structural approaches, it is possible to cross-verify the results. The procedure of this work, which integrates both structural and geotechnical aspects into design considerations finally allows to come up with a suggestion for a safe final dam design, which in the end emphasises the

Hybrid Rowe cell for measurement of complex conductivity

Suguru Shirasagi

Peat is sediment consisting of incompletely decomposed organic matter deposited in swamps and marshland. It has served for a long time as beneficial material to human beings. Meanwhile, it has posed huge challenges to geotechnical engineers due to its unique characteristics such as anisotropy, low stiffness, high compressibility and strong creep susceptibility. In order to realise more effective and efficient site investigations, it is highly expected to apply geophysical techniques as well as core borings, CPT's, sampling and laboratory tests because the techniques promise to be great contributions not only for two- or three-dimensional mapping but also for accurate interpretation of its physical, chemical and engineering properties in a non-destructive way. A new apparatus has been developed, called the hybrid Rowe cell. It combines the functions of a traditional hydraulic cell and an electrical capacitor. This can simultaneously measure electrical properties and physical, mechanical and hydrological properties of soil samples, allowing to investigate their correlation accurately. The thesis focuses on the study of the applicability and calibration of the new hybrid cell, and the relationship between electrical conductivities of bulk peat and pore water which saturates the sample. The applicability of the hybrid cell was examined by using water as calibration. The results were then compared with values reported earlier and the experimental set-up was also compared with a similar one found in literature. As a result, it was proven that the new cell successfully prevented electrode polarisation and was applicable for this type of measurements. The electrical measurements on peat showed a relationship between the electrical conductivities of bulk peat and pore water which could be predicted well by a model previously developed for peat as well as the modified Archie's law. In addition, the modified Archie's law could be considered to be a persuasive model for the electrical behaviours observed in this study. For future research, it is expected that the frequency of input currents is extended to a lower range (<20 Hz), input electric currents are properly controlled to avoid non-linear effects, and also hydraulic consolidation tests are performed parallel to electrical measurements.

Denoising terrestrial laser scanning data for roughness characterization of rock surfaces

Dogan Altundag

The research presented in this thesis focuses on analysing the influence of terrestrial laser scanner (TLS) instrument noise on rock surface roughness characterisations. Data acquired by terrestrial laser scanner devices always contains noise. Therefore, using raw laser data for roughness characterisation can cause inaccurate and unreliable results. The described procedure can be applied to obtain roughness values. Linear profiles are generated on the rock surface and from these profiles roughness data is obtained using both terrestrial laser scanner and manual measurements. The manual measurements are used as reference data to compare with the results of laser scanning. From the measured data, fractal parameters are obtained to quantify roughness using the roughness length method. Both raw data and denoised laser scanner data is used to obtain fractal parameter values. In order to denoise the raw data, first the noise level is estimated. This noise level is used to determine threshold values in a wavelet based denoising approach. Before applying the threshold, the data is decomposed into approximation and details coefficients where the details also contain the noise. The separation is performed using both discrete wavelet transform and wavelet packets, which have a different method of operation. After decomposition of the data the threshold method is applied to the coefficients in order to remove the noise. The obtained roughness values from the denoised dataset are compared with the roughness values obtained from the manual measurement dataset. A total of 10 different thresholding methods are applied to the coefficients obtained from both discrete wavelet transform and wavelet packets. From the resulting denoised data the fractal parameters are obtained and compared with the reference data. Threshold values that result in the fractal parameter closest to that of the reference data are the following: for the x-directional profile, Donoho-Johnstone's *Fixed form hard* thresholding method and Birgé-Massart's *Penalized medium soft* thresholding method resulted in very close fractal dimension values to the reference values, when applied to the coefficients of wavelet packets. For the y-directional profiles, Birgé-Massart's *Penalized medium hard* thresholding method resulted in the closest value to the reference data.

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