Newsletter

nl-iaeg ingeo kring

Mining & Dredging edition

Winter 2011/2012

Urban geological mapping in Catalonia: Tarragona case study - Subsea cable installation and ploughing for offshore wind farms - Book review: 'Rudolph Glossop and the Rise of Geotechnology' - Cover article: Oil sands mining - Reservoir Geology excursion to Germany - Maasvlakte 2 project: reclaiming future land with historical sand - Geo-Frontiers 2011 - Internship Cofra B.V. - A.P. van den Berg successfully tests its latest deep water CPT technology for geotechnical site investigation - IAEG 2010 Congress: twenty years since Amsterdam and at the opposite end of the world - The use of dredged sludge as a fill in the Osthafen (Bremerhaven, Germany) - De Ondergrondse - Book review: 'Nutzung von Massenschwerebewegungen im Tagebau' - Mining gas hydrates: where and how to look - EUROCK 2009 and 2010 - Book review: 'Geological Engineering' - Dredging rock - OceanflORE - Thesis abstracts

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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Cover photos

Athabasca oil sands, Alberta, Canada

These two images from the Landsat satellite show the growth of surface mines over the Athabasca oil sands between July 1984 (back cover) and May 2011 (front cover). The Athabasca River runs through the centre of the scene, separating two major operations. To extract the oil at these locations, oil producers remove the sand in big, open-pit mines, which are tan and irregularly shaped. The sand is rinsed with hot water to separate the oil, and then the sand and wastewater are stored in tailings ponds, which have smooth tan or green surfaces in satellite images.

Owner: NASA Earth Obervatory

Source: http://earthobservatory.nasa.gov/IOTD/view.php?id=76559

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Editorial

Ir. Erik Schoute

Dear reader,

On behalf of the editorial board, I am pleased to present to you this edition of the Ingeokring Newsletter. Central theme of this winter 2011/2012 edition is *Mining & Dredging*. This issue is again filled with a broad range of articles like conference reports, thesis abstracts, book reviews, and theme related articles. The search for an interesting cover image is always one of the more challenging (and therefore exciting) parts of making a Newsletter. With the theme of this Newsletter in mind, we were mainly looking for photos of open-pit mines, large excavating equipment, vintage photos of dredging equipment, etc. But for some reason no image really stood out. Besides that, the requirements for a good cover image are that it needs to cover both the front and back of the Newsletter, and therefore an ideal image would be one that has the main part of the image on the front, and a less important part of the image on the back. For instance, with the cover of the Geophysics edition of 2009, the satellite image showed the Netherlands on the front while parts of the North Sea and Great Britain were shown on the back. The source of this image was NASA's Visible Earth website, which contains a collection of beautiful satellite imagery of the entire planet (visibleearth.nasa.gov). When I was visiting the website again recently, to look for satellite images of open-pit mines, I stumbled upon a recent photo of the Athabasca Oil Sands in Alberta, Canada, the world's largest oil sands deposit. In the accompanying captions, a link was given to a similar NASA website called Earth Observatory (earthobservatory.nasa.gov). At this site, a

23 July 1984

15 May 2011

wonderful series called *World of Change* shows series of satellite images taken over the years highlighting many different aspects of Earth's constant changes, from the urban sprawl of Dubai to the dramatic shrinking of the Aral Sea. The development of the Athabasca Oil Sands is shown in 28 images taken between 1984 and 2011. Large-scale mining activities started here in 1967. As can be seen in the images, from 1984 to 2000 a relatively slow development took place, and between 2000 and 2011 a more rapid expansion of the mining complex is visible, partly triggered by an increased oil price. To illustrate the overall growth of the mining activities in the Athabasca Oil Sands, the cover of this Newsletter shows exactly the same area on the back and front, the back photo taken in 1984 (the first of the *World of Change* series about the Athabasca Oil Sands), and the front taken in 2011. A dramatic increase in the mining complex' boundaries is visible, whereby the sheer size of the complex is illustrated with the scale bar given in the images above. Michiel Zandbergen, who received his MSc degree in Engineering Geology in Delft in 2005, is currently working at the Athabasca Oil Sands mining site as a geotechnical engineer for Klohn Crippen Berger. Michiel has written an interesting contribution to this Newsletter, about oil sands mining in the Athabasca Oil Sands.

Our gratitude goes out to the authors of the published articles and to the companies who have made printing of this issue (financially) possible by sponsoring through advertisements. We hope you will enjoy reading this Newsletter!

Urban geological mapping in Catalonia: Tarragona case study

Guido Rutten, Bert Lietaert, Dimitrios Baltoukas & Dominique Ngan-Tillard *(Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Introduction

The *Institut Geologic de Catalunya* (IGC) is in the process of creating very detailed geological maps at a scale of 1:5,000 (see Figure 1). The availability of accurate, detailed information on the subsoil should facilitate decision-making in urban planning, geo-engineering works, soil pollution remediation, and other important environmental issues that society should deal with in the future. Students from TU Delft's Engineering Geology Master conducting fieldwork in the region were invited to join IGC's geologists on a site visit for the mapping of Tarragona. The visit was organised within the framework of a collaboration agreement signed between TU Delft and IGC.

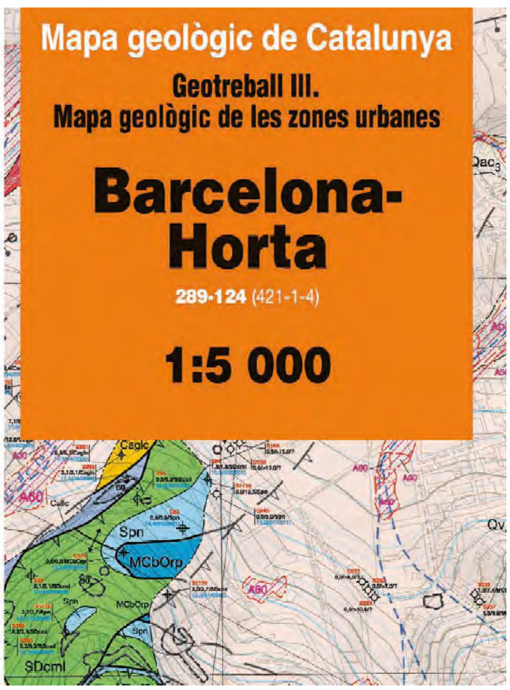


Figure 1 New series of urban geological maps at 1:5,000 scale produced by IGC.

Antecedents for urban mapping in Catalonia

The origins of this project can be traced back to 2005, when a major collapse occurred during extension works for Barcelona's metro line 5. The collapse created a surface cavity of 25 m in diameter and 32 m deep, undermining two apartment blocks which partly collapsed leaving more than 50 families homeless. The cause of the failure could be contributed to different factors such as shallow works in a bad quality rock mass covered by colluvial Quaternary deposits and anthropogenic materials infilling a river valley. Soon it became clear that a systematic urban geological mapping program would provide valuable resources to limit geohazards in future construction works. In this context, the Catalonian government created IGC, whose principal mapping program generates urban geological maps. The objective of the urban mapping project is to provide accurate geological information of county capitals and towns of more than 10,000 inhabitants in Catalonia. The urban zones of 131 towns will be surveyed for this project, totalling an area of about 2,200 km^2 to be mapped in 15 years, starting in 2007. According to the 2008 census, 82% of the population of Catalonia (ca. 7,242,500 inhabitants) lives in the areas to be mapped in this project. Given the detailed scale and free availability of the data, this project is unprecedented in the world.

Tarragona geological trip

Welcomed by IGC geologists Miquel Vilà and Aline Concha and two of their contractors David Albalat and Narcís Carulla, we took off to find out what they were doing exactly, which problems they have encountered so far and ,of course, which geological features Tarragona has to offer. During the different stops we discussed the geological maps and cross-sections the team had prepared, but also the usability of their product. One of the stops was located near Punta del Miracle (Figure 2) at the eastern end of the Platja del Miracle. There are several superb outcrops at this location, where it is further possible to observe an unconformity between Jurassic dolostones and Miocene conglomerates and bioclastic sandstones. The Jurassic basement is folded and affected by different sets of deformation structures. The Miocene rocks horizontally overlie the Jurassic basement and at this place define a coarse normal grading sequence. From a paleontological point of view it is conspicuous that abundant ichnofossils are present at the unconformity surface and macrofossils of coastal environments in the Miocene. In the centre of Tarragona we stopped to see the re-

Figure 2 Unconformity at Punta del Miracle.

mains of an old Roman city wall (Figure 3). Tarragona used to be called *Tarraco* in Roman times and was an important city in the Roman province of Hispania. Nowadays there are many remnants of this period: parts of an old city wall, the amphitheatre, and the aqueduct. The building stones for these structures are mainly Miocene calcarenites, originating from quarries surrounding the city of Tarragona. The decay of these monuments is highly dependent on the mineralogy and the fabric of the rock, as well as on the environmental conditions to which the monuments are subjected. Different forms of decay are observed on these monuments. For the ancient city wall, the most important ones are granular disintegration, differential erosion, and development of black and orange patinas. The black patinas are mainly formed by

Figure 3 Roman city wall (top) with differential weathering (bottom).

the dissolution of the limestone matrix by acidic rain. When the dissolved limestone precipitates again, black pollution particles are taken up and a black crust/patina is formed at the outside of the rock. The orientation of the city wall towards the sea is one of the reasons for the occurrence of granular disintegration. Water 'spray' coming from the sea is rich in NaCl. This spray penetrates the pores of the building stones and NaCl precipitates inside the pores. Salts can swell and shrink under the influence of water and temperature,

Figure 4 Roman Amphitheatre in Tarragona.

causing stress inside the rock. Furthermore, these oscillations in temperature and water adsorption also cause the clay minerals inside the matrix to swell and shrink. These processes cause stress inside the rock and initiate granular disintegration. Last, we visited the Roman amphitheatre which forms the heart of the historical city. Our guides drew us a simple sketch of the geological setting of the amphitheatre, showing that the theatre was constructed of soft rocks (marls, mudstones, and gypsum) from Keuper facies (Figure 5). At this place, the Keuper beds are vertical and confined to the south by laminated Triassic limestones and to the north by massive Jurassic dolostones. This is a very early example of urban planning where buildings are constructed on soft rocks that are easy to excavate. The IGC urban geological mapping program tries to capture these kinds of geological features, amongst others by providing tables with rock mass properties of the different stratigraphic units.

Catalonian Urban Geological Map 1:5,000 series

This field trip allowed us to become familiar with the main features of this ambitious Catalonian Urban Geological Mapping program. The project presently integrates the following types of information in a GIS environment:

- Data from pre-existing geotechnical reports, historical geological and topographical maps, and from historical aerial photographs.
- Data from available borehole databases.
- Geological characterisation of outcrops inside the urban network and neighbouring areas.
- Geological, chemical, and physical characterisation of representative rocks, sediments, and soils.
- Orthophotographs (0.5 m pixel size) and digital elevation models (5 m grid size) made from historical aerial photo-

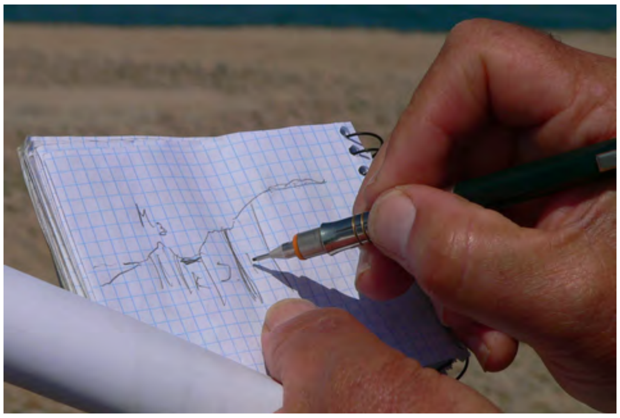


Figure 5 Sketch of the construction of the Roman amphitheatre in Keuper marl.

graphs, to depict land use changes, artificial deposits, and geomorphological elements that are either hidden or destroyed by urban sprawl.

- Detailed geological mapping of Quaternary sediments, subsurface bedrock, and artificial deposits.
- Data from subsurface prospecting in areas with insufficient or confused data.
- 3D modelling of the main geological surfaces such as the top of the pre-Quaternary basement.

All gathered data is harmonised and stored in a database. Analysis of the database allows to compile and print the 1:5,000 scale urban geological maps according to the 1:5,000 topographic grid of Catalonia. The map is composed of a principal map, geologic cross sections, and several complementary maps, charts, and tables. Regardless of the geological map units, the principal map also includes the main artificial deposits (such as infilled river valleys and road embankments), very recent or current superficial deposits, contours of outcropping areas, structural data and other relevant information gathered in stations, sampling points, boreholes indicating the thickness of artificial deposits and the depth of the pre-Quaternary basement, contour lines of the top of the pre-Quaternary basement surface, and water level data. The complementary maps and charts may change depending on the gathered data, the geological features of the area, and the urban typology. However, the most representative complementary maps that the printed urban map includes are the Quaternary subsurface bedrock map and the isopach map of thickness of Quaternary and anthropogenic deposits. The map also includes charts and tables of relevant physical and chemical parameters of the geological materials, harmonised downhole lithological columns from selected boreholes, and photographs and figures illustrating the geology of the mapped area and how urbanisation has changed the natural environment. Engineering geological parameters such as the mean discontinuity spacing and material strength are given per formation.

Feasibility of urban geological mapping

Clearly, we were impressed by the work of IGC's team, notably their enthusiasm and dedication to their work. Nonetheless, critical questions had to be asked. Who will use their maps, and is it worth the money and time spent on it? What is the legal responsibility of IGC, when constructors use their data? And why don't these constructors just share their knowledge? Being a pioneering project, the answers to some of these questions remain yet unanswered. It is sure that for delicate construction projects, detailed studies including boreholes will still have to be carried out to obtain greater accuracy, but less complex projects will certainly benefit from the available data. The IGC staff members pointed at the mention on the maps indicating that the information shown is only valid at a scale of 1:5,000. The

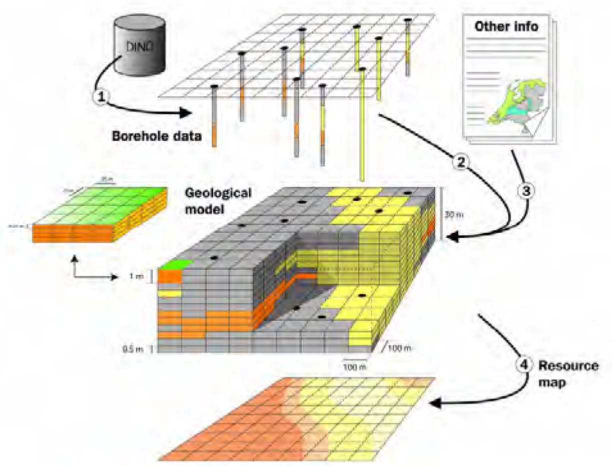


Figure 6 GeoTOP model used for resource predictions (source: www.dinoloket.nl).

suggestion of a legal disclaimer on each map, to avoid claims in case data proves to be false, was taken up with interest. Spain, including Catalonia, lacks regulations stipulating that constructors must share the geological knowledge they obtain in their projects. The framework chosen by Catalonia to bridge this gap might be of interest to other countries, especially in the light of the rise of international project tendering, where ground investigation data should be accessible for all bidding parties. Who has to pay for the data remains a question! The Netherlands is also working on the obligation to deliver ground data for any engineering project, coupled to the compulsory use of archived data for new projects and the necessary declaration of wrong or dubious data. The system is expected to be launched in 2015. Synergy with other EU countries regarding for example data format is searched.

Approach to urban mapping in the Netherlands

Our guides were curious about the Dutch approach to urban mapping. This article gives us the opportunity to depict the state of the art in (urban) mapping in the Netherlands using information from DINOLoket and presentations from the First International Conference on Frontiers in Shallow Subsurface Technology in Delft (January 2010). A very large amount of ground data, mainly borehole logs, Cone Penetration Tests (CPT's), and groundwater information is already available in DINOLoket, the database of the Dutch Geological Survey. In 2006, the Dutch Geological Survey started the TOPINTEGRAAL project in the Netherlands. This project aims to build a detailed 3D geological model of the upper 30 to 50 m of the subsurface of the Netherlands, based on the existing context of geological mapping, and characterise the various geological units in terms of lithological, hydraulic, geochemical, and geotechnical properties. This upper part of the surface is referred to as the GeoTOP.

The 3D geological model consists of millions of grid cells with a dimension of 100 x 100 m in the horizontal plane and 0.5 m in the vertical direction. Each grid cell is coupled to estimates of specific parameter values such as stratigraphic unit, lithology and lithofacies, and physical and chemical parameters. Evidently, the boreholes are the base of the model. They provide detailed information about the characteristics of the subsurface at one specific location. First, they are schematised into units that have uniform sediment characteristics, using both lithostratigraphic and lithofacies criteria. For example, for a certain member of a given formation, sandy river channel deposits and clayey floodplain deposits are distinguished. Second, the upper and lower boundaries of each lithostratigraphic unit are modelled. Each cell of the 3D model is attributed to a given lithostratigraphic unit. Third, data within a lithostratigraphic unit is interpolated stochastically in 3D to predict the probability of each of its cells to belong to a certain lithofacies. Engineering and scientific judgment is integrated into the interpolation process. High-resolution airborne altimetry data can reveal inverse landscape related to sand channels buried within more compressible confining layers. They are used to constrain the interpolation process. For sand, an estimated grain size class is provided. In the future, CPT's will be integrated into the 3D model and geotechnical parameters will be estimated, using empirical relations with cone parameters. The GeoTOP concept can be used for different user applications, both in urban and non-urban environments. Examples are groundwater studies, subsidence calculations, volume calculations of natural resources, and site selection for large infrastructure works. An interesting application model in an urban environment is the use of the 3D model in the planning for a new subway in Rotterdam. Here, the model was used as a background (frame) model in order to plan detailed studies of sediment variation at sub-grid scale (1-10 m). Using the GeoTOP methodology, ground models can be produced at the scale required for engineering works. Recently, a 20 x 20 x 0.5 m high-resolution GeoTOP model has been built for the RandstadRail tunnel project in Rotterdam by integrating borehole and CPT data available in local databases and not yet in DINOLoket. For comparison, the smallest feature shown on the 1:5,000 urban geological maps of IGC is about 5 m wide. The RandstadRail model predicts the depth to the first Pleistocene sands and probability to encounter buried sand channels. Such information can be used to select best tunnel alignment and limit geohazards during construction.

Conclusion

For us, the Catalonian Urban Geological Mapping program is certainly an example of a very interesting, multidisciplinary project in engineering geology. We would like to thank the geologists of IGC for their enthusiasm in showing us their work and their curiosity about the Dutch approach to urban geological mapping. Those interested in the project are invited to visit IGC's well-documented website (www.igc.cat). Finally, we wish the IGC a very fruitful future for this initiative. During the excursion it was decided that IGC staff would join the N420 slope stability excursion and be initiated to the rock Slope Stability Probabilistic Classification system (SSPC) during the 2011 TU Delft fieldwork. The SSPC system has been developed in Catalonia by Robert Hack (ITC), Niek Rengers, and the late professor David Price, and it has been exported to many countries by Robert, Niek, and ex-ITC students. It is still being used by TU Delft during its Spain engineering geology fieldworks. And one day, the SSPC parameters might be displayed at the back of the 1:5,000 urban geological maps produced by IGC!

Subsea cable installation and ploughing for offshore wind farms

Ir. Lennart van Baalen *(Visser & Smit Marine Contracting, Papendrecht, the Netherlands)*

Introduction

The work of Visser & Smit Marine Contracting (VSMC) involves installing, trenching, and connecting subsea power cables to offshore wind farms. VSMC is part of the Volker Wessels conglomerate and is involved in many offshore cable projects in the North Sea and the Baltic Sea. In order to stay ahead of the competition, VSMC continuously tries to expand its knowledge about the operations they perform. Main aspects of these operations are cable properties, cable installation equipment, and soil properties along the seabed route. In this article these aspects will be discussed.

Figure 1 VSMC's cable-laying vessel Stemat Spirit.

Offshore wind farms

Offshore wind farms are generally located in water depths ranging from 20 to 40 m and consist of 50 to 100 windmills that are linked with infield cables to an offshore substation platform inside the wind farm. From this substation platform, an export cable is running towards the shore. The export cable is typically the longest and heaviest cable to be installed and has a diameter of 150 to 250 mm and weighs about 90 kg/m. Since most windmill farms are about 20 to 50 km offshore, an export cable may weigh as much as 4,500 ton. These cables are installed in one single section, since a single joint made at sea may cost as much as 1 million euro. When loading such a long cable length, the cable-laying vessel is moored directly at the cable manufacturing facility to load the cable in one piece. Loading speed is between 3-10 m/min., and depending upon cable size loading may take several days, see Figure 2.

Figure 2 Loading a power cable.

Stemat Spirit

VSMC operates several cable installation vessels and barges of which the *Stemat Spirit* is the largest and most modern vessel. It is a fully equipped and highly specialised cable-laying vessel for large power cables, with a load carrying capacity of 4,500 ton. It is also very manoeuvrable, even in extremely shallow water, and has sufficient deck space for handling and trenching equipment. The vessel uses dynamic positioning (DP) to automatically maintain position by using its own propulsion. DP has significant speed advantages over anchor systems, but the system does affect a vessel's capability to work in very shallow water, such as near onshore landing points. Therefore, to keep the cable-laying vessel in position in very shallow water, *Stemat Spirit* also uses an anchor system. Hence, it is able to control its position, speed, and heading by a combination of DP and anchor winches.

Cable installation

Typically, the offshore cable-laying operation starts near-shore at the shore connection, which is one of the most critical operations. Here, an open trench will be made, or a horizontal directional drilling whereby a borehole is drilled underneath the sea defence. The *Stemat Spirit* will approach the landing location as close as possible. Due to the retractable thrusters and the flat bottom, it may even be positioned on a dry beach at low tide (see Figure 3). Now, a pull-in wire from an onshore winch pulls the export cable from the vessel towards land using either floating devices attached to the cable or using cable rollers positioned at the beach. Environmental conditions like water currents and wind can severely affect these operations. Once the final

Figure 3 Stemat Spirit *on a dry beach at low tide.*

onshore destination has been reached, the vessel can start laying the cable. During cable installation the power cable is hangs like a 'lazy S' between the vessel and the seabed. Tensioners, a pair of hydraulically operated tracks, grip the cable and apply tension to the power cable during laying in order to prevent the cable from bending too much at the seabed. The synchronisation of the moving vessel, the turntable which pays out cable, the tensioners and the subsea trenching machine at the seabed must be carefully orchestrated.

Cable burial

The potential risks for damage to a subsea power cable are risks caused by dragging anchors, fishing gear, and dropped objects. Therefore, most cables are buried in the seabed to a depth of 1 to 3 m below seabed, which is called trenching. The larger burial depths are typically in higher risk areas or instable seabeds. Infield cables are most often trenched using a Remotely Operated Vehicle (ROV). These are available in all sizes and can be equipped with water jets, pumps, or even cutting chains. Export cables may also be trenched using a subsea cable plough. The *Stemat Spirit* uses VSMC's *Sea Stallion 4* cable plough in order to trench the cable up to 3 m below seabed, see Figure 4. Cable ploughing is a brute force method and therefore capable of burying cables in soils like very stiff overconsolidated clay, even when it contains large boulders. The cable plough is towed by the cable-laying vessel, whereby the ploughshare cuts a narrow slit in the soil into which the cable is guided. The use of a cable plough may require very high tow forces (up to 120 ton) and may severely slow down the installation process.

Research on offshore cable ploughing

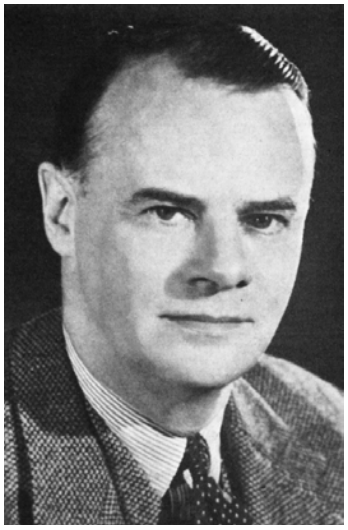
VSMC uses dynamic simulations using computer software like *Orcaflex* in order to calculate the effect of the sea state on forces and stresses on the power cable during installation, forces and movements during crane operations, and motions of the vessels. However, for offshore cable ploughing no calculation methods or simulation models are yet available. For this reason, VSMC is developing a simulation model to assess the plough performance. The aim of this research is to gain a better understanding of shear planes in the soil and forces acting on the plough while trenching through different types of soils, varying from loose sand to very stiff overconsolidated clay. This simulation model will be evaluated using the actual logged computer data gathered during previous projects. When ploughing offshore cables, all information like pull forces, speed, and burial depth is logged. In addition, during ploughing at the landfall site on the beach extra site investigation, plough performance observations, and other measurements are performed in order to gain information to evaluate the computer simulation model. Research like this will help the company to stay ahead of competitors in the offshore power cable installation industry.

Figure 4 Cable burial using a cable plough.

Book review
Rudolph Glossop and the Rise of Geotechnology

Reviewed by ***ir. Erik Schoute*** *(Senior Engineering Geologist, Royal Boskalis Westminster nv, Papendrecht, the Netherlands)*

Rudolph Glossop and the Rise of Geotechnology starts with the words *'this book is not a biography but a story about the development of the art and science of ground engineering in the United Kingdom during the 20th century'*. And that summarises more or less the strength and weakness of this publication. The fact that the reader is provided a unique window into the development of geotechnical engineering in the UK, through numerous diary entries, journals, letters, and articles is very interesting and makes for a great read. On the other hand, because the book is not written as a biography, the person behind the great geotechnical engineer whose work is highlighted, remains slightly unknown to the reader. An introductory biographical chapter with some photos would have been a welcome starter. Having that said, it is time to dive into what the book is actually about. Rudolph Glossop (1902-1993) studied mining engineering at Royal School of Mines (Imperial College London), where he graduated in 1924. During his career as a professional geotechnical engineer he worked in Canada, the Gold Coast (modern-day Ghana), and in the UK, where he was main board director of John Mowlem & Co. Besides this, he was also a founding member of the British Geotechnical Society and the Engineering Group of the Geological Society.



Rudolph Glossop in 1951 (photos in this article are from the reviewed book).

This book looks rather chaotic at first instance , with different type settings for journals, papers, letters, etc. But once the stories and personal accounts unfold, the interesting content takes over and the layout turns out to be of minor importance. The book's main chapters are 'Selected journals', 'Later journals', 'Selected diaries and letters', 'Selected writings', and 'Early technical papers'. Especially the 'Selected journals', which span 86 pages and cover Glossop's time at the Royal School of Mines, at the Gold Coast, and during World War II, read like a novel. The 'Later journals' (32 pages) are more written as diary entries and cover the period between the end of World War II and 1960. The subjects of this chapter are a portrait of Dr. Louis Leakey, a visit of Glossop to the Norwegian Geotechnical Institute in 1955, a short entry on Persian highways, and a visit to North America in 1958.

Then, in the chapter 'Selected diaries and letters', the reader gets probably the most intimate insight into the personal friendships that Rudolph Glossop had with the great names in geotechnology. The diary entries from 1960 and 1961 cover mostly engineering challenges during the construction of Derwent Dam in Derbyshire. The selected letters cover correspondence between Glossop and Laurits Bjerrum (started after Glossop's visit to the NGI in 1955, mentioned above), Karl Terzaghi (spanning the period between 1945 and 1962), and Alec Skempton. Most letters are however from the mentioned persons to Glossop, which leaves the reader behind a bit disappointed as it would have been interesting to see a few more of Glossop's own letters as well.

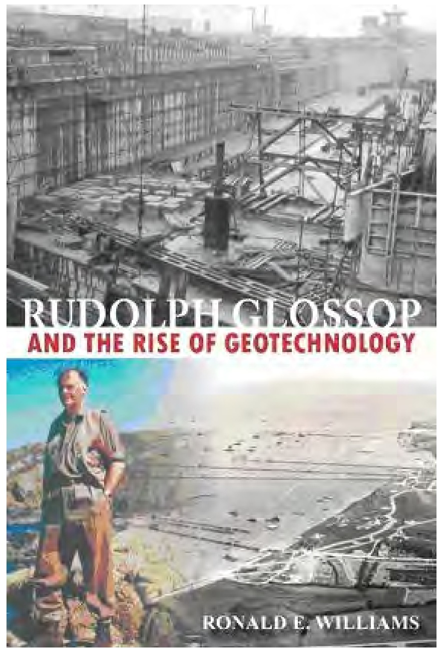
A remarkable letter from Terzaghi to Glossop says: *'Five weeks ago, when I was grabbed and deposited in the hospital I realized that this was not my first heart attack. I experienced the first one three years ago, while I was coping with a vicious landslide in Brazil. Since there was no physician at the site I had to proceed in accordance with my own medical instincts. I absorbed a generous double scotch whisky (…) mixed with hot water, slept ten hours like a log and next morning I declared myself perfectly cured.'*

Rudolph Glossop, Karl Terzaghi, and Alec Skempton in Avebury, UK, shortly after World War II.

What follows is arguably the most confusing and superfluous chapter of this book. The chapter 'Selected writings' spans 47 pages and contains copies of articles written by Glossop. Main topics are the development of the British Geotechnical Society and the Engineering Group of the Geological Society, and their respective journals, *Géotechnique* and *Quarterly Journal of Engineering Geology*, and the influence of Karl Terzaghi on the development of geotechnical engineering in the UK (which is presented by means of a book review written by Glossop on Terzaghi's *From Theory to Practice in Soil Mechanics*, a 1939 Annual Report of John Mowlem & Co., a personal tribute to Karl Terzaghi written by Glossop after Terzaghi's death in October 1963, and two more papers by Glossop).

The book ends with the chapter 'Early technical papers' (48 pages), in which two copies of papers are printed. The papers were written by Glossop and Hugh Golder ('The Construction of Pavements on a Clay Foundation Soil', 1944), and Glossop and Alec Skempton ('Particle-size in Silts and Sands', 1945). The papers give a nice overview of the state-of-the-art of both topics in the 1940's, illustrated with photos and drawings. The concluding chapters of the book are somewhat confusing, in that there is one list that covers people mentioned in the diary entries (chapter 'Selected diaries and letters'), while another list is a name index of all people mentioned in the book. A single name index would have been more clear, perhaps with an additional subject index. Further, a bibliography of Glossop's publications is given, and a list of references of which it is not clear in which chapters these publications are mentioned. It would have been more elegant if the two pages of advertisements at the back of the book were omitted.

Concluding, it can be said that this book is interesting enough to be picked up every now and then to read a few interesting chapters and think about the pioneering work that was done for our profession by Glossop and his contemporaries.



Rudolph Glossop and the Rise of Geotechnology

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Cover article: Oil sands mining

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Introduction

Oil sands deposits in the Fort McMurray area in northern Alberta (Canada) are currently subject of large investments and vast mining projects by several oil companies. The major mining companies in the region at this moment are Suncor, Syncrude, Shell, Imperial Oil, and Canadian Natural Resources Ltd. (CNRL). See Figure 1 for the locations of oil sands deposits in Alberta.

These oil sands projects are very complex as they allow for mining, extraction and upgrading in a single process. The size and extent bring several geotechnical and hydrogeological challenges. Surface mining for example can be challenging due to its proximity to aquifers. The tailings ponds which can reach up to 80 m high can be very challenging due to variable subsurface conditions and their exceptional height. This article will explain the basics of oil sands mining operations and the associated tailings pond construction and operation in northern Alberta.

Figure 1 Oil sands deposits in Alberta, Canada.

Figure 2 Athabasca River and open-pit mine.

First, the local geology of the Athabasca oil sands deposit in northern Alberta will be reviewed. The definition of oil sands and the oil sands extraction operations will be looked at next, followed by a more detailed overview of a mine start-up and common oil sands mining operations. The storage of tailings and the reclamation of both the mine and tailings ponds are discussed in the final two sections.

Geology

Most of the bitumen reserves in the Athabasca oil sands deposit are contained in laterally discontinuous, upward-fining channel sand bodies in the Cretaceous McMurray Formation. In the western part of the deposit, significant reserves occur in laterally extensive marine bar sands at the top of the McMurray Formation. The McMurray Formation was deposited in a north-south trending depression on an erosion surface of Devonian limestone. The highly variable relief on this paleo-topographic surface is the most important single control on the distribution of facies and reserves (Flach, 1984). Division of the McMurray Formation into the lower, middle, and upper members was found to be valid over most of the mineable Athabasca oil sands deposit. The lower member is of fluvial origin and fills in the deepest lows on the Devonian limestone surface. Rapid sea level rise during middle member time resulted in a lowering gradient of the fluvial system and a change in channel type from the shallow (5 to 10 m), commonly coarse-grained channels of the lower member to deeper (20 to 30 m), narrow, sinuous, high suspended load channels of the middle member. The middle member channels were subject to invasion by salt water and were associated with coastal plain and lake deposits. During upper member deposition, the open sea had

Figure 3 Winter drilling.

invaded the northern and western parts of the area where upward-coarsening offshore marine bar sands are common (Flach, 1984). The McMurray Formation, which can be up to 60 m thick, is covered by Pleistocene (glacial tills and glacial fluvial/lacustrine deposits) and Holocene deposits (aeolian, fluvial, and lacustrine deposits capped by *muskeg*, a peat formation). These deposits can be very shallow and the McMurray Formation can be found less then 3 m below surface.

Most of the geological drilling is primarily conducted in winter conditions (see Figure 3), as travelling over frozen muskeg deposits provides safe and easy access for equipment. Before drill rigs head into the target areas, light wide track dozers clear pathways and 'pound in the frost' to get a frozen crust where the drill rigs can travel over.

Crude bitumen and bitumen extraction

What is crude bitumen and how is it extracted from the oil sands deposits? The crude bitumen contained in the Canadian oil sands is described by Canadian authorities as *'petroleum that exists in the semi-solid or solid phase in natural deposits. Bitumen is a thick, sticky form of crude oil, so heavy and viscous (thick) that it will not flow unless heated or diluted with lighter hydrocarbons. At room temperature, it is much like cold molasses'.*

Crude bitumen can be extracted from the oil sands through several different methods and processes. One of the most used methods around the Fort McMurray area involves open-pit surface mining (see Figure 2); this basic method consists of the removal of overburden and subsequent ore mining and processing. Deposits can currently be economi-

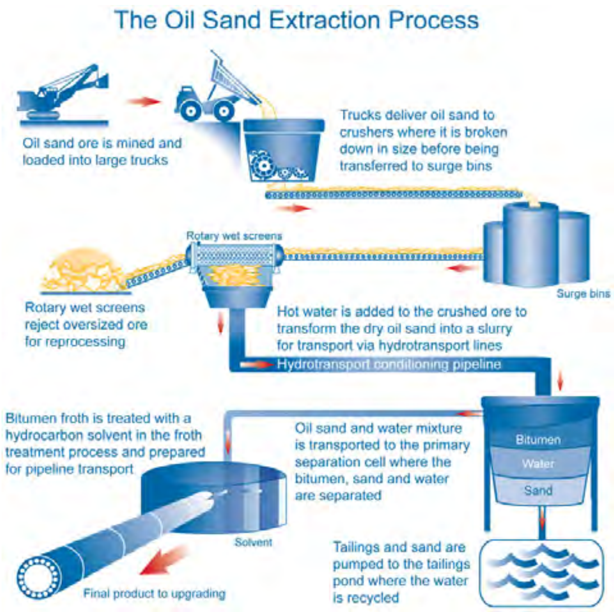

Figure 4 Schematisation of the oil sands extraction process (Hatch, 2008).

cally mined down to a depth of 70 m, a (further) increase in oil price is likely to increase the mineable depth. Depths of up to 100 m might be possible at oil prices in the range of $150 per barrel. The mining operations and extraction processes will be looked at in more detail in the following sections.

Another major method is *Steam Assisted Gravity Drainage* (SAGD). This method uses two directionally drilled boreholes. The upper borehole is used to inject steam into the oil sands deposit, which makes the bitumen less viscous. The bitumen then flows (assisted by gravity) to the lower borehole where it is collected and pumped to a refining facility. This technique needs about 70-80 m of overburden to contain the steam, it is therefore an ideal method for deeper oil sands deposits that cannot be mined economically. Other methods used include bitumen extraction by *Cyclic Steam Stimulation* (CSS). This is a method where steam is injected into a borehole over a certain period of time and, after a 'soaking' period, bitumen is pumped to the surface. 'Cold flow' is also used, where bitumen is fluid enough to flow by itself. The major downside of this method is the limited recovery. The above mentioned methods and processes will not be reviewed in further detail.

Oil sands mining

In this section, a closer look will be taken at the oil sands mining operations in and around the Fort McMurray area. Oil sands mining started here in 1967 when Great Canadian Oil Sands (now Suncor) started their mining operations. Bucket-wheel excavators were the technology of choice at the time, combined with conveyer belt systems to transport the ore to the plant for processing. Modern day mining operations use electric and diesel powered shovels in combination with heavy haulers (up to 450 tons) to remove the overburden materials and mine the ore. The reasons for using shovel and truck excavation methods are the increased reliability of equipment, better management and more flexibility in ore mining, ore blending for improved recovery, elimination of clay interburden, and less required manpower.

The start-up phase of a mine consists of the removal of vegetation and construction of light vehicle/haul roads and surface water drainage systems to allow site access. This phase is followed by the construction of (semi-)temporary camps, maintenance buildings, and office accommodations for staff and consultants to aid the mine operations. The next construction phase is the (partial) removal of overburden materials to uncover oil sands. Overburden materials are either hauled to waste dumps or, when suitable for construction purposes, hauled to the starter dikes of the tailings ponds or other (infrastructure) projects. If required, dewatering wells are installed to facilitate future mining operations. These wells are either for Pleistocene channel aquifer dewatering/depressurisation (to prevent pit floor blowouts) or pit wall dewatering. The ore mining operations commence when sufficient overburden materials have been removed and processing facilities are commissioned. Bench heights vary generally between 5 and 20 m, with a preference for heights around the higher range (depends on excavator type used). Benches higher than 20 m can be excavated but require close geotechnical monitoring as this type of excavation is often based on 'controlled' slope instability. Like in every ore body, separate bodies of waste materials (oil sands with generally less than 7% bitumen) exist that require to be moved to facilitate further mining operations. These *interburden* materials are regularly used as construction materials for (in-pit) tailings pond dikes, if unsuitable for construction purposes, the materials are hauled to the various waste stockpiles on site. The excavated ore is hauled to one or more central collection areas where crushers grind the large (frozen) lumps to smaller size particles (generally smaller than 2-3 cm).

Oil sand extraction from ore

The major challenge is to separate the 'sticky' bitumen from the solid particles. After crushing the ore it is mixed with warm water in large vessels or *surge bins* where the mix is conditioned. The conditioned slurry is pumped to screens to remove any large particles that could not be properly conditioned. These large particles are rejected and hauled to stockpiles on site or are reintroduced into the crusher or the

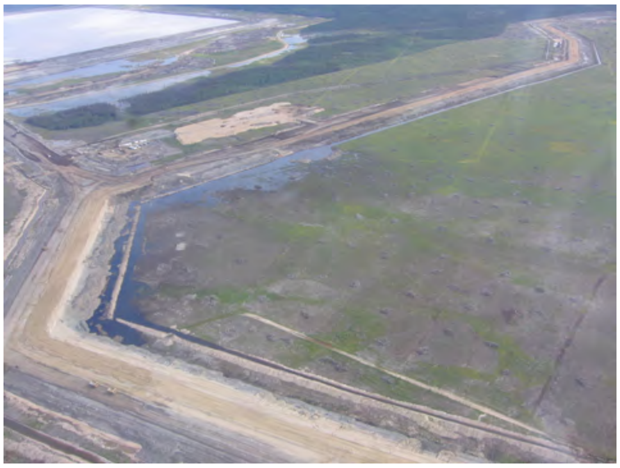

Figure 5 Tailings pond starter dike.

surge bins. The water and oil sands mixture is pumped to a separation cell. Here, the mixture separates into three layers: sand, water, and bitumen. The bitumen is skimmed off the top to be cleaned and further processed. The clean sands from the bottom of the separation vessel are pumped to the tailings ponds. The middlings zone, a combination of finer particles (silt/clay), water, and some bitumen, is transported to other systems for further processing. The middlings zone materials still contain small quantities of bitumen. To prevent this bitumen from being pumped to the tailings pond, secondary recovery processes remove this bitumen and increase the overall bitumen recovery. The finer particles and water that are left are thickened by using cyclones. The separated water is recycled and reused for plant operations and the thickened tailings are pumped to the tailings ponds for storage. All recovered bitumen is, as mentioned earlier, cleaned and then transported to upgraders where the bitumen is transformed into various hydrocarbon products. See Figure 4 for a schematisation of the extraction process.

Tailings ponds

As mentioned in the previous section, two tailings streams leave the processing plant and are stored in tailings ponds. But what is a tailings pond, how does it work, and how is it operated? This section will look at the construction of tailings ponds, their operation, and the associated geotechnical challenges. An oil sands tailings pond is a containment structure to hold process-affected water and waste products (tailings) from extraction operations. Tailings ponds are very large structures with sizes that are often larger than 10 km^2. Besides their large dimensions, their heights can also be extraordinary (up to 80 m). The potential risk of a dam failure is relatively low due to stringent design criteria, but the impact of an actual dam breach would be catastrophic. The various dams are therefore subject to extensive geotechnical monitoring. In fully operational tailings ponds several hundreds of piezometers, slope inclinometers, settlement markers, and other monitoring instruments are no exception. Other systems that can be in place include pressure relief and seepage management systems. A pressure relief system includes depressurisation wells that assist in pore pressure reduction in the foundation under the tailings pond and dikes to maintain stable slopes and allow for construction to full operating heights. Seepage management systems control the water flows out of the tailings pond and return the water back to the pond itself creating a closed water system. A tailings pond often starts its life as a series of mechanically placed starter dikes (see Figure 5). As mentioned before, suitable mine waste is used for this purpose. Toe berms, shear keys, or combinations of both are often required to allow for stable slope conditions due to the weak Pleistocene and Cretaceous (Clearwater Formation) clays commonly found in the Fort McMurray area.

When the starter dikes are completed and mine and extraction operations have commenced, the sand tailings from the separation cell are used to build the tailings dams to higher elevations and increase the storage volume in the pond. In the summer season the sand tailings are used to build cells on the dike, the cell construction is managed by dozers and berms to assure proper compaction and dike dimensions. During the cold winter months, fog forms off the warm tailings making placement operations difficult and unsafe. The sand tailings are therefore placed in the form of tailings *beaches*. During beaching, sand tailings flow freely into the tailings pond and create long beaches (100 m and up). The construction speeds are governed by the weak foundation materials and the associated pore pressure dissipation, which are closely monitored by an array of geotechnical instrumentation.

The fine tailings stream from the plant poses more challenges. The fine and weak nature of thickened tailings does not allow this type of material to be used for construction purposes. The thickened tailings are therefore often stored in separate ponds surrounded by centreline dikes. These centreline dikes are made of either sand tailings or suitable mine waste. Once the tailings ponds have been filled to their maximum allowable height, the deposition of tailings will often continue in the mine pit itself. The mined area is usually converted into an in-pit tailings pond by means of in-pit dikes. These tailings ponds will then trail the mining pit advance, delayed by several years.

Each barrel of oil produces about 0.3 m^3 to 0.5 m^3 of fine tailings. These fine tailings pose challenges with regards to settlement, consolidation and future reclamation. The main legacy of oil sands mining will be the massive tailings ponds, which can take many decades before they can be reclaimed. The tailings deposits are therefore now subject to new regulations by the ERCB (ERCB, 2009), which requires them to have an undrained shear strength of 10 kPa within 5 years after active deposition to allow for easier and quicker reclamation. Several different techniques are currently being implemented by the various oil companies to comply with the new regulations.

Reclamation

Several of the older open-pit mines and tailings ponds are now getting closer to the end of their operating life. The oldest tailings pond of Suncor (commissioned in 1967) has been reclaimed in 2010. A total of more than 600,000 trees were placed over an area of 225 hectares (CTV News, 2010). More reclamations are to follow in the coming decades. In order to prepare for these projects, mining companies are required to stockpile soils and seeds/soil mixtures from within their lease boundaries. Any land that has been disturbed by the mining and tailings operations is to be transferred back into a natural state using these stockpiled materials. As mentioned in the previous section, reclamation of tailings ponds is a long and slow process. Closed loop water management systems are required to control the outflow of process affected water from the tailings ponds for many decades to come. Additionally, impervious layers are placed on the tailings pond and dikes to prevent precipitation inflows into the tailings ponds and limit the amount of process-affected water to be managed.

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‘Een project is nooit
een kunstje dat je
kunt herhalen.’
Micheline, projectleider serious gaming

Reservoir Geology excursion to Germany

Drs. J.C. Blom *(Lecturer, Applied Geology Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

The life of an earth scientist working in the oil industry is an interesting one. He has to make sense of a very large assortment of different data and based upon his interpretation of this data he has to tell his boss where and how many hydrocarbons can be found and how many he thinks can be recovered. Then, based on his interpretations, his boss may decide that the company will invest huge amounts of money and manpower over a period of several years to see if our earth scientist was actually right. Sometimes he indeed is, and everybody is happy. The interesting part is that the subject of all this activity, the underground hydrocarbon field, remains completely invisible. It is usually located at a great depth and virtually inaccessible for humans. We can make images of it using seismics, but these have a resolution in the order of tens of metres. We can drill holes in it and recover cores, but these are only 10 cm wide at best. The entire range of data in between the core and the seismics remains hidden, buried below kilometres of rock. Yet, to have data in these ranges is very important for our earth scientist, since this holds vital clues about the shape, size and characteristics of the reservoir. A sandstone is not just a sandstone, it is not a homogeneous collection of small rounded quartz pebbles nicely stacked on top of each other. It is a sandstone that was deposited in a certain location in a certain way under certain circumstances, resulting in a sediment body with a unique shape and size, and with specific characteristics. In short: every rock is unique, and therefore every hydrocarbon field is unique. There are however also certain characteristics that are more or less the same for certain types of sediment bodies. For instance, all sediments

Figure 1 The unconformity between the Weissliegend and the overlying Zechstein.

Figure 2 The 2010 excursion group in the Zechstein evaporites, the seal for the gas fields in the Rotliegendes.

that were deposited by aeolian processes (i.e. wind) in general show an excellent sorting of the different grain types and sizes. All sediments deposited by turbidity currents may show a certain order of deposits from bottom to top, the so-called Bouma sequence. To better understand these features and to be able to recognise them on well logs or in cores it is very useful to look at rocks that are exposed at the surface in order to gain a better understanding of the rocks that still lie deeply buried, out of reach. The same is true for structures that may influence a reservoir. Faults are sometimes sealing, but sometimes not. Fractures may enhance the production, but are not always homogeneously distributed. In order to understand these problems it is very useful to study rocks in the field. To help MSc students in Petroleum Engineering and Reservoir Geology do this, TU Delft organises a fieldtrip during which they study analogues for Dutch reservoir rocks that outcrop in Germany. It is a 7 day trip which starts just across the border near Ibbenbüren and continues all the way to Thuringia, before returning to Bad Bentheim via the Harz. During the trip, several quarries and natural outcrops are visited and rocks from the Carboniferous until the Cretaceous are studied. In all cases, the outcrops that are visited are representative for reservoir rocks in the Dutch subsurface, and the approach is always more or less the same: first, study the rocks you see in the outcrop, describe their lithology, morphology, and reservoir properties, and try to find a scenario for their deposition. Second, imagine you are looking at a reservoir: can you determine the type of reservoir as described in the previous step from the data that you have available (seismics, cores, well logs)? What are your concerns while drilling this reservoir? How do

Figure 3 The author standing in front of Bentheimer Sandstone, the reservoir rock of the Schoonebeek oil field just 50 km to the north of this outcrop.

you want to drill and how would you produce from this reservoir, are there any safety concerns you need to take into account? At every stop, there is a number of questions the students, working in groups, have to answer to guide them through this process. Working in groups has an added benefit. In the MSc program of Petroleum Engineering, students in general come from a wide range of backgrounds, some with a BSc in Earth Sciences, but others may have a background in chemistry or engineering, while others bring several years of experience in the oil industry. But all with the common goal to work as a reservoir engineer or reservoir geologist. This group work encourages discussions and an understanding of each other's background and characteristics. To illustrate the excursion, the photos in this article show some of the outcrops visited.

Figure 4 At work in the Cretaceous limestones near Brochterbeck, an analogue for the limestone reservoirs of the North Sea, like Ekofisk.

Maasvlakte 2 project: reclaiming future land with historical sand

Ir. Fedor Meulenkamp *(Senior Engineering Geologist, Royal Boskalis Westminster nv, Papendrecht, the Netherlands)*

Introduction

The Maasvlakte 2 (MV2) project is a large-scale land reclamation project in the Netherlands, extending the Port of Rotterdam seawards with about 2,000 ha. For this land reclamation in water depths up to about 18 m, approximately 200 million m^3 of sand will be dredged by contractor PUMA (*Projectorganisatie Uitbreiding Maasvlakte*, a joint venture between dredging companies Boskalis and Van Oord) from one of the designated offshore borrow areas. These borrow areas are situated approximately 8 nautical miles offshore from the project site. Each day, several trailing suction hopper dredgers (TSHD) dredge sand from these areas with original seabed levels of about 20 m below MSL (Mean Sea Level) to reclaim new land at the project site by dumping, *rainbowing*, and pumping ashore. Before the project started, 3D models of the soil characteristics in the borrow areas were created based on extensive desk studies and additional soil investigations. These soil models are used in the present construction phase to optimise the quality control of the sand and to ensure that the specific sand deposits are used for the right locations in the construction of MV 2.

Figure 1 Top view of the MV 2 project under construction. The coloured areas are being reclaimed with different types of sand. Red arrow indicates location of TSHD Oranje *as visited by the author on 30 July 2010 (see Figure 2).*

Figure 2 TSHD Oranje, *pumping sand (deposited between 100,000-300,000 yrs ago) ashore via a floating line, thereby reclaiming the outer MV 2 area from sea.*

Local geology

Local geology at the project site and the borrow areas can be subdivided into Pleistocene deposits with main formations of Kreftenheye and Kedichem, and Holocene deposits with the Bligh Bank, Elbow, and Banjaard as geological formations. Especially the Kreftenheye Formation consists of medium coarse to coarse subangular fluvial sands deposited in an era in which the sea level was approximately 150 m lower than today (Weichselien glacial period). These sands were deposited by the rivers Rhine and Meuse and are dominated by quartz and volcanic minerals from the Eifel region (located in south-east Germany). More to the present coastline, clay layers are deposited during the Late Weichselien (approximately 11,700 years ago) in a time that the sea level started to rise and the icecap retreated. After this period the sea level rose quickly towards the present coastline and an environment was created similar to the Wadden Sea as we know it today. Holocene formations of the Bligh Bank (Subatlantic period, 2,500 years ago to present), Banjaard (Boreal/Atlantic period, 9,000-5,000 years ago), Buitenbanken (Preboreal period, 10,000-9,000 years ago) and Elbow (Boreal period) were deposited in this interglacial period creating the upper few metres below seabed. These deposits generally consist of reworked marine and fluvial sands and are much finer in grain size than the Pleistocene sands of the Kreftenheye Formation. Near the coast, Preboreal and Boreal clay layers of the Elbow Formation are found (up to 8 m thick), locally eroded by ancient gullies and filled with Banjaard and Bligh Bank deposits of fine sands.

3D soil modelling of borrow area and project site

In 2006 and 2007, soil models of the soil conditions in the borrow areas and beneath the project site up to depths of -47 m MSL were prepared. For these models, extensive desk studies were performed on existing data like geophysical data, geological maps, and borehole data. Further, in 2006 and 2007, additional soil investigation campaigns consisting of boreholes and cone penetration tests (CPT's) were done (see Figure 3) to supplement the existing data and to fill in blank spots. All soil information (e.g. individual layer thicknesses and soil properties like d_{50}) from more than 950 boreholes and CPT's, 9,500 sieve analyses, and geophysical data from TNO-NITG was interpolated into 3D models, see Figure 4. The models formed the basis of geotechnical engineering for the MV2 project, planning and productions of the dredgers, and quality control of the reclaimed sands. The historic

Figure 3 Drilling in the borrow area during the 2007 soil investigation.

value of these sand deposits however was not included in these 3D models. During dredging in the borrow areas as well as during unloading at the reclamation site, all kinds of ancient relics related to the living environment of the glorious mammoth, the Neolithic, and the present became exposed.

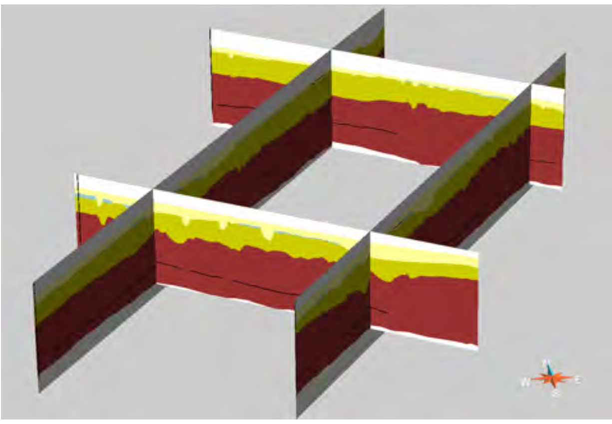

Figure 4 Visualisation of the 3D soil model of MV2 (brown layer represents the Kreftenheye Formation).

Dredging historical sands

The 200 million m³ of sand dredged from the designated offshore borrow area is partly of Pleistocene age, a period in which mammoths lived in this area. In those times, the mammoth walked around in tundra- or steppe-like environments at the locations of the present borrow areas. These areas were dry land as the sea level was approximately 150 m lower than today. At the end of the Pleistocene, when the climate became warmer and the sea level rose, the mammoth became extinct. Bones of these extinct mammoths were covered by metres of Pleistocene and Holocene sediments. While removing these sediments by dredging, we are actually 'digging into the past'. In some parts of the borrow area the seabed level is already deepened up to -40 m MSL. Combining knowledge of dredging location and depth with soil characteristics, an estimate of the period in which particular sand is dredged can be made. For example, shell content in combination with the grain size distribution might reveal what kind of depositional environment was present in a particular time period. Different depositional environments such as fluvial, marine, or eolian are related to certain geological periods. Furthermore, Kedichem sand (deposited between 0.8 and 1.6 million years ago) is finer grained than Kreftenheye sand. Shallow silt and clay layers or lenses indicate Holocene deposits. Figure 5 shows a representative borehole from the 2007 soil investigation campaign. In this borehole, Holocene deposits (roughly younger than 10,000 years) can be seen up to ca. -21 m MSL. Beneath these Holocene deposits we see the Pleistocene sands. Based on shell content it is estimated that from circa -21 m MSL to -34 m MSL the sands were deposited during the late Pleistocene (more precisely the Eemian period from 128,000 to 116,000 years ago). It can be seen from the log that below -33.8 m MSL the shell content disappears and a gravel fraction is introduced. These deposits are related to the late Saalian period (around 128,000 years ago). It is at these depths in

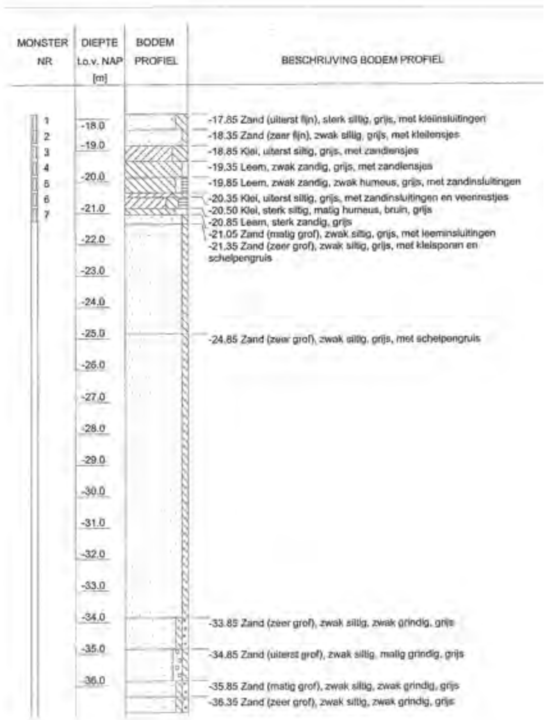

Figure 5 Borehole log representing the different sediments and depths. First used for engineering purposes, later on to determine the historic time period.

the borrow area where for example TSHD *Oranje* was dredging historical sand on 30 July 2010 and, just one hour later, pumping it ashore into the newly created MV2 (see Figure 2). Collected bones and other relics found during dredging operations are collected, analysed and stored. The most interesting bones are exposed in FutureLand, the MV2 information centre of the Port of Rotterdam.

Editorial note: this article has appeared in its original form in the biannual bulletin of the European Marine Sand and Gravel Group (EMSAGG), Issue 21, December 2010.

Geo-Frontiers 2011

Dr.ir. Leon van Paassen *(Assistant Professor Geo-Engineering, Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geosciences, Delft University of Technology)*

Dallas, the city that is known for the oil barons in the eponymous TV series and as the place where JFK got shot, was the host city of the Geo-Frontiers 2011 conference which was held from 13-16 March 2011. The conference was co-organised by the Geo-Institute of the American Society of Civil Engineers (ASCE), Industrial Fabrics Association International (IFAI), North American Geosynthetics Society (NAGS), and Geosynthetics Materials Association (GMA). The conference proceedings include 496 technical papers which cover a wide range of research and practical applications in geotechnical engineering, including:

- Foundations and ground improvement
- Geo-environmental engineering
- Geo-hazards (earthquakes, landslides, erosion)
- Geosynthetics
- Geotechnical testing and site characterisation
- Slopes, embankments, and earth-retaining structures
- Soil-rock mechanics and modelling
- Transportation materials and pavements

The papers were presented in 86 technical sessions over a 3 day period and published in an ASCE Geotechnical Special Publication (GSP) *Advances in Geotechnical Engineering* after peer-review by at least two reviewers and the editors, Jie Han and Daniel Alzamora.

The conference also included several special events. Four prestigious lectures were presented at the conference including the Terzaghi Lecture by Kenneth Stokoe, who gave an outstanding lecture on how to use seismic measurements in geotechnical engineering; the Peck lecture by Antonio Bobet who presented the seismic design of underground structures and the lessons learned from failure of the Dakai Station, which got severely damaged during the Kobe earthquake in 1995; the H. Bolton Seed Lecture by Norbert Morgenstern, who addressed the large geotechnical engineering challenges involved in mining the Alberta oil sands; and the Mercer lecture by Junichi Koseki on how to use geosynthetics to improve the seismic performance of earth structures. There was a student paper competition tavern which was held in a poster session. Students also participated in the Geo-Challenge in which they designed, built, and tested a scale model of a mechanically stabilised earth wall. Several short courses were offered by international experts in their fields at the beginning of the conference.

Besides the technical program there was sufficient time to exchange and discuss experiences, either on the huge exhibition area or during several formal and informal dinners. In my particular interest were three sessions on bio-mediated ground improvement methods. The topic which seemed to get limited attention a few years ago, now involved 18 groups worldwide which are actively developing these new technologies. The *biocommunity* in geotechnical engineering seems to get some critical mass.

Internship Cofra B.V.

Jan-Willem Vink *(Master student, Geo-Engineering Section, Department of Geotechnology, Faculty of Civil Engineering and Geo-sciences, Delft University of Technology)*

Introduction

After finishing my Bachelor in Civil Engineering in 2009, I started my Master in Geotechnical Engineering at Delft University of Technology. In the first year of my Master I attended several courses and based on what I learned it was the obvious choice for me to start an internship. I figured that during an internship I could check whether I would be able to put my theoretical knowledge into practice, and also, whether I could meet my own expectations and those of the company. Like most other students, I found it hard to make a decision between a contractor or a consultancy. An internship offer by contractor Cofra B.V. caught my attention because it had a diverse job description. They offered an opportunity to work at their geotechnical department in Amsterdam and to go abroad to Kuwait (if possible) to perform site supervision and give geotechnical support at a project. This way, I could experience some engineering work with a contractor and I'd have the opportunity to work on an actual project abroad. I thought that getting to experience both might facilitate future career choices. Cofra B.V. is part of the internationally operating dredging company Royal Boskalis Westminster nv. Cofra's main focus is on ground improvement solutions. Cofra is European market leader when it comes to the installation of vertical drainage and is continuously working on the development of new ground improvement techniques.

Part 1: Amsterdam

The first part of my internship I spent at the geotechnical department of Cofra in Amsterdam. I started on 30 November 2010. After being introduced to the company I was all set to work in the office of Jeroen Dijkstra (an engineering geologist who also studied in Delft). My first project was to interpret the logged installation data obtained during the installation of *BeauDrain-S* (vacuum drain) at the International Cruise Terminal in Singapore. Vacuum drainage is needed in this project for the fast consolidation of the softer layers and to limit the horizontal movement of the subsoil by limiting the surcharge. Vacuum drainage can however not be applied in the harder sandy layers which behave drained. This would have a highly negative influence on the functioning of the vacuum drain; it would suck in too much water and would lead to no vacuum pressure inside the drain. The penetration force of every drain was measured at 25 cm intervals during installation. Using this data and the location of the drains, an overview of the subsoil was created as shown in Figure 1. Drain distance at this project was 1 m in square, giving an indication of the huge amount of data available about soil resistance (which includes point resistance as well as shaft friction). On the profiles that I prepared I had to draw the tube section of the *BeauDrain-S* drain and see whether the installation depth was correct. Before installation commenced, a detailed site investigation was performed and the spatial thickness of reclamation sands was estimated. With this method it can be checked whether installation depth of the polyethylene hoses was correct as well as the total depth of the drain. These polyethylene hoses were designed to penetrate and be applied in the reclamations sands. The predetermined length of the prefabricated polyethylene hose is connected to the drain before installation. It turned out to be designed very well.

My second project was to assist with a tender for the 2012 Olympic Games site in Sochi, Russia, making settlement calculations for road embankments combined with drains. Using Russian cross sections of the subsoil that were interpreted from local drilling cores, I made some general profiles of the subsoil which would be used for settlement calculations. Besides the profiles I also determined soil parameters with the help of Russian speaking colleagues at Hydronamic, the engineering department of Boskalis (The geotechnical department of Cofra works closely together with Hydronamic), and helped with settlement calculations. Consequently, total drain length of the project was determined using the drawings and calculations. At a later stage more

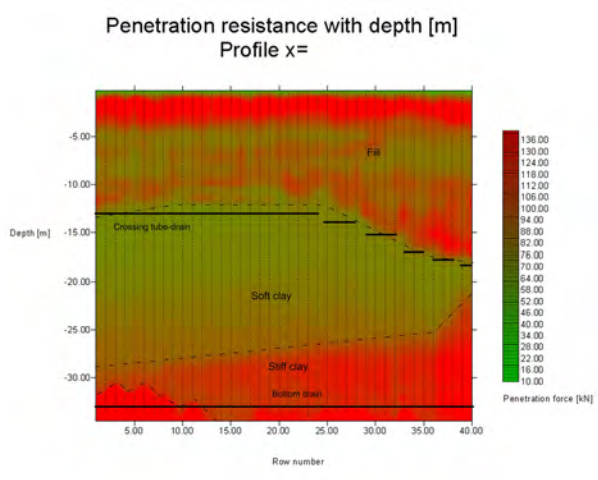

Figure 1 Cross section.

detailed calculations will be required when more soil data becomes available. In the meantime, Cofra has become contractor of the Sochi project and has recently started installing the drains.

The third and biggest project that I worked on in the office was the CDC (*Cofra Dynamic Compaction*) project in Kuwait. I tried to analyse CDC compaction data as more knowledge of the compaction was desirable, using the before and after CPT data after every pass. One of the main goals of this research was to define and classify the compactable and non-compactable soil(s), so that for future purposes a more clear assessment of the compaction process can be made beforehand regarding the specific deposits found at the site being sabkha deposits. Unfortunately no clear correlation could be made.

Besides the projects, I also attended an internal consultancy meeting. Here I noticed that informing clients about risks and limitations that come with settlement calculations and design is also important. With the client wanting to optimise as much as possible, the risks of certain assumptions taken in the design should be known to him. I learned that it is better not to reel towards the client too much, because in the end the engineer is responsible. Since that will be me, it was very insightful.

Part 2: Kuwait

On 25 January 2011 I left for Kuwait. Cofra's task at the two projects that I worked on in Kuwait was to make sure the bearing capacity of the subsoil would be increased in order to keep settlements of future buildings to a minimum. Different compaction techniques were performed on site: DC (dynamic compaction), DR (dynamic replacement), and CDC (Cofra Dynamic Compaction, Cofra's own developed Rapid Impact Compaction method).

Personnel on site consisted of one geotechnical engineer (Dries Stork), two project managers (Peter Piek and Manfred Chang), and two mechanics. When I started, the project was in its end phase and due to the tight working schedule a lot of areas still had to be delivered to and approved by the client. Cofra operated at the site with three CDC machines at the same time; two Caterpillar 385 machines and one O&K RH40 machine. A CDC unit consists of a crane and a hydraulic hammer with a 16 ton drop weight, attached to the boom (see Figure 2). The maximum drop height of the weight is 1.2 m, performing about 40 to 50 blows at every grid point. Drop height, number of blows, foot size, and number of passes can be adjusted, all depending on the soil type, the required level of compaction, and compaction depth. A GPS logger on the crane registers data for every grid point where the machine compacts, registering amongst others coordinates, date, time, total settlement, settlement per blow, and the number of blows. This data can be used to gain insight in the level of compaction of the soil and can partly be correlated with CPT's (which were assessed in the past). This data had to be processed every morning. Usually, a CPT of the area would be made to confirm the data and determine whether another pass with the CDC machine was required.

Figure 2 CDC machine (O&K RH40) in action.

Based on all data and from experience one could determine the energy that was needed to compact the ground; adjusting the drop height and the number of blows of the hydraulic hammer. The continuous works required a good planning and schedule to keep things going. The machines had to work day and night shifts and were operated by foreign operators. This sometimes created difficult but often hilarious situations in which cultural and language differences met up. It was a difficult situation to be dropped into as a newbie, but very educational nevertheless. It was all about finding balance; keeping the machines working, doing what is best for the soil, cooperating with other contractors to get the job done in time, interpreting CPT data correctly, determining the number of passes, number of blows, drop height, and foot size. Next to the above, my task was also to check the helpers that were responsible for changing the polypancos. These are hard rubber disks that are positioned between the hammer and the changeable foot. This disk functions as a protector of the steel components of hammer and foot, thereby providing enough energy to the hammer (and soil) without damaging the internal components (see Figure 4). The expensive polypancos had to be changed approximately every hour before they would get too hot and break. They needed to be cooled for at least 48 hours after every use. After a while I moved to another CDC project in Kuwait, which was still in the start-up phase and had (only) one machine running (an O&K RH40). This resulted in some more rest for me and with the experience gained so far it was more or less a piece of cake. However, I did realise that this was also an important phase because at this stage it was necessary to start off good.

Unfortunately, the survey system (CMS) on the RH40 machine at the site had some problems when I arrived. Dries Stork and Manfred Chang had put a lot of effort in refitting parts and performing all sorts of other measures. Dries Stork and I somehow managed to refit the motherboard component of the computer using the spare parts that the surveyor had left behind, carefully marking all wires and components in the process. It is clear that a lot more comes into play when working in the field as a geotechnical engineer. Some improvising, logic thinking, and an active approach are the keys to success.

Figure 3 Hammer and passed grid.

Figure 4 Polypanco.

During my work abroad there was also time for some leisure. The 'quest' for finding a camel kept the crew busy for a while, but eventually a colleague found a huge camel farm. Enthusiastically he arranged a trip and managed to take some pictures of himself on a camel, so he could show his kids at home. Later on it turned out to be a dromedary, same difference!

The beautiful and luxurious shopping malls were the 'place to be' in Kuwait, especially on Thursday night and Fridays. These are the places were Kuwaiti meet. There is no alcohol allowed in the country so the shopping malls can be considered as the place to go out instead of bars and pubs. The cultural differences with the Western world, along with the similarities and their wealth, were striking.

Reflection

This internship has strengthened my ambition to work for a contractor, especially the work at the geotechnical department which is a practical engineering/consultancy position within a contractor. The different activities sparked a lot of enthusiasm in me; it turned out that I was able to contribute. Spending so much time during the Masters on a very specific matter can come into use in practice. Of course the lack of experience is an important downside when working for a contractor. But with good common sense, some theoretical background, and support from colleagues you can still achieve a lot. In conclusion I can really say I got what they promised me: diversity.

The good experience, the enthusiastic people at Cofra, and my own interest made me decide to start graduating at Cofra, on the subject of CDC.

A.P. van den Berg successfully tests its latest deep water CPT technology for geotechnical site investigation

Advertorial A.P. van den Berg (Heerenveen, the Netherlands)

Introduction

Ever since the 1980's, engineers of A.P. van den Berg have been making CPT technology available for seabed soil investigations for a.o. civil works, installations for oil- and gas exploitation, and construction of wind farms. Every day, A.P. van den Berg works on the development of new applications that enhance the reliability of CPT's. As a result, the water depth for CPT offshore activities has increased to up to 4,000 m. Also, CPT depths have increased during the last years. A.P. van den Berg is a global player, having earned its spurs in the world of CPT's, which is once again proven by the new developments and the cooperation with highly regarded geotechnical research institutes. A.P. van den Berg equipment is widely used by many geotechnical survey companies that care strongly about service and reliability. By applying A.P. van den Berg technology these companies achieve groundbreaking and unique results.

Deep Water WISON-APB system

Offshore soil investigation extends to ever-increasing depths, where the conditions place greater demands on the technology to be used. The *Deep Water WISON-APB* system was specially designed for these conditions. In the last quarter of 2011, one of the customers has taken the system into use for the first time. On 10 February 2011, A.P. van den Berg successfully tested the system on its premises. Pre-testing is a standard procedure, in this way potential problems can be resolved early on in the process. The situation at sea, where the equipment sometimes descends up to 3,000 m depth, is simulated in a small scale test. For this test A.P. van den Berg deployed heavy artillery. A crane lowered the device from 25 m high into a 14 m long pipe, after which the equipment was tested for its performance. The test results were very positive, a number of final tests will be performed after which the equipment is ready for use.

The *Deep Water WISON-APB* system is designed for use in a drill pipe of a geotechnical survey vessel and offers the customer many advantages. For example, the multipurpose tool makes it possible to change between CPT and core sampling in just minutes. An electric constant tensioning winch with a reinforced umbilical cable allows fast downhole operation to 3,000 m depth. Of course attention has been paid to high speed data communication by means of fibre optics. A.P. van den Berg feels very strongly about ease of use: easy operation of the *Deep Water WISON-APB* system is available from the control room by means of two touch screens. Finally, A.P. van den Berg cares about the environment. The *Deep Water WISON-APB* system is fully enclosed to prevent spills and does not affect the deep sea environment.

More information can be found at www.apvandenberg.com.

IAEG 2010 Congress: twenty years since Amsterdam and at the opposite end of the world

Michiel Maurenbrecher MSc CEng

When the Netherlands hosted the 6th IAEG Congress in 1990, little did I know that I would attend each subsequent congress, and that I would travel almost a complete great circle of 180° to get to the 11th Congress twenty years later! Well, almost 180°. If one drives a diameter through the Earth from Amsterdam it will not appear in Auckland itself but in the Auckland Islands. Another significant fact about the IAEG congresses is that most have been held in major port cities. Since Amsterdam they were, in chronological order, in Lisbon, Vancouver, and Durban, the exception being the last, in Nottingham. The break with harbours should not have been as the venue was originally supposed to be London. What is significant about these congresses is the almost total lack of themes devoted to coastal and offshore engineering geology. As if geology seems to stop at the beach front! The Nottingham congress did have one or two papers on coastal cliff stability under a theme *Engineering geology of slopes in urban setting*. The last two congresses (I can say this now) have much in common: both in island nations and subject to lots of rain so both look very green. During the experience of taking the airport bus into Auckland one can be forgiven to thinking one had landed at a major British provincial airport. The suburban Auckland architecture resembles much of that of Nottingham. The weather in New Zealand in early spring was cooler at the tail end of the southern hemisphere winter than the early autumn in England at the tail end of the northern hemisphere summer.

The title of the 11th Congress was *Geologically Active*. The first morning, back in my hotel room from breakfast, I switched on the TV intend on getting a radio station with some music but an announcer of the TV channel that appeared was saying something about 'breaking news'. '*The minister will shortly make a statement about the earthquake at Darfield near Christchurch*'. 'Welcome to New Zealand' I thought. A government spokesperson appeared to say the government will do all it can to help the earthquake victims of Christchurch. This was followed by pictures of damaged houses, ruptures, and liquefaction (sand eruptions). I have a cousin in Christchurch (Alex Maurenbrecher), he seemed to be unaffected. That is, most of the Maurenbrechers I visited after the congress including his brother and sister did not mention him being affected. Charlie Price however, who I saw at an earlier meeting celebrating 100 years of Engineering Geology at Imperial College in July 2010, was affected. He had to move out of the vicarage (his wife is a vicar) as it was listed as unsafe. So I did not see much of him at the congress except for the last day. He was trying to get a return flight to Christchurch to get there as soon as possible with regard to moving to a new home. At the end of the congress, Professor Paul Marinos gave a presentation on the damage and features of the earthquake having forsaken a day or two of the congress to make a field visit to Christchurch. Since the congress, Christchurch experienced a second strong earthquake eight months later. This was soon eclipsed by the devastating earthquake in Japan. That is why I mention offshore and coastal zones. The last earthquake coming near to that of Japan must have been the 1964 Alaska earthquake near Anchorage (9.2 on the Richter scale). This earthquake caused extensive tsunami damage and in its aftermath there are still papers being written about it. Very fresh in everyone's mind is the Sumatra earthquake just over a year previously, which also occurred offshore. Plenty of papers at the congress were about earthquakes, especially the 2008 Wenchuan earthquake in China. Most IAEG congresses appeared to be dominated by slope stability (some coastal) and materials testing. I attended the congress for the first time on a personal basis having now truly retired at 65 years old. I presented a paper on engineering geology and wine ('Wines and winelands: Engineering geology fieldwork in Hispania Cisterior', P.M. Maurenbrecher, D.M. Ngan-Tillard & B. Aline Concha Dimas), something one would expect from a retired engineering geology lecturer. At Nottingham I promised Fred Baynes, who represented the interests of New Zealand, that I would come if there would be a theme on wine. The paper still had a Delft connection, however, as our engineering geology fieldwork for the last twenty years has been in Priorat, one of the two highest-regarded wine producing regions in Spain carrying the special *DOCa* qualification (*Denominación de Origen Calificada*). One could not help doing fieldwork and not taking an interest in the wine. After a bit of lobbying in Nottingham, the New Zealand congress organisers had little option but to include a *terroir* theme. After all, New Zealand is producing excellent wines on both Islands. The new millennium celebration for me in Holland started 12 hours earlier in order to toast my nearest Maurenbrecher relatives. I managed to obtain a bottle of *méthode champenoise* from New Zealand. So whilst the rest of the Netherlands and the world was preparing for celebration, a bottle was opened at the Groot

Hertoginnelaan with my Hartevelt in-laws as they also have family there. The bottle of Lindauer did New Zealand proud: the cork reached the high ceiling without any problem and the wine indeed sparkled. It is not the intention to write about the 11th Congress in this Newsletter as so that one can expect one or two more instalments in the coming year. There were a few highlights that have stuck in my mind: only one presentation about Holland from Helmut Bock from just across the border from Enschede and, as he put it, a short bicycle ride away from Niek Rengers. He gave a historical coupling presentation between Niek and Fred Baynes up to the end of the congress when he handed over to Carlos Delgado from Spain. In my time lag stupor whilst reviewing my presentation he appeared in the computer room and we started a conversation on Spanish wine. We were trying to remember a few wine quality designations used for Spanish wines and finally managed to sort this out: *vino de mesa*, *Crianza*, and *Reserva*. I think he too was suffering from time lag and endless meetings. Helmut's presentation was about the Bentheimer Sandstone used in the construction of the Royal Palace in Amsterdam. Blocks of this sandstone were, back in 17th century, earmarked to be used in a gate archway for Batavia and transported there as ballast on a VOC ship coincidentally named *Batavia*. The VOC ship foundered off the west coast of Australia. The wreck was located three centuries later by divers who then salvaged the blocks. The blocks were re-assembled and now form the gate to a gallery at the Western Australian Museum - Maritime in Fremantle, the birthplace of Fred Baynes. I should note that there was a paper co-authored by Cees van Westen about mapping landsliding in the Nilgiri Hills of southern India. One significant ITC connection was the winner of the poster competition by Tahir Topal. The award came at the very end of the conference so that his poster was taken down by the time I had a chance to photograph it. He sent me a copy since but because I still work with antiquated Microsoft software I am unable to download it. Tahir is professor of Engineering Geology at Middle East University of Technology, Ankara (when the TU Delft engineering geology students made their study trip to Turkey in 2002 we stayed two nights at MET and visited Tahir's department).

The next instalment will be about the social functions (including wine tasting Auckland-style following the milestone first in an IAEG congress of the *terroir* theme) and field trips (both IAEG and my own trip through the land of faults all the way to the north of South Island visiting family on the way as well as sampling local wine).

The use of dredged sludge as a fill in the Osthafen (Bremerhaven, Germany)

***A.H. Nooy van der Kolff** (Royal Boskalis Westminster nv), **D. Lesemann** (PHW mbH), & **K. Petereit** (PHW mbH)*

Abstract

Expansion of Bremerhaven's Osthafen, in particular its car handling facilities, required reclamation of approximately 6.1 hectares of the existing harbour basin and deepening of the remainder. Because federal regulations restrict dumping of the dredged, often contaminated sludge, it was decided to use the dredged materials as a permanent fill. These materials consisted of soft organic silts and were to be used in the lower part of the fill. The dredging and reclamation operations as planned comprised placement of a sheet pile wall closing off part of the harbour basin, dredging of the remainder of the basin, depositing of the dredged mud behind the sheet pile wall, placement of a geotextile and a sand capping on top of the sludge, and installation of a system of vertical drains to accelerate the consolidation process of the mud. After deposition of the sludge and installation of the geomembrane on top of the sludge stability problems occurred when spraying the initial lifts of the sand capping. The problems were attributed to the very low shear strength of the mud and the generation of gas as a result of the organic nature of the sludge. In order to overcome the problems it was decided to install a vacuum drainage system to increase the shear strength of the sludge. After 3-6 months pumping the first lifts of sand could be placed without stability problems. It is concluded that dredged sludge can be used as construction material provided that its specific properties are taken into account. The use of a vacuum consolidation system to increase the shear strength of the sludge has played a key role during construction.

Introduction

Up to the end of the 20th century, contaminated dredged material removed from harbours and waterways was usually disposed of at sea or used to raise and/or fertilise land (PIANC, 2009). As a result of growing environmental awareness, new methods had to be developed to handle these materials. Two main approaches can be distinguished: confined storage and treatment of the contaminated materials. Both methods are expensive and need special facilities like large confined storage facilities (CDF's) or treatment plants (drying plants, soil washing plants, sand separation plants, and filter presses for dewatering). They are often combined: stored material may be treated at a later stage, while the remaining product after treatment may need to be stored. This paper describes another option: the use of dredged sludge as a permanent construction material in a confined environment. It is illustrated by the case history of the Osthafen project in Bremerhaven, Germany.

Osthafen project

Annually, more than 1.3 million cars pass through Bremerhaven, making it one of the largest car importing and exporting ports in Europe. Expansion of Bremerhaven's Osthafen, and in particular its car handling facilities, required reclamation of approximately 6.1 hectares of the northeastern part of the existing harbour basin and the construction of an additional berth for transhipment purposes. In addition, to accommodate deep sea car carriers (DSC's) the remainder of the harbour needed to be dredged to a depth of NN -9.11 m. Figure 1 presents an overview depicting the dredging and reclamation areas. The bottom at the proposed reclamation site ranged between NN -6.60 m and NN

Figure 1 Overview of project site depicting dredging area (grey) and fill area (yellow).

-2.70 m. The water level in the harbour was approximately NN +1.23 m. Results of site investigations indicated that the seabed sediments consisted of a soft to very soft sludge (thickness varying between 2 m and 6 m) covering a layer of soft organic clay (thickness approximately 5 m). These cohesive deposits cover strata of loose sand. Because current federal regulations in Germany restrict offshore dumping of dredged, often contaminated sludge, it was decided to use the dredged materials as fill. These materials consisted of approx. 170,000 m^3 of soft organic silts with poor engineering properties and were to be used in the lower part of the fill. The construction activities and dredging and reclamation operations envisaged:

- Placement of an anchored sheet pile wall (620 m) closing off part of the harbour basin.
- Dredging of the remainder of the basin to a depth of NN -9.11 m.
- Deposition of the dredged mud behind the sheet pile

wall to a maximum elevation of NN +0.80 m (average thickness of the mud 4.5 m, maximum thickness near the sheet pile wall 7.0 m).

- Placement of a geotextile (sand mat) on top of the sludge.
- Careful deposition of a number of thin layers of sand on top of the sand mats using a spreader pontoon followed at a later stage by thicker lifts of sand by discharging through a conventional pipeline (total volume approximately 260,000 m^3).
- Installation of a system of vertical drains to accelerate the consolidation process of the mud.

The specifications for the dredging operations prescribed a method that would limit the increase of the water content to a maximum value of 10% of its original value. It was believed that this would reduce the strength loss of the soft, partly organic deposits expected as a result of dredging, transporting, and placing. The sand fill had to be dredged at the outer Weser and transported to site. Client for the project was Bremenports GmbH & Co. KG, the consultant responsible for the design of the reclamation area and dredging was PHW mbH (*Planer für Hafenflächenrecycling und Wasserbau mbH*). Dredging and reclamation works were carried out by Heinrich Hirdes GmbH, a subsidiary of the Royal Boskalis Westminster Group, in close consultation with Bremenports and PHW.

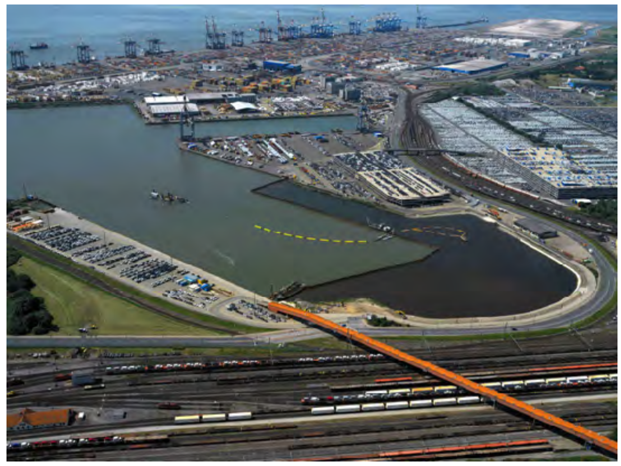

Figure 2 Overview of project during dredging operations (dredging pontoon, floating pipeline, and spreader pontoon).

Dredging and initial reclamation operations

Prior to dredging in the Osthafen, a sheet pile wall anchored by raking piles was installed to contain the fill and provide a suitable quay wall for the vessels. Dredging was carried out with an environmental grab attached to a hydraulic crane. This system was used to ensure a minimum disturbance and dilution of the sludge and organic clays during excavation and subsequent lifting of the grab through the water column. The dredged materials were dumped into a feeder of a specially developed high density solids (positive displacement) pump. Unlike the conventional centrifugal dredging pumps, the positive displacement pump can handle the dredged materials without adding any process water and, thus, without increasing the water content of the sludge. The sludge was pumped through a floating pipeline to a pontoon and placed on the bottom of the reclamation area by a diffuser positioned just above the seabed. The vertical position of the diffuser was regulated on the pontoon, while the horizontal position was controlled by moving the pontoon using its winches, wires, and anchors. Figure 2 gives an overview of the project area during dredging operations. Regular monitoring of the water content both in situ and after placement in the reclamation area indicated an increase of 6-10%, well below the specified maximum. As a result of dredging, transport, and deposition, the original undrained shear strength of the dredged material reduced from approximately 2.0-5.0 kPa to 0.1-0.5 kPa. Once the sludge in the reclamation area had reached an elevation of NN +0.30 m, two layers of sand mats (*Terrafix B 301 G 31*) with a nominal tensile strength of 105 kN/m were installed on top to provide a separation between the sludge and the covering sand layers. In addition, the geotextile sand mats had to distribute local stress concentrations induced by uneven loading as a result of variations of the thickness of the sand capping. Sand was dredged in the outer Weser and transported to the Osthafen by a trailer suction hopper dredger. After arrival at the site the dredger was coupled to a floating pipeline that was connected to a spreader pontoon in order to carefully discharge the sand in thin layers (thickness of the first four layers was 0.1 m) on the sand mats. The water level in the reclamation area was raised as much as possible to provide sufficient draught for the spreader pontoon. After each lift bathymetric surveys were undertaken to determine the actual thickness of the sand layers.

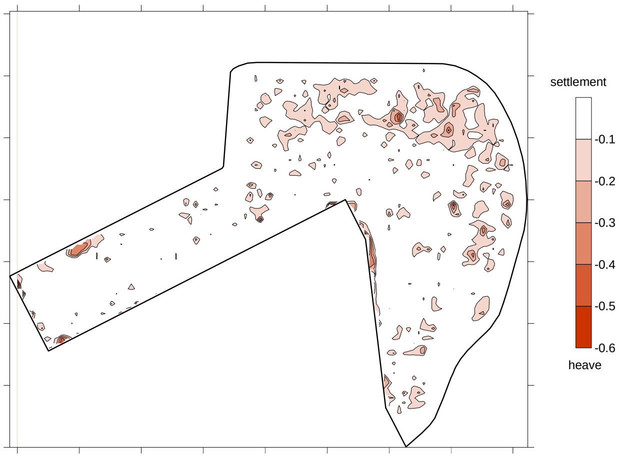

Figure 3 Result of a bathymetric survey of the fill surface after occurrence of deformations.

Surface deformations

Despite the careful execution of the spreading operations, serious stability problems occurred when the total thickness of the sand fill reached approximately 0.8 m. Filling had to be ceased as excessive deformations of the dredged sludge had created 'blisters' at the surface rising more than 0.5 m above the original level of the fill and occasionally unveiling the underlying sand mats. Figure 3 depicts the results of a bathymetric survey carried out immediately after the occurrence of these deformations. Investigations including field vane tests and laboratory tests on samples taken from the sludge were carried out by the University of Bremen and others to identify the most likely cause(s) of the stability problems. Although the causes were not fully understood, the problems were generally believed to be a result of a combination of the very low shear strength of the mud after dredging, transport, and placement (average value of 0.35 kPa instead of 0.50 kPa as assumed in the design calculations) and the generation of gas as a result of the organic nature of the sludge. Gas bubbles have been observed at various locations and are known to adversely affect the undrained shear strength, in particular at low effective stresses (Wheeler, 1988). In addition, as a result of the limited permeability of the geotextile gas accumulations under the sand mats reduced the load at these locations. As a consequence, the undrained shear strength of the sludge may

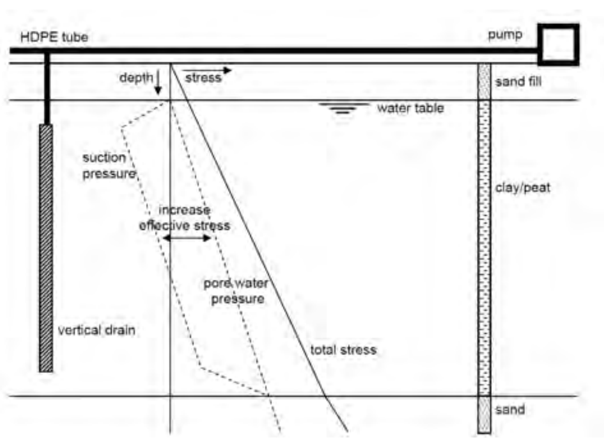

Figure 4 Principle of BeauDrain-S *system including diagram of total, effective and pore water stresses.*

have varied significantly from one place to another. In combination with a load that varied as well this may have caused the surface deformations. Sand sliding from the 'blisters' towards the depressions surrounding these structures have probably further aggravated the differential settlements. However, other more exotic causes like mud flows and liquefaction phenomena have also been suggested and can not be excluded.

Corrective measures

Whatever the cause of the excessive surface deformation might have been, it was clear that the undrained shear strength of the dredged sludge and the underlying soft clay had to be increased before the sand capping could be placed. Various options to reduce the excess pore water

Figure 5 BeauDrain-S *system after lowering of water table and connection of HDPE tubes.*

pressures were considered. After discussions with all parties involved (i.e. client, consulting engineer, and contractor) it was agreed that the most feasible option seemed to be to accelerate the consolidation process by installing a vacuum consolidation system before continuing filling operations. The vacuum consolidation system is based on the principle that by reducing the pore water pressures in the sludge mass an atmospheric pressure is mobilised that acts as an isotropic load on the sludge without the introduction of shear stresses. In this specific case, lowering the pore pressure in the sludge would also help releasing the dissolved methane gas. It was decided to use the so-called *BeauDrain-S* system of Cofra B.V., a subsidiary of the Royal Boskalis Westminster Group. This system consists of a number of prefabricated vertical drains each separately connected by HDPE tubes to special pumps, see Figure 4. The HDPE tubes extend to a certain depth below the sludge surface. This allows the top 1.0-1.5 m of the sludge to act as a sealing membrane between the soil mass with its low pore pressure and the atmospheric conditions. The special pumps are developed in the dredging industry and can separate and remove both fluids and gasses. The *BeauDrain-S* system was preferred above other systems using the same principle because it did not require access of heavy equipment to the area as the vertical drains could be installed over water and it did not need an expensive and vulnerable geomembrane. After installation of the vertical drains at a typical spacing of approximately 0.8 m by floating equipment, the water in the fill area was lowered as much as possible below the top of the sludge in order to gradually raise the vertical effective stress. Although at that moment the drains were not yet connected to the pumps it could be observed that they already started to drain the sludge as water was flowing from the open top end of the drains. Once the mud had gained sufficient strength to allow for access of personnel to the area, all drains were connected to vacuum pumps and pumping was started, see Figure 5. Measurements of the undrained shear strength by vane testing using a special calibrated vane indicated that after approximately 6 months it had increased from an average of 0.35 kPa to 1.50-2.50 kPa. In the following months it continued to rise to an average value of 5 kPa approximately 1.5 year after pumping had started. Only in the upper metre that was not drained by the *BeauDrain-S* system, the mud did not gain strength. After approximately 3 months filling was carefully resumed. To allow for visual control during these operations it was decided to initially import dry fill and use a light-weight dozer with wide tracks to spread the fill. During these operations the deformations of the surface and the development of the undrained shear strength of the mud were closely monitored. Most of the deformations observed were attributed to the low undrained shear strength of the upper metre and were considered to be of minor importance. During dry filling, no significant instabilities were encountered. Once the dry sand fill had reached an average thickness of 1.5 m, filling was continued by discharging through various light-weight portable discharge pipes connected to a conventional pipeline that was located on the original shore just

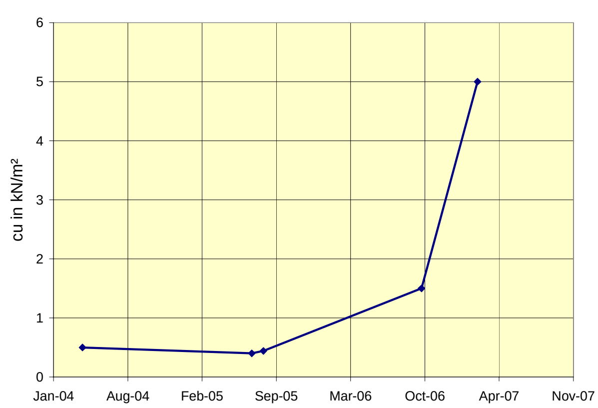

Figure 6 Increase of average undrained shear strength with time.

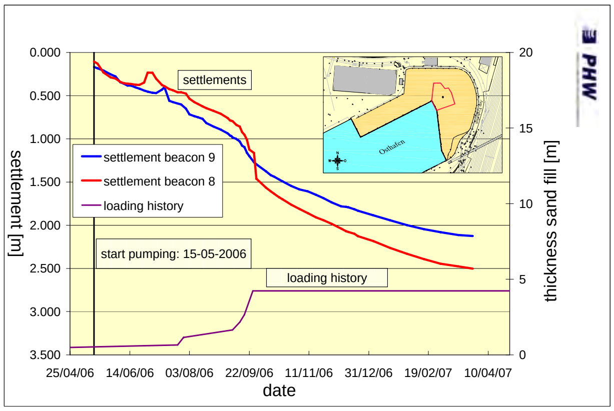

Figure 7 Typical time-settlement records observed after start of the pumps.

outside the reclamation area. Discharging was carried out through 4 to 6 discharge pipes at a time at reduced concentrations of the sand-water mixture. This resulted in gentle slopes of less than 1(v):20(h), preventing further surface deformations. Moreover, the presence of multiple discharge points enabled the dredger to maintain its discharge rate at an economic level.

Monitoring of undrained shear strength and settlements

Although monitoring of the undrained shear strength and settlements was undertaken from the beginning of the project, it was intensified after occurrence of the surface deformations. As the undrained shear strength of the mud was extremely low, the field vane had to be calibrated by comparing the results with in situ dynamic viscosities that were directly measured with a device called Nautisonde (originally developed to determine nautical depths). Figure 6 presents the average value of the results of the field vane tests, clearly showing an increase with time. After the start of the pumps in the period between May and June 2005 it took 1-2 months before an increase of the undrained shear strength was noticed. This is believed to be a result of the large volumes of water that had to be extracted from the sludge in the initial, underconsolidated phase before any effective stress of the sludge could develop. During this period the volumes of water to be removed exceeded the maximum capacity of the *BeauDrain* system. As a result of the use of the spreader pontoon and its anchor wires that did not allow the presence of settlement beacons installed on top of the sand mats, no continuous series of settlement measurements could be taken. The settlement records depicted in Figure 7 have been reconstituted from bathymetric measurements, readings from settlement beacons, and, at a later stage, settlement plates. During the first 1-2 months, settlements developed rapidly without resulting in an increase of the undrained shear strength (see also Figure 6). The placement of the first lifts of dry sand resulted in horizontal deformations of the upper metre of the sludge that could not be drained by the *BeauDrain-S* system. This caused a 'wave' of sludge (or gas) in front of the sand fill to pass underneath the sand mats. These deformations lifted the settlement beacons approximately 0.2 m. Increasing the thickness of the fill from approximately 1.5 m to 4.0 m by hydraulically placed sand fill did not lead to further measurable horizontal deformations. The pumps were stopped after most of the primary settlement had occurred.

Conclusions and recommendations

Experience gained on the Osthaven project indicates that dredged sludge can be used successfully as a construction material, provided that the problems associated with its typical properties are addressed adequately. During controlled placement of the first lifts of sand fill by a spreader pontoon, large deformations of the underlying dredged sludge were observed and filling had to be stopped. The undrained shear strength of the sludge after placement in the fill area appeared to be less than anticipated during the design phase of the project. Although other causes may have contributed as well, the unexpected low strength is generally attributed to a more than anticipated strength loss during dredging, transport, and placement of the sludge in combination with gas generation as a result of the organic nature of the material. As it was not possible to place the first lifts of sand fill without excessive deformations, the application of the *BeauDrain-S* vacuum consolidation system has proven to be crucial to the project. This system could be installed from floating pontoons and increased the undrained shear strength of the underconsolidated sludge by mobilising (part of) the atmospheric pressure as an isotropic load. After a few months pumping, the first lifts of dry sand could be placed. In addition, the system released and removed (part of) the gas dissolved in the organic-rich sludge. Last but not least it can be concluded that the excellent cooperation between all parties involved (client, consulting engineer, and contractor) has been a key factor to the successful completion of this challenging project.

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Editorial note: the original version of this article has been presented at the 32nd PIANC MMX Congress, Liverpool, UK, 10-14 May 2010.

De Ondergrondse

Thomas Nijssen *(chairman 2010-2011 board)*

Introduction

De Ondergrondse is the student association for master students Geo-Engineering at TU Delft. The association represents the entire Geo-Engineering Section: Geomechanics, Engineering Geology, Geo-Environmental Engineering, Foundation Engineering, and Underground Space Technology. The association itself was created in October 2006 after merging De Ondergrondse and DIG in the wake of the launch of the new Master Geo-Engineering. Every November the entire board of De Ondergrondse changes. The main goals of De Ondergrondse are to attend the interests of master students Geo-Engineering and the promotion of contacts between students and the business community. Through the newsletter, members are kept informed about the latest developments in the world of underground construction and other related aspects of Geo-Engineering. During the year a lot of different activities are organised, e.g. lunch lectures, excursions, and a study trip abroad. The study trip is a yearly returning event and De Ondergrondse has in recent years already visited many locations all over the world. In 2010, De Ondergrondse went to Scandinavia. Main goal of the trip was to find out more about the engineering difficulties and problems that our Scandinavian colleagues encounter during the design and construction of their geotechnical projects. And further, how do they deal with ground related risks? Is their approach different from the Dutch approach? During several excursions to building pits and presentations of Scandinavian engineers we got a clear picture of their problems. In this article you will find reports of students describing several parts of the study trip. Further, a report of a visit to a railway tunnel in Brussels is presented.

Scandinavia study trip 2010

Day 3: Stockholm City Line (Citybanan) (by Arjan Kochx)

On the third day of our study trip we visited the construction site of the *Citybanan* (Stockholm City Line). This is going to be a brand new railway line that will be situated underneath the city centre of Stockholm (see Figure 1). It will consist of two lines that will be used by commuter trains, regional trains, long distance trains, and freight trains. The reason why this line is so necessary becomes clear when looking at the rail network available in the city. The capacity has reached its absolute maximum with a quarter of a million passengers each day and 24 train movements per hour. Since eight out of ten trains in Sweden depart from Stockholm, delays within this hub will have consequences for the entire country. The new line aims to double the current capacity when it starts operating in 2017, from 24 to 48 trains per hour. This number is anticipated to full demands up to the year 2030. The line is being built in an approximately 6 km long tunnel, that runs from Tomteboda to Stockholm South Station and consists of the main tunnel, a service tunnel, a rescue tunnel, and two new stations (*Odenplan* and *City*). The total cost after completion is estimated at SEK 16.3 billion. Due to this huge size of the project and the risks involved, the contract has been split into three different parts and will be constructed by different companies. To further minimise the risks from an operational point of view a very intensive geological survey has been undertaken before the whole operation started. Both geophysics and boreholes have been used to get the required information to analyse the geohazards that could be encountered. Together, all this data contributed to the most complex geotechnical model ever used for a project in Sweden. The major part of the tunnel is constructed in very hard granite. To excavate this rock, drilling and blasting is used for pretreatment. The channel crossing between *Riddarholmen* and

Figure 1 Citybanan plan.

Söder Mälarstrand however, is realised by using prefabricated tunnel elements that will be submerged. We visited the entrance and part of the tunnel in the granite at Sodermälm at the location where the blasted tunnel is going to be connected to the submerged tunnel. What was noticed instantly was the shotcrete lining (concrete that is conveyed through a hose and pneumatically projected at high velocity onto the bedrock surface) in the tunnel, preventing small blocks from falling onto workers on the site, as well as the reinforcements applied to ensure the stability of the tunnel as a whole. Two other interesting items were the blasting face and the sluice construction near the future connection with the submerged tunnel. Near the blasting face (where holes are drilled and explosives are inserted) there wasn't any shotcrete applied yet so the fresh granite could be seen. At this location, some spots were marked with red paint showing potentially dangerous rocks that either needed to be removed and filled with grout or be otherwise supported with bolts. The sluice is a temporary construction, which ideally won't have to be used. It is built at the connection between the blasted tunnel and the submerged part. If this connection fails for some reason, the possibility exists that the tunnel might be filled with 8 m of water which would be disastrous, but will be prevented with this measure.

The geology within the 6 km long tunnel consists mainly of granite which was visible during the visit, but also pegmatite dikes and eskers have been identified. These eskers (long winding ridges of stratified sand and gravel) could be a possible threat as when these artefacts are encountered during tunnel construction, a major sand inflow into the tunnel is expected together with high unwanted settlements on the surface. Especially a large esker that has been located underneath the historical church is expected to be challenging to pass. Freezing is probably the only type of remediation possible here, which is effective but expensive. The quality of a rock mass is determined using two more or less similar systems i.e. the RMR system and the Q system, from which the most critical value is taken. The rating of the quality of the rock mass within these systems depends on the material of bedrock and the kind and number of faults and joints. Together with strength parameters of the rock this gives an estimate of the stability of the rock mass as a whole. Based on this information the decision is made how to reinforce the tunnel, e.g. spot bolting (only support problem spots) or systematic bolting (bolting within a certain grid, depending on rock conditions). Due to the water close to the tunnel and the abundant faults within the rock at this place, there is a high risk of water inflow through these faults. To prevent this hazard, grout is injected under high pressure through a screen of injection wells up to a depth of 15 metres in front of the blasting face. After this injection, further preparations for the blasting procedure are undertaken. Somewhere between 190 and 200 holes are drilled into the rock face, which will later on be filled with explosives. The blasting can theoretically remove up to 4 m of rock each time. Restrictions on building underneath the city centre allow for only 2 m blasts each time. After this blast, the blocks are removed and the new face is examined. Potentially dangerous blocks are marked. In case of underblast, more rock is removed and in case of overblast, the theoretical profile needs to be filled with cement. When all measures are taken the whole cycle can start all over again.

In total, the whole underground experience took us approximately 15 minutes. After leaving the tunnel and being back into broad daylight, the engineering geologist working on site explained about the tunnelling operation as a whole. After a new rock face is exposed, the engineering geologist examines the rock face. He tries to estimate the quality of the rock on site, this is important since the type of reinforcement within the tunnel will depend on it. After this interesting talk we were spoiled with a nice smoked salmon sandwich and a fresh cold beer preparing us for some action again, namely 'exploring the city for the rest of the day'.

Day 8: Hamina dredging project (by Ruud Arkesteijn)

We arrived at exactly the right time at the port of Hamina, a town in Finland close to the border with Russia. In the site office of Terramare, sandwiches, drinks, cupcakes, and, more importantly, an enthusiastic atmosphere were awaiting us. We were warmly welcomed by Gert Jan Peters and Maarten de Wit. Gert Jan is a Dutch project manager who has been working for Terramare, which was acquired by Royal Boskalis Westminster nv 10 years ago. Maarten is doing his internship at Boskalis and together with Gert Jan they are now working on their second project together. Fortunately, it was not just another project we were about to visit…

Gert Jan provided us with a presentation about the project, showing us the decision-making process, an overview of the port, and the most important aspects of the project. In December 2008 Terramare signed the contract for making the port accessible for Panamax size oil tankers, for which a minimal water depth of 13 m is required. This was seen as a necessary development for the port to stay ahead of nearby ports in Russia. The works in the access waterways started in spring 2009. The seabed that needs to be removed consists of rock, soft soils, a contaminated upper layer, and some boulders every once in a while. For the soft soils an old hopper dredger called *WD Medway II* was brought in to fulfil one

Figure 2 Split barge.

of her final tasks in this beautiful stretch of the Baltic Sea. The contaminated upper layer was dredged separately and put in a dump site which was closed off from the outside environment using a screen of air bubbles. For removing big boulders an extra fee of €9,000 is paid by the client. Considering that over 300 boulders have been found so far, one can understand this makes for a worthwhile bonus. The most exciting task within the project is the removal of the rock bottom. For this task, explosives are used to crack the massive rock body in smaller removable parts. Unfortunately we were not able to witness a blast in real-time during the site visit, as they had already blasted the morning before. But Maarten gave us a good impression by showing a self-made movie of a blast. After the project description and some nice movies and images it was time to go to the actual work site. The group was divided into two groups before boarding two survey vessels. First we took a look at the dump site for both blasted and excavated rock as well as dredged soft soils. All removed material was deposited at a part of the port where land reclamation would make it possible for the port to grow. We circled around a split barge which was just about to drop a load of blasted rock (see Figure 2). These vessels dump their material simply by splitting the entire hull lengthwise to make it possible for the material to fall through the bottom. The total width of the gap becomes around 2.5 m but nonetheless there happened to be a piece of rock which was twisted in such a way that is was not able to fall through. Gert Jan told us that this happens about twice a week. The vessel had to return to backhoe dredge *Nordic Giant* with an opened bottom. Obviously these kinds of problems are not good for the project's planning and economy, but it was nice to witness. A 40-minute boat ride through the typical Finnish archipelago brought us to the *Nordic Giant* and drill barge *Playmate*. Slowly we neared the pontoons which were visible on the horizon. In the meantime we were able to chat with Gert Jan and Maarten and ask questions about different projects, but also about the extraordinary life as a globetrotting engineer. When we arrived at the *Nordic Giant* it became clear where the name of the barge comes from. The length of the pontoon was equal to the width of a football field and the size and teeth of the bucket seemed to be perfect for yet another sequel to *Jurassic Park*. On board, everybody was stunned by the impressive sounds and sights of this floating factory. All of a sudden the crane turned 180 degrees until the bucket was right above our heads. As we stepped aside, the bucket with a size of about 22 m^3 was put on the pontoon and we were able to take pictures as the whole group (!) was standing inside. Later we were shown around in the cabin of the crane operator. It's the control room for all movements of the pontoon, the crane, and the bucket. It was remarkable to see how much comfort there was in this cabin; it was nicely isolated from the outside noise, it had air-conditioning and the crane operator was leaning back in his chair. He had taken off his shoes and his feet were up, and he was mumbling what turned out to be Finnish swearing words. A few moments later it became clear why he was not so happy: he lifted the bucket and the whole crane, cabin and probably the whole pontoon were vibrating and tilting as he brought an incredibly big piece of rock to the surface. It seemed to be twice the size of the bucket and therefore it was approximately 40 m^3 or 90 tons. As this size is far too big to fall through the split barge, the crane opera-

Figure 3 On board backhoe dredge Nordic Giant.

tor had to put it back at the seabed so it could be localised and blasted later. As a visiting group we were thrilled to see the biggest possible piece of granite, but it was actually the result of a default in one of the blasting rigs. The mixture of water and explosive fluid was not as it should have been. As a result, half of the blasted area had less fractures and therefore bigger chunks of granite. Before we went back to the shore we visited drill barge *Playmate* up and close from the vessels. We also circled the *Nordic Giant* once more as the split barge with the large piece of rock stuck in it was placed alongside. The crane operator tried to twist the piece of rock so it would fall through and could be blasted along with the other massive block. As we arrived at the starting point of the boat trip, there was still a bit of time left to climb one of the tall port cranes to get a nice overview of the port, the dump site, the archipelago, and even the *Nordic Giant* and *Playmate* behind one of the islands. At the end of the visit everybody was really thankful to Boskalis and especially Gert Jan and Maarten to arrange this awesome visit at the end of our study trip. Some brought small pieces of blasted granite home as a souvenir. De Ondergrondse would soon find a tooth of the bucket of the *Nordic Giant* in their mailbox.

Excursion to the Josafattunnel in Brussels (by Tim van der Biezen)

Last year, De Ondergrondse organised a field trip to a project in the centre of Brussels. Grontmij has co-designed a portion of a new railway tunnel. We got the chance to take a close-up look at the project and get to know how the Belgians tackle their engineering problems and learn how it is to work for a company like Grontmij. We started the day with a lunch followed by a slide show of what we can expect after we have finished our education. It was a very useful talk about career options and what we can do ourselves to get the most out of our potential. We focussed on what are important choices in the future, such as a specialisation, future companies, and career planning. After this promotional presentation disguised as advice, an in-depth look at the project followed. It is a joint effort between Grontmij and other companies. The emphasis was on the part of the tunnel designed by Grontmij and this is where we went underground.

You can look at Brussels from above and divide it in two: a big area in the west and a smaller area in the east. The one in the west is where the big train stations are (Brussels North, Brussels Central, and Brussels South). The project of the Josafattunnel focuses on the eastern area. The goal is to connect this part of the capital with the important railroads of the west as well as the airport. Big projects like this always come with difficult challenges. One of the conditions is to minimise the disturbance of life above ground level. The Belgians have some techniques of their own to conquer this. An interesting one is the infamous technique of *beschoeide sleuven*. It is a relic of the mining ages and quite a low-tech approach to tunnelling; basically it is a substitute for a water-blocking slurry wall. The method is infamous because it apparently has a track record of accidents, some of which were probably fatal ones. A guy with a shovel starts digging on ground level, putting slates of concrete against opposite walls and keeping them apart using shoring braces. These trenches go down many metres, and so do the workers. One can imagine what happens when procedures are ignored. For us it was very surprising to see this method in reality, we saw the workers descending on little elevator chairs, having their cigarette break in the middle of the tunnel and see them balancing halfway down the shaft. According to our guide, no accidents have happened yet on this project. We continued our tour and saw much more interesting stuff. We arrived at the end of a gallery and saw a couple of workmen digging their way through. One had his sleeves pulled up and a cigarette in his mouth, using his power drill to cut through big chunks of sandstone. All the while there were men with automated wheelbarrows disposing of the sand, which was dumped in a hole in the ground where it was taken away by an underground train. We also came across the installations for compensation grouting. Just as with the *Noord/Zuidlijn* in Amsterdam, this tunnel is also partially built below buildings, including the Dutch embassy. All across the area above ground there is a measuring system set-up, with an instant alarm system for when the buildings start subsiding. The maximum tolerance is set for a drop of 20 mm, but apparently they are currently 3 mm higher. We concluded our trip with a drink in a local pub, where they were completely unprepared for the sudden arrival of 20 customers. Nonetheless it was a fitting end to a day filled with learning opportunities made possible by the people of Grontmij.

Book review
Nutzung von Massenschwerebewegungen im Tagebau

*Reviewed by **dr. Peter Verhoef** (Senior Engineering Geologist, Royal Boskalis Westminster nv, Papendrecht, the Netherlands)*

Early in the 1980's, when the works for the Eastern Scheldt Storm Surge Barrier were prepared, Rijkswaterstaat had a great interest in methods of quality control and durability assessment of armour stone. Many rock types imported from different countries were under scrutiny. The most convenient transport for the rock was by water, so rocks from Norway, Finland, and Scotland were considered and of course the traditional rocks coming from quarries situated along the rivers Rhine and Meuse. Dutch aggregate and rock trading firms own some of the traditional quarries. One of these is the Sooneck quarry, which is situated along the river Rhine near the village of Trechtingshausen. At TU Delft, the Engineering Geology section carried out a comprehensive program into the durability assessment of rocks intended for hydraulic construction purposes, research which was supported by Rijkswaterstaat for many years. The rock at Trechtingshausen is a quartzitic greywacke type of sandstone. In some of the shipments of armour stone that were transported to Holland, poorer quality rock was present. In 1984 Tjaart Kuipers, then a structural geologist at TU Delft, made a site visit to the quarry and reported that there was a type of structural control in the quarry that had resulted in the winning of shale-rich rock that was intercalated in the greywacke rock sequence. The structure in the quarry was fascinating; it showed the asymmetric parasitic folding and thrust faults related to late Carboniferous (Varistic) structures that mark the geology of the *Rheinisches Schiefergebirge*.

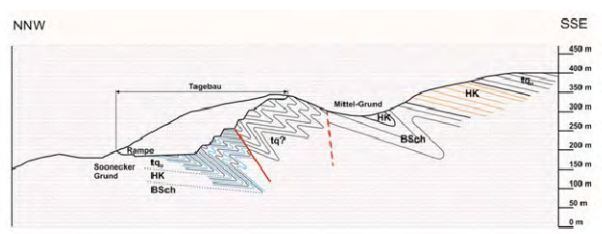

Figure 1 Geological cross section through the Sooneck quarry (all illustrations are taken from the book).

As a result of this work, the Section of Engineering Geology at TU Delft had made contact with quarry owner De Beijer in Arnhem, and later with the quarry manager in Trechtingshausen, Willem Douw. It appeared that Willem, a mining engineer, was working on an innovative plan for the future mining operations of the Trechtingshausen quarry. He was planning to create a transport system that would use gravity instead of trucks to carry rock fragmented by blasting at the higher benches to the base of the quarry. The idea was to use an ore pass or chute (artificial gully) to transport the rock blocks downslope towards the breaker, sieve installation plant, and stockpile area. Considerable cost savings can be obtained by using a mass movement rock pass to move material downwards in an opencast mining environment. The execution of this idea would reduce the environmental impact of the mining operation considerably. Willem Douw set out to investigate the application of this idea to the Sooneck quarry by examining all aspects to come to a mine planning for the future, based on the use of a gravity pass, and he also patented the idea. Using a chute is a practice that has been used in the past and is known in Europe since Roman times. A copper gravure of the Drachenfels from 1646 (Figure 2) shows the *Steinrutsche* that was used to deliver the trachyte building stones to the border of the river Rhine for construction of the Kölner Dom.

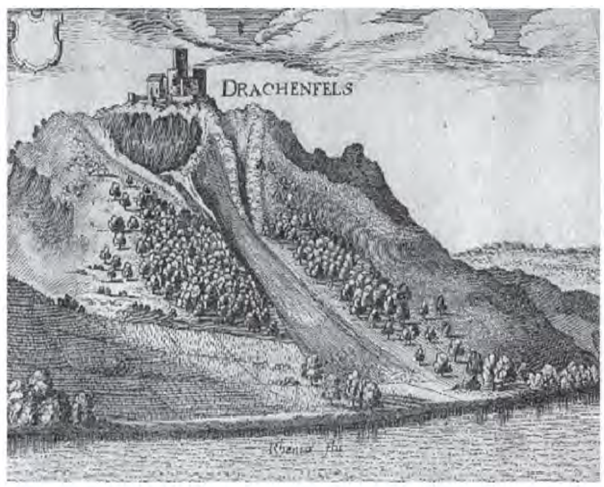

Figure 2 On this copper gravure of the Drachenfels, a volcanic neck of trachyte, the chute used to slide building stone blocks downslope from the quarry below the castle is clearly visible.

It is clear that the subject of rock slope stability was crucial for the project and this required a thorough assessment of the structure of the rock and how this would affect the stability. In 1998 Willem Douw contacted me to discuss the possibility to have a study carried out in the quarry. This became the MSc thesis work of Xander van Beusekom (Beusekom, 1999). There were several difficult challenges, of which the safe access to the rock face was not the least. Nowadays we would solve this probably by using LIDAR optical remote sensing equipment as described by Siefko

Slob in his thesis (Slob, 2008), but then, as many direct measurements as possible were made in the rock mass. Xander assembled a large number of data which were analysed using Robert Hack's SSPC system (Hack, 1996). The stability of the rock mass surrounding the chute and the position and design of the chute (too shallow: blockage and stalling problems; too steep: too high velocities of rock aggregate, bouncing rock blocks) were crucial. Xander's work led to a best position for the chute, which was at another location than would be favourable from the perspective of mine operations (see Figures 3 and 4). In his book *Nutzung von Massenschwerebewegungen im Tagebau*, Willem Douw has comprehensively recorded the research done to develop a mining plan for the Sooneck quarry concession. Although using ancient principles, it is an innovative and modern mining design which uses a 'building with nature' concept that reduces environmental impact significantly. The resulting book is prepared with care and gives a review of the use of gravity chutes in mining with examples from history. The development of the design for the Trechtingshausen case is comprehensively described, with much attention for the local geomorphologic and geological setting. It includes a

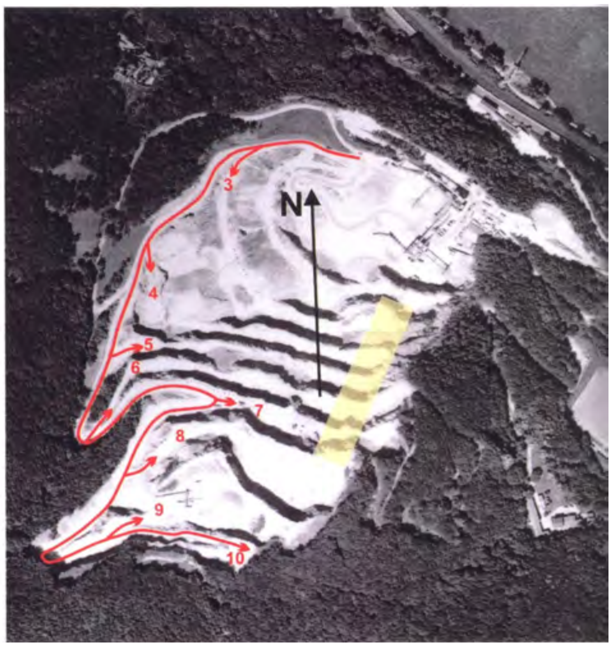

Figure 3 Ortho-photo of the Sooneck quarry in 2001, showing the preferred position of the pass in yellow. This preference is dictated by the road entrances to the bench levels.

consideration concerning the genesis of the Rhine gorge south of Bingen. Anyone interested in open-pit quarry development should take the effort to study this work. It includes a nice overview of the history of mining methods used in the area. The analysis of the mine design and mine planning shows the benefit of the rock engineering methods used. It is enjoyable reading and as a second benefit for myself it was a good refresher of my German. The PhD thesis was successfully defended in 2007 at the University of Mainz.

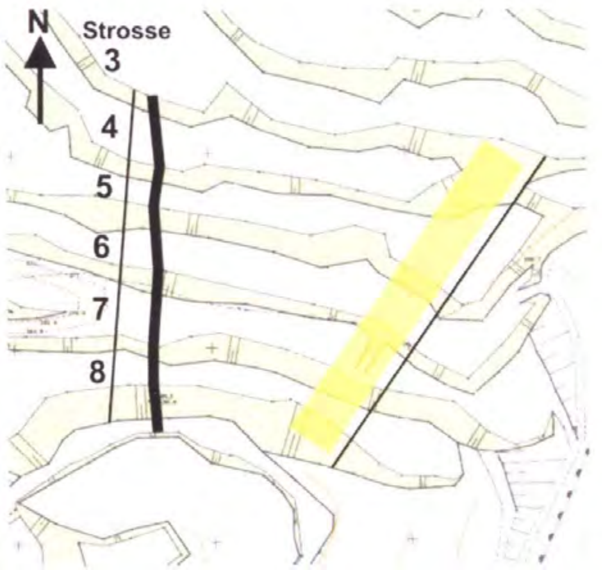

Figure 4 The black line on the left shows the best position for an ore pass based on the SSPC analysis by Xander van Beusekom in 1999.

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Nutzung von Massenschwerebewegungen im Tagebau

Willem Douw

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Mining gas hydrates: where and how to look?

Edwin Tervoort *(Fugro, Leidschendam, the Netherlands)*

Abstract

Gas hydrate is an unconventional resource for methane gas, where gas is trapped in 'ice'. Vast quantities of gas hydrates are present below the seabed of the continental margins. A promising method of mining methane from these gas hydrates is by releasing the methane using dissociation. Dissociation can be promoted by depressurising gas hydrates. Applying this method to gas hydrate depots directly on top of small conventional gas reservoirs creates a 'perpetual gas reservoir'. National governments and major energy companies have become interested in gas hydrates and hence in quantifying and characterising gas hydrate deposits. Common strategies and techniques have been developed to investigate gas hydrates. Bottom-simulating reflector (BSR) on seismic reflection data and seismic velocity provide the best means for localising potential gas hydrate occurrences at a site, also electrical conductivity is a particularly useful measurement to identify gas hydrates. Downhole geophysical logs provide a continuous profile of bulk properties of the sediments containing gas hydrates. Coring confirms the results of these indirect measurements. Pressure cores and pressure core analyses provide key pieces of information to quantitatively and qualitatively assess gas hydrates. These technologies have become standard components of gas hydrate site investigations and have matured to become more routine operations.

Introduction

Gas hydrates consist of water that appears as a solid, but the crystal structure differs from ice by forming cages (or *clathrates*) around guest molecules that support those cages. One such guest molecule is methane. Gas hydrates are only stable below a 'cold overburden' (high pressure and low temperatures). These conditions are met on land below permafrost and further below all the world's oceans on the continental margins (below a large water column with cold bottom water temperatures). It has been estimated that the quantity of methane stored in gas hydrates potentially exceeds the total amounts of already produced methane gas combined with methane gas stored in conventional gas reservoirs and other unconventional resources (e.g. gas in shale or tar sands). Research mainly focuses on developing methods to produce methane from gas hydrates. Methane can be released from its cages by dissociation of gas hydrates, either by thermal perturbation or by depressurisation. Thermal perturbation increases the temperature of gas hydrates outside the gas hydrate stability zone (GHSZ, see Figure 1). Recent production tests performed in an onshore test well through the permafrost at Mallik (Northwest Territories, Canada) have shown that thermal injection was not very successful as it was hard to increase and maintain the temperature over a large volume. Depressurisation reduces the pressure in the reservoir outside the GHSZ (see Figure 1). This depressurisation could be well maintained during the production tests at Mallik. The prime location for gas production from depressurising gas hydrates is where a small conventional gas reservoir sits directly below a large gas hydrate deposit in permeable sediment. By extracting gas from the gas reservoir, the pressure in the reservoir is lowered and dissociates the hydrate deposit above, which feeds the reservoir with gas. This results in a 'perpetual gas reservoir' that is filled while gas is extracted. Gas hydrates can host different kinds of guest molecules in their clathrates and therefore methane can be replaced by another guest molecule, for instance carbon dioxide (CO_2). Methane production could therefore be combined with CO_2-sequestration (storage of CO_2) and as a result could make methane production from gas hydrates CO_2-neutral. Gas hydrates can be regarded as a geohazard (geological feature or process that has the potential to result in severe damage to humans, property, or the natural or built environment).

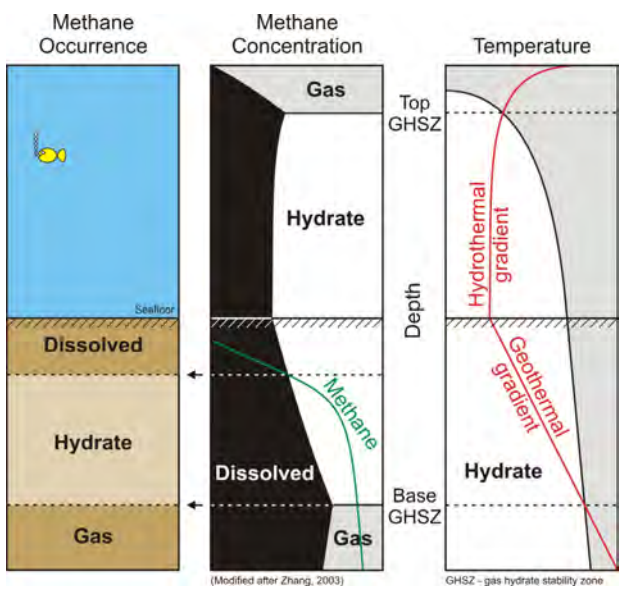

Figure 1 Methane occurrence in a marine environment. Gas Hydrate Stability Zone (GHSZ) is defined by temperature and pressure; below the Base GHSZ and above the Top GHSZ the temperature is too high (for the given pressure) to form gas hydrate. Only when the methane concentration is above methane saturation (i.e. maximum solubility of methane), methane will occur as hydrate (and dissolved gas) within the GHSZ. Outside the GHSZ, methane will occur as free (and dissolved) gas, when the methane concentration is above saturation. When the methane concentration is below saturation, methane will only occur as dissolved gas.

For example, the release of gas can cause increasing pore pressures and hence reduction in effective stress in the host sediments, causing slope failure. Dissociation of gas hydrates can also remove cementing properties of gas hydrates in sediments, which reduces the support of the sediment. This loss of support can cause well instability. Gas hydrates have drawn a lot of attention from national governments, major oil companies, and environmentalists, besides the scientific community. All these parties have the same interest of quantifying and characterising gas hydrate deposits. This paper will therefore focus on gas hydrate site investigation and applied techniques. As most gas hydrates can be found below the oceans, the focus of this paper is on offshore gas hydrate site investigation.

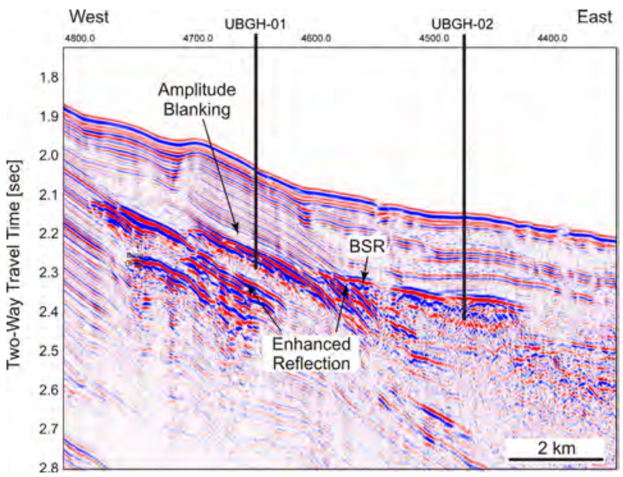

Figure 2 Gas hydrate indicators on seismic data. Bottom-simulating reflector (BSR) is associated with the interface between gas hydrate (above) and gas (below). The depth of the BSR corresponds to the depth of the base GHSZ, which is defined by the geothermal gradient. Gas appears as enhanced reflections and gas hydrate as blanked amplitude reflections in this section. Example is taken from the Ulleung Basin in the East Sea between South Korea and Japan (courtesy of KNOC).

Strategies and techniques for gas hydrate site investigations

Gas hydrate site investigations are usually designed with the same objectives in mind: defining the limits of the GHSZ, quantifying the gas hydrate concentration, determining the distribution of gas hydrates over the site and the morphology of gas hydrate within the sediment, and characterising the in situ (bulk) properties of the sediments containing gas hydrates. To achieve these objectives, gas hydrate site investigations usually consist of three phases: 1) reconnaissance phase to localise potential gas hydrate occurrences at the site, 2) logging phase to determine the bulk properties of the sediments and to specify gas hydrate occurrences with depth, and 3) coring phase to confirm the indirect measurements of the previous phases and to quantitatively and qualitatively assess gas hydrates.

Reconnaissance

Potential gas hydrate occurrences are identified during a reconnaissance survey by applying geophysical techniques such as seismic reflection. Optionally, visual observations from near-seabed cameras are included in these surveys. For instance, observations of gas seeps from seabed in deep water environments can be indicative for potential gas hydrate accumulations below seabed. These gas seeps indicate that components to form gas hydrates (gas and water) are present at the right temperature and pressure. The presence of a bottom-simulating reflector (BSR) on seismic reflection data is associated with the presence of gas hydrates. The BSR is an anomalous seismic reflection following the contours of the seabed (see Figure 2). The anomalous reflection of the BSR possibly marks the interface between gas hydrates above and free gas below. The BSR can also reflect a relict feature of a former interface between gas hydrates and free gas that no longer exists. The BSR can be regarded as the base of the GHSZ, below which gas hydrates are not stable and only free (and dissolved) gas exists (see Figure 1). The base of the GHSZ is mainly a function of the geothermal gradient which generally does not vary significantly across the site. The depth relative to seabed of the base of the GHSZ is therefore constant and hence follows the contours of the seabed. Gas hydrates might appear as blanked amplitude reflections on seismic reflection data (see Figure 2). Gas hydrates also affect the seismic velocity. The cementing properties of gas hydrates increase the rigidity of the host sediment, which slightly increases the seismic velocity. This might show on seismic reflection data as pull-up structures. In contrast, free gas below the base of the GHSZ reduces the seismic velocity and might appear as pull-down structures.

Logging

The increase in seismic velocity for hydrate bearing sediments can be determined by downhole geophysical measurements like sonic logging. There are also other geophysical properties measured by downhole logging that are indicative for gas hydrate. A particularly useful downhole measurement is electrical resistivity. Electrical resistivity is much higher for sediments containing gas hydrates than for sediments without gas hydrates. This is because gas hydrates only consist of fresh water, which has a very low electrical conductivity (and therefore high resistivity) properties, while pore water is generally saline with high electrical conductivity. Other downhole geophysical measurements to consider for indicating gas hydrates are gamma density (slight decrease) and neutron porosity (slight increase). The natural gamma ray is not affected by the presence of gas hydrates. Caliper tool measurements show in general a more ragged borehole wall when gas hydrates are present. Downhole geophysical measurements can be applied to quantify gas hydrates. However, this only works well for gas hydrates in sand, where the porosity can be derived from sonic, density, or neutron logs, and where the saturation of gas hydrates in the pore space can be determined from electrical resistivity. However, gas hydrates also occur in clay, where they tend to form complex networks of veins (see Figure 3). Electrical resistivity will vary strongly with gas hydrate morphology and needs ground truthing by using pressure cores for quantitative assessment. Downhole measurements are performed by wireline or logging-while-drilling (LWD) techniques. The wireline measurements are usually combined with pilot holes that are drilled before the actual coring. The pilot holes are for verifying any potentially hazardous gas accumulations.

Coring

Strategic planning of pressure cores (and non-pressurised cores) is important for gas hydrate site investigation and relies heavily on the results from the logging phase. Pressure cores are cores that are recovered and retained at their in situ pressure. Pressure cores typically comprise less than 10% of a cored section in a borehole, leaving approximately 90% of the remaining sediment to be characterised by other means. The properties of any sediment not recovered in pressure cores and non-pressure cores are inferred from

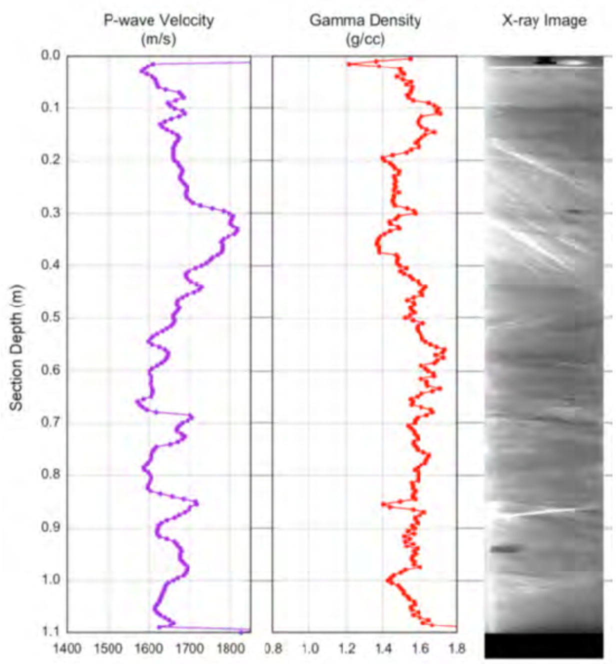

Figure 3 Non-destructive measurements on a pressure core. X-ray image shows gas hydrate (white) morphology in clay (grey). Note the complex structure of fine veins. Where gas hydrate is present, the P-wave velocity is higher and the gamma density is lower than where gas hydrate is absent (courtesy of GeoTek).

Figure 4 Pressure corer handling. The pressure corer (FRPC) is hoisted into the derrick to be lowered in the drill string. The drill string acts as casing during the pressure coring operations (courtesy of Fugro).

downhole measurements. Pressure cores provide the key information for meeting the objectives of defining the limits of the GHSZ, quantifying the hydrate concentration, determining the gas hydrate morphology within the sediment, and characterising the bulk properties of gas hydrate bearing sediments. Pressure cores ensure the recovery of gas hydrate bearing sediments with minimal disturbance and hence the preservation of gas hydrate morphology. The HYACINTH pressure corers have been utilised since 2002 at various sites and consist of two types of wireline pressure coring tools: the Fugro Pressure Corer (FPC) and the Fugro Rotary Pressure Corer (FRPC, see Figure 4), previously referred to as HYACE Rotary Corer (HRC). The FPC is a percussion corer and the FRPC is a rotary corer, which were developed to cut and recover core in a wide range of lithologies. The HYACINTH system also involves techniques for analysing pressure cores in a non-destructive way and storing pressure cores for further laboratory studies. Particularly, the pressure cores are scanned offshore for X-ray, seismic velocity, and natural gamma ray to determine the bulk properties of the core (see Figure 3). After that, sub-samples of the pressure cores may be depressurised to collect quantitative gas samples. The remaining pressure cores can be stored with the pressure retained and can be transferred to onshore laboratories for detailed investigations. Pressure coring and pressure core analysis have become standard components of gas hydrate site investigations for industry and academia. These technologies have matured and the success rates improved to levels where the operations have become more routine. Note that pressure cores provide the only means for confirming the absence of gas hydrate and for describing gas hydrate morphology in sediments qualitatively and quantitatively (see Figure 3). Gas hydrate bearing sediments recovered by conventional coring techniques are disturbed by gas exsolution and gas hydrate dissociation, due to the reduction in pressure (and the increase in temperature) upon retrieval. Nevertheless, gas hydrates can be inferred from non-pressurised cores by examining infrared thermal images acquired along the entire length of the core immediately after retrieval. Dissociation of gas hydrates is an endothermic process and results in thermal anomalies (cold spots). Non-pressurised cores are also examined for pore water chemistry and sedimentological descriptions. The coring phase always includes in situ temperature measurements to help determining the gas hydrate stability zone (see Figure 1). The coring phase may be augmented with in situ pore water sampling, strength, and pressure/ permeability measurements.

Gas hydrate quantification from depressurising pressure cores

The amount of gas hydrate present in pressure cores can be determined by depressurising the core. Depressurising promotes dissociation of the gas hydrates to its components: gas and fresh water. The total amount of gas and water after depressurising the core is therefore the sum of the amount of (dissolved) gas and pore water at in situ conditions and the amount of gas and water from gas hydrate dissociation (see Figure 5). Hence, the amount of gas hydrates can be determined from the difference in the total amount of gas or water after depressurising in comparison to the amount of gas and water at in situ conditions. The amount of gas at in situ conditions can be determined from the solubility of the gas and the amount of pore water. When the measured gas concentration is larger than the calculated solubility of the gas at in situ conditions, then the gas that cannot dissolve originates from gas hydrate dissociation. The assumption is that gas is fully saturated and does not occur as free gas next to gas hydrate. The amount of pore water at in situ conditions can be determined by pore water freshening due to dissociation, i.e. the dilution of the concentration of dissolved elements in pore water. This requires a baseline concentration for the dissolved elements before dissociation. Chloride is a conservative ion that can be used as element to determine the baseline concentration. The in situ chlorinity of pore water can be determined by direct measurements from in situ pore water samples or by analysing pore water extracted from cores (pressurised or non-pressurised). The ratio of the chlorinity after dissociation and the in situ chlorinity is a measure for the pore water freshening and hence the amount of gas hydrate dissociation.

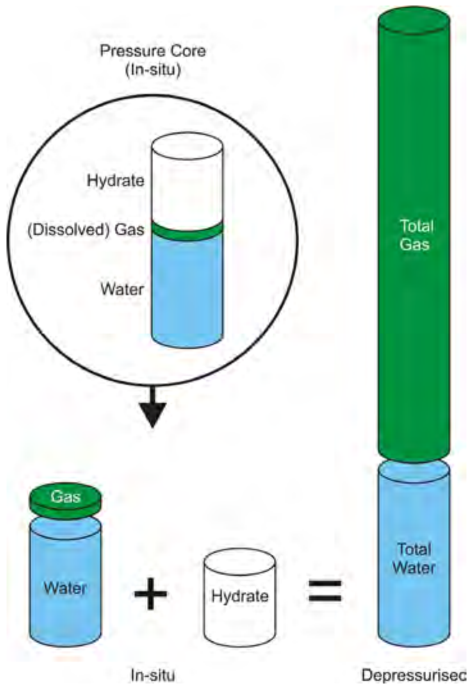

Figure 5 Gas hydrate quantification from depressurising a pressure core. The total gas content is the sum of the amount of dissolved gas at in-situ conditions and the amount of gas from hydrate dissociation. The total water content is the sum of the amount of pore water at in situ conditions and the amount of water from hydrate dissociation. Therefore, the amount of gas hydrate present in pressure cores is determined from either the difference in the amount of gas or the difference in the amount of pore water in the pressure cores before and after depressurising.

Acknowledgements

The author gratefully acknowledges Fugro's commitment and support for improving practice and also likes to thank other Fugro colleagues for their review of this document. The opinions expressed in this paper are those of the author and are not necessarily shared by Fugro.

EUROCK 2009 and 2010

Michiel Maurenbrecher MSc CEng

EUROCK 2009

EUROCK symposia appear to come in two to three consecutive years between those of the international ISRM conferences. The last two international ISRM conferences were held in 2007 (Lisbon) and 2011 (Beijing). One EUROCK conference appears to be missing in the sequence, as 2008 does not appear in the list. The theme of EUROCK 2009 was *Rock Engineering in Difficult Ground Conditions-Soft Rocks and Karst*. The conference was held in Cavtat, Croatia (across the bay from Dubrovnik) on 29-31 October 2009. Three papers from the Netherlands were presented at the conference:

- 'Strength and deformation of biologically cemented sandstone' by L.A. van Paassen, M.G.M. van Loosdrecht, A. Mulder, D.J.M. Ngan-Tillard, and T.J.M. van der Linden (paper based on the PhD research and thesis of Leon van Paassen; presented by Theo van der Linden).
- 'Microstructural degredation of Maastrichtian limestone' by D.J.M. Ngan-Tillard, W. Verwaal, P.M. Maurenbrecher, and L.A. van Paassen (paper describes research work which was done on limestone core samples taken for the A2 bypass tunnels around Maastricht; presented by Dominique Ngan-Tillard).
- 'Pocket cards to aid description of carbonate rocks as core or outcrops' by P.M. Maurenbrecher and D.J.M. Ngan-Tillard (these cards were designed recently as an aid for describing core taken from the site investigation for the A2 tunnels; presented by Michiel Maurenbrecher).

Standing in line to check in for the first flight to Zagreb was (now former) ISRM president Prof. John Hudson. As symposia are also meant for informal discussion between engineers and scientists having similar interests I introduced myself by asking him if he was heading for Cavtat. I had met Prof. Hudson many years ago at a meeting at Imperial College with David Price and Mike de Freitas about the possibility of him giving guest lectures at Delft. Professor Hudson had just come from Helsinki where he had looked at Carrara marble cladding which was showing distress and the best connection for a flight to Croatia was through Schiphol. He asked me if I was presenting a paper and I replied that it had to do less with research but more with practical aids for rock mechanics engineers or engineering geologists for logging of rock at outcrops or from samples. The flight to Zagreb and the connection to Dubrovnik went smoothly. As the plane approached the coast, the karstic features in the terrain could easily be seen in the form of many sinkholes. The coast and islands had steep cliffs suggesting collapsed caves. The name *karst* comes from Croatia. Dubrovnik airport is on an elevated plateau between the higher plateau of Bosnia-Herzegovina and the present shoreline. A shuttle bus from Hotel Croatia soon found the Dutch ISRM party and Professor Hudson at a very smart hotel in idyllic surroundings on a wooded limestone peninsula across a small bay from the fishing and resort town of Cavtat. The rooms overlooked a flat sea with a setting sun to the west, with boats plying their way to and from Dubrovnik across a larger bay from Cavtat just to the north. Early October in the Balkans was still warm enough to have dinner outside at a harbour restaurant. The following day started with an intense programme of keynote lectures, by Ivan Vrkljan (the symposium organiser) on rock mechanics in Croatia, followed by John Hudson about stresses in rock masses, and Evert Hoek giving a lecture written together with Paul Marinos on tunnelling in overstressed rock. This was followed by coffee on the hotel terrace overlooking the bay of Cavtat and as a backdrop the cliffs leading up to the limestone plateau in Bosnia-Herzegovina. The morning sessions continued to lunch and further sessions, with a terrace tea break. The full day of technical presentations and listening made way for the welcome cocktail with catering on yet another terrace of the hotel. By the first day all the presentations from the Netherlands were given leaving my own for the next day, after lunch, in the main hall which looked quite empty possibly because the subject matter looked less interesting than that of the parallel session which I would have rather attended (on case histories). At least John Hudson came to listen to what I had to say and started a discussion after the presentation. Later he asked me if I was prepared to have the cards published by the ISRM. He enlisted Prof. Resat Ulusay who has been producing special ISRM publications on characterisation and testing of rock. The most recent development in this direction is that Prof. Ulusay would like to publish this for 2012 to accompany the latest special publication on testing. The favourable reaction to the pocket card concept and those already designed has resulted in a third paper on pocket cards for the following EUROCK 2010 symposium in Lausanne (see later on in this article).


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EUROCK 2010

Since EUROCK 2010 was held in Lausanne it could easily be reached by plane to Geneva and from there directly by train to Lausanne. Hence there was sufficient incentive to participate. The theme for EUROCK 2010 was *Rock Mechanics in Civil and Environmental Engineering* and the Netherlands' contribution consisted of two papers:

- 'Pocket cards for analyzing slopes and quarry rock size by stereo and Cartesian graphics' by P.M. Maurenbrecher and D.J.M. Ngan-Tillard.
- 'Data integration to assess geo-hazards in Maastricht A2 tunnel' by D.J.M. Ngan-Tillard, P. Kouwenberg, P.M. Maurenbrecher, and B. Vink.

Much of the symposium highlighted the recent completion of the base railway tunnels through the Alps through posters and presentations. Besides tunnels, dams, and associated hydraulic structures, for which Swiss engineers are renown, more specifically the effects of cavitation, water hammer, and water jet impingement on rock were treated. The latter processes can cause significant damage, dislodging rock blocks and inducing slope instability. An excellent presentation by E.F.R. Bollaert on his PhD research at EPFL (*École Polytechnique Fédérale de Lausanne*), the venue where EUROCK 2010 was held, showed a model in which the dislodgement/scour of rock blocks in the plunge pool at the toe of a dam is simulated. One of the papers ('Three dimensional mixed continuum-discontinuum numerical simulation of the Beauregard Landslide') presented by Mark Diederichs, a Canadian researcher from the Department of Geological Sciences and Geological Engineering at Queen's University (Kingston, Ontario), dealt with the Beauregard Landslide. This landslide impinges on the left abutment of a large concrete arch-gravity dam in the Valgrisenche valley which drains towards the NNW into the upper Aosta Valley in north-west Italy. The dam is situated in the High Alps with Mont Blanc 15 km north-west, and the Gran Paradiso 10 km east with steep glacier valleys and large, relatively dormant landslides. The dam never went into operation as during filling of the reservoir in 1969, the land started moving and displacements were measured on the dam. The Vajont and Malpasset dam disasters were still very fresh in every dam engineer's memory. During the discussion afterwards I asked out of interest if they had used aerial photographs to look for landslide features during the initial feasibility study. Prof. Barla, one of the co-authors, said that the authors couldn't answer the question as the study didn't cover this. The field excursion up the Rhone valley the following day in a way offered an answer to my question. Besides, it definitely offered lots of opportunities for contributions to the next IAEG Congress of 2014. One had to get up earlier than the three days of presentations to catch the metro to EPFL as there we were expected to board the bus for the 'Technical Visit'. At the start of the Rhone valley we made a coffee stop, after already having made a few short stops associated with debris flows and villages on the move, as their homes and vineyards moved downhill. Usually the authorities compensate the owners by purchasing their properties. These are then sold again to a highest bidder with the knowledge that eviction without compensation may be necessary if further movement renders them dangerous to live in. Prominent on the south flanks of the valley were pipelines and tunnels transferring water from upper dammed reservoirs to power stations at the base. These high pressure conduits sometimes leak, thereby causing major landslides. After Sierre, the A9 motorway reverts back to the original two lane trunk road and only reverts back to a motorway just beyond Visp, near Brig. This is because the A9 was planned through tunnels which transpired to be in large massive landslips that only became apparent when movement was detected as tunnelling progressed. Remote sensing by LIDAR revealed the whole mountain slope to be part of a landslip not apparent on earlier aerial photos due to the thick forest cover. A few kilometres west of Visp at the new Lötschenberg base tunnel, high-speed trains exit the tunnel after going underground just east of Bern. During its construction, water ingress occurred at a section beneath the village of Albinen, a few kilometres into the tunnel. The village pond drained and chalets subsided. Upland, peat on which the chalets were founded consolidated causing subsidence. Similar to the villages on the move further down the valley, the owners were compensated by the tunnel consortium. Before Visp, the tour bus entered the tunnel leading into the Visp valley to the major resorts of Zermatt and Saasfee. As we were on a technical visit we did not go as far up the valleys as most tourists do to go skiing and mountaineering or just take the trains and cable cars to look at the impressive scenery which awaits you at both these resorts. This excursion highlighted features probably oblivious to the majority of tourists (including this excursionist up till then) and in its way equally impressive. Firstly, a large debris flow/rock fall on the east flank of the Visp valley best viewed from Stalden, followed by a rather dramatic rock fall at St. Niklaus a few kilometres up the road along the Matter-Visp. On the Internet it probably looks more dramatic, as an animation shows blocks larger than the size of a house free falling down the slope. An earth embankment 'dam' acts as a trap to protect the road and railway below. Then the main course appeared as the Randa rockslide. As a tourist to Zermatt I had for many years had a passing interest in the large pile of rock blocks from massive boulder to cobble sizes and presumed that some sort of mining activity had taken place as there appeared to be an adit on the north side of the huge spoil heap. But that turned out to be wrong. The slide explains more features seen along the Matter valley, such as the large level area at Täsch sufficient to offer space for parking cars and a railway terminus/station. As the level space reaches its northern downstream limit the landscape becomes hummocky offering this time room for a newly constructed nine-hole golf course, purportedly the highest elevation for a golf course in the Alps, advertised on a billboard when driving south from the village of Randa. According to our guide the level areas found in the valleys are the result of ancient slides blocking the river causing lakes to form and depositing sediment to create flat lake beds. Eventually the rockslide barrier erodes and the river cuts a new channel in the bed of the drained lake. The Randa slide occurred in 1991 and gradually took place as rock blocks were heard to continually fall at short intervals for a period of 24 hours. As it was misty it was difficult to see what was happening. When visibility returned the river was damming up behind the slide debris and had already flooded a barn causing several cows to drown (the farmer was not allowed to rescue them because of visibility). The animals were the only casualties. The first action was to dig a tunnel through the west bank of the river beneath the slide so the river could be diverted through it and prevent the water from rising further as it would threaten the railway and road. We stopped for lunch at a chalet hotel with a good view of the slide from its terrace. Possibly wine tasting was what was missing on the excursion, as we had passed through the main wine district of Switzerland which produces excellent *Fendants* and *Johannisberger* white wines. After EUROCK 2010, Switzerland can never be the same again for me. I always found the scenery dramatic. But there is now for me (and my fellow excursionists) real drama in these mountains, and the Beauregard Dam at Valgrisenche could also do with a bit more drama. As one follows the border from the Matterhorn (shared with Italy, known there as Monte Chervino) to the east, then follow the border from the Mont Dolent (shared by Switzerland, Italy and France) south-west to Mont Blanc, and then continue along the border south-east to near the French ski resort of Tignes/Val d'Isère one comes to within a short few kilometres from the Beauregard dam just across the nearby border. The Times Atlas of the World published in 1967 shows a natural lake just upstream from Valgrisenche. Back across the border into France, the atlas shows a similar lake labelled as Tignes dam. Could it be that the original lake in Italy was caused by the slide on which the Beauregard dam was later founded, the purpose of the dam to increase the capacity of the lake? I hope the 12th IAEG Congress in 2014 in Turin will compliment the EUROCK 2010 excursion up the Rhone valley with one up the Aosta valley. See you there!

Book review
Geological Engineering

*Reviewed by **Floris Schokking** (GeoConsult B.V., Haarlem, the Netherlands)*

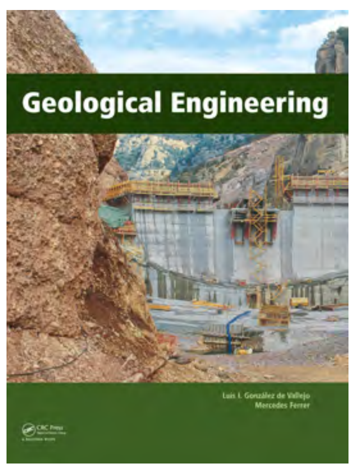
Geological Engineering is written by Luis González de Vallejo and Mercedes Ferrer in collaboration with Spanish, Portuguese and English scientists in geotechnics, engineering geology, and hydrogeology. The book relies heavily on the knowledge and experience of Luis González de Vallejo, researcher and professor at Complutense University in Madrid and consultant to large infrastructural projects in Spain and Central- and South America. *Geological Engineering* is a translated and revised edition of the original work in Spanish.

The title *Geological Engineering* is eye catching in the sense that the term is seldomly used in Europe as a descriptive name of an academic curriculum in the applied earth sciences. Here, the term appears to have been used on purpose to express the activity of the application of concepts and methods from engineering geology, soil and rock mechanics, and hydrogeology in design and construction for civil engineering. This vision is also expressed in the structure of the book of which the first two parts constitute the components and tools used for research, design, and construction, as elucidated in the two subsequent parts.

Never earlier appeared a book of engineering geology and geotechnical engineering in which the various subjects and aspects are treated in such a clear and distinct way. By well justified schematisations, without the loss of important details, virtually all relevant aspects of geological engineering are presented. In separate text blocks examples are given of geological concepts and engineering routines, with cases, worked out calculations and charts which are used in engineering design. Many examples that demonstrate how geological conditions and processes influence the stability and safety of constructions are given. As such is the deformational behaviour of soil or rock under influence of forces resulting from constructions or excavations related to historical and recent geological processes. The main line throughout the book with respect to this aspect is how the geological features are related to the problems which are to be solved by the engineer, without touching the fundamentals of the geological concepts in play. That reality is often more complex than one would wish, and which the ready-for-use offered chunks seem to suggest on first sight, is not disregarded.

Ample attention is given to in-depth literature and referral to standard works of the state-of-the-art of the disciplines involved, magazines, recent results of international (multidisciplinary) working groups and a variety of other useful sources of information. As a result of the structure of the book and treatment of the most relevant subjects in geological engineering, both in soil mechanics and rock mechanics, the book may be of interest to a broad range of users. It forms an ideal text book for students in engineering geology and geotechnical engineering, in both the Bachelor and Master phase. But also practising engineering geologists, geotechnical engineers, and consultants may use it effectively. The clear and schematised manner in which the subjects are treated may make the book even suitable for those who have a less direct involvement in geological engineering, such as managers and decision makers, but to whom the understanding of concepts and principles are of importance.

With *Geological Engineering* Luis González de Vallejo and Mercedes Ferrer have succeeded in offering a standard work, of interest to a wide group of users, with subjects in a broad range of geologically differing areas. It can serve as a support in the application of geology to research, design, and construction in civil engineering.



Geological Engineering

Luis I. González de Vallejo & Mercedes Ferrer

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Dredging rock

***Dr. Peter Verhoef** (Senior Engineering Geologist, Royal Boskalis Westminster nv, Papendrecht, the Netherlands)*

Figure 1 CSD Ursa *(CSD* Taurus *in background) at Ras Laffan (Qatar), 2003.*

Introduction

One of the attractive aspects of working for a worldwide operating dredging contractor is the variety of project conditions that are encountered. Many projects involve dredging of rock, which implies the use of special equipment and excavation methods. The interpretation of soil conditions at a project site is an important decision factor for the choice of dredging equipment. If a mistake is made and the wrong equipment is chosen, excavation of soil or rock may turn out to be a difficult and costly undertaking. When a tender for a new dredging project comes in and rock is expected in the dredge area, common questions related to rock are:

- What is the volume of the rock?
- Can the rock be excavated directly by a hopper dredger or a backhoe dredger or do we need a heavy cutter suction dredger?
- Is pre-treatment of the rock by drilling and blasting needed before we can dredge it?

Once these initial questions are answered more detailed work has to be done to estimate production rates and tool consumption (due to wear and tear) for the equipment selected for the project. The main objective of site investigations for dredging projects is to assemble all information needed to answer these questions (among others) and to make a proper selection of dredging equipment. Unfortunately it is the rule rather than the exception that important information is lacking in site investigation reports for dredging works. Important and often decisive parameters are commonly not determined or not in sufficient quantity. A lot can be improved in the current practice of site investigations for dredging works. To illustrate the importance of typical geotechnical parameters used for rock, this paper describes the rock mechanical aspects involved in the dredging of rock. If any rock or cemented soil is present or expected, a specific approach of the site investigation is needed. The direct excavation of rock by mechanical equipment is treated using the concept of *rock-tool interaction*. The cutting mechanisms in massive rock are discussed and it is shown that the simple rock strength index tests UCS (Unconfined Compressive Strength) and BTS (Brazilian Tensile Strength) are very useful parameters to describe these mechanisms. When rock is directly excavated using mechanical means and the rock slurry is transported hydraulically through pipelines using pumps, then the wear of cutting teeth, pump impellers and pipes is an important costing factor. To be able to judge the amount of wear expected, it is helpful to be aware of the fundamental wear processes that can occur, *abrasive* wear and *adhesive* wear, and to know the rock parameters that are related to these. Rock strength parameters (UCS and BTS) and the mineralogical composition of the rock material (mineral- and rock hardness) are important geotechnical parameters relevant to wear and tear.

The dredging process

Several stages of the dredging process can be distinguished, each stage requires specific knowledge of soil or rock properties:

- Excavation: loosening, fragmentation, or cutting of rock or soil.
- Lifting: raising the excavated material to the surface
- Transport using hydraulic or mechanical methods.
- Deposition of the material in a dump or reclamation area.

Excavation and lifting of the excavated slurry can be done with the following equipment:

- Trailing suction hopper dredger (TSHD)
- Cutter suction dredger (CSD)
- Backhoe dredger (BHD)
- Grab dredger

Table 1 Classification of various dredging equipment.

Trailing Suction Hopper Dredgers (TSHD)			
Size	**Classification**	**Hopper capacity [m³]**	**Examples**
XXL	Mega	>30,000	Cristóbal Colón; Queen of the Netherlands; Vox Máxima
XL	Jumbo	10,000-30,000	Congo River; Gerardus Mercator; Oranje; HAM 310
L	Large	7,000-10,000	Geopotes 15; Breydel; Alexander von Humboldt; Barent Zanen
M	Medium	3,000-7,000	Volvox Iberia; Coastway; Francesco di Giorgio; Mellina
S	Small	<3,000	Amazone; Coronaut; Vlaanderen XXI
Cutter Suction Dredgers (CSD)			
Size	**Classification**	**Total installed power [kW]**	**Examples**
XXL	Mega	>20,000	D'Artagnan; J.F.J. de Nul; Athena
XL	Jumbo	12,500-20,000	Texas; Marco Polo; Phoenix; Castor
L	Large	7,500-12,500	Vlaanderen XIX; Hector; Edax; Hondius
M	Medium	2,000-7,500	Amstel; Ortelius; Merwede; Kalis II
S	Small	<2,000	HAM 250; Ceres; Petrus Plancius; Seckin
Backhoe Dredgers (BHD)			
Size	**Classification**	**Total installed power [kW]**	**Examples**
XXL	Mega	>3,000	Simson; Samson; Wodan; Vitruvius
XL	Jumbo	1,500-3,000	Baldur; New York; Pinocchio; Il Principe
L	Large	700-1,500	Razende Bol; Maricavor; Jerommeke; Delilah
M	Medium	350-700	Dinopotes; Zeeolifant; Obelix; Durme
S	Small	<350	Goodwin Sand; Captain A.J. Fournier

Other dredgers used in dredging works are the stationary suction dredger and the bucket ladder dredger. When rock cannot be excavated directly with mechanical excavators, pre-treatment by drilling and blasting is required. After fragmentation by blasting, the fragmented rock can be dredged by a TSHD or a BHD. Transport of excavated material can take place in various ways. A TSHD can transport the slurry, a soil-water mixture, to the disposal area by itself. A CSD commonly transports the material hydraulically as slurry through pumps and pipes. The available pump capacity and the length of the pipelines needed are important considerations for a project. Another possibility using a CSD is to load the excavated material in barges that moor up alongside the dredger. Barges are commonly used when excavation takes place with either a backhoe or a grab crane and when offshore dump sites are used. TSHD's have several possibilities to dispose of the soil. Dumping can occur by opening doors in the bottom of the dredger, by pumping the slurry ashore through a floating pipeline, or by spraying the slurry on the reclamation area (*rainbowing*).

The choice of dredging equipment is mainly guided by:

- Rock and soil properties in the dredging area.
- Workability.
- Transport distance to deposit areas.

Workability entails environmental working conditions such as water depth, current, wave heights and swell conditions, and sediment transport. Before a dredging project commences, special studies are carried out to establish the

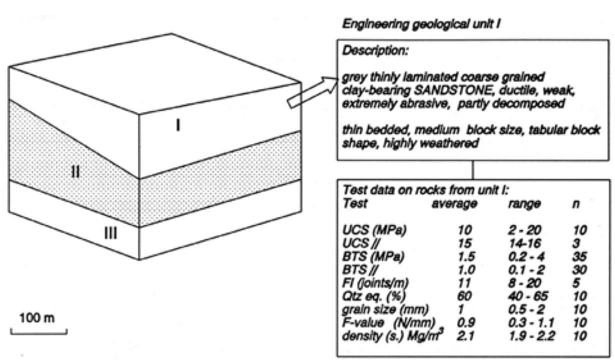

Figure 1 Geotechnical model with engineering geological units (Verhoef, 1997).

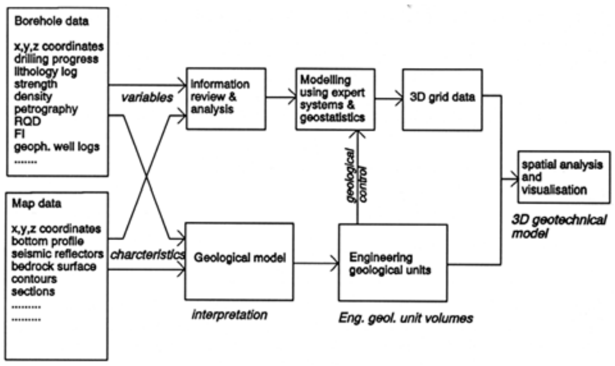

Figure 2 Flow diagram for the processing of data to arrive at a geotechnical model for the dredging of rock (Verhoef, 1997).

workability conditions at the site (expected weather conditions during the time period that the work is to take place, expected wind velocities and wind directions, tidal currents, and related current rates). In this way the amount of downtime for a certain type of dredger can be determined. Depending on weather, tide, and swell, time windows can be established within which the work can be done. TSHD's are the only types of dredgers that may work in open seas. CSD's are limited by the length of the spud poles and the ladder and the degree of swell which can be allowed. The other dredger types also have limiting working conditions that depend on water depth and swell conditions. By using swell compensation and dynamic positioning systems in several dredger types one tries to increase the workability range. There is a trend to increase the capacity of especially TSHD's, CSD's, and BHD's. The ability of dredgers to excavate rock has improved by increasing the size and power of the equipment (see Table 1).

Evaluation of soil conditions

The purpose of site investigations for dredging works is:

- To determine the volume of rock and soil to be excavated.
- To determine the 3D distribution of soil and rock units (*engineering geological* or *geotechnical* units, see Figure 1).
- To assess the geotechnical properties of the engineering geological units.

To realise these goals it is necessary to assemble sufficient data. The most efficient way to process the data is by using GIS systems. This implies that data is processed in digital format. Most common types of data used are bathymetry of the seabed (e.g. in ASCII format); position and depth of boreholes and elevations of the distinguished ground units; coordinates of surfaces from geophysical surveys such as reflection, refraction, or geo-resistivity surveys; and results of in situ and laboratory tests with known spatial coordinates (depth and position of test location or sample). See Figure 2 for a flow diagram. It is surprising to find that even today most of the site investigation data supplied with tender documents is not intended for processing by the contractor. Presentation of the results of site investigations merely in the form of reports with maps and drawn sections is insufficient. The client and designing engineer should realise that the contractor has to make his own interpretation of the information provided. He will interpret the data based on the experience of past works and in the near future wants to carry out the work. It is therefore obvious that the site investigation information should be presented in a digital format that allows handling and processing of the ground investigation data by the contractor.

Rock characteristics

Nearly all natural rock masses near the Earth's surface are transected by fractures. These fractures are a key element in the concept of *rock mass*. The term fracture is used as a general term for open fractures in rock. In rock mechanics the term *discontinuity* or *defect* is used for fractures to avoid confusion with the restricted definition for fracture in geology (in geology a fracture is a discontinuity over which displacement has occurred parallel to the discontinuity plane; a joint is a discontinuity without obvious displacement parallel to the discontinuity plane). *Rock mass* is defined here as *a volume of rock on a scale larger than the excavation equipment*. The mechanical properties of a rock mass are mainly determined by the spacing and geometry of the fractures and the strength and ductility of the rock material. The interaction diagram of Figure 3 shows that the capacity of the excavation tool and the strength properties of the rock mass determine whether the tool can excavate the rock. For the rock this implies that it should be characterised in such a way that a useful choice of the excavation equipment from Table 1 is possible. Obviously we think of parameters that describe the rock material strength (Unconfined Compres-

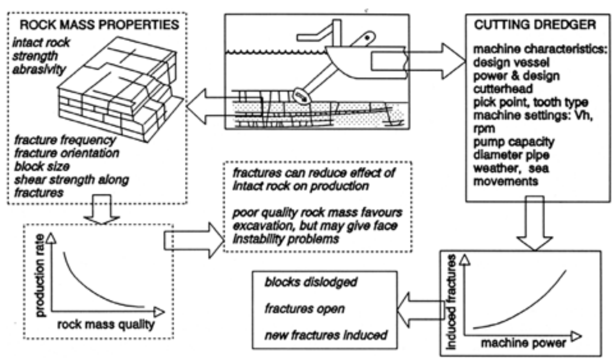

Figure 3 Interaction between rock mass and excavation tool (Verhoef, 1997).

sive Strength (UCS), Brazilian Tensile Strength (BTS), and Point Load Strength (PLS or $I_{s(50)}$)) and fracture geometry (Rock Quality Designation (RQD), Fracture Index (FI), and block size). In order to build a database of excavation performance of dredgers, we have to use the information of dredging projects in which both soil properties as well as dredging performance (production rates, tool consumption) are well recorded. For a number of reasons such data is often not easily accessible. The most important reason is that contractors consider such data as confidential in-house information. But in addition, it is also not that easy to register properties of rock being excavated under water. Excavation works on land do not have this restriction. There is a reasonable amount of publications of excavation performance in rock of tunnel boring machines, rock trenchers, and bulldozer rippers (Alvarez Grima (2000), Alvarez Grima et al. (2000), Alvarez Grima & Verhoef (1999), Deketh (1995), Deketh et al. (1996), and Verhoef (1997)).

Assessment of excavation method based on rock properties

The rippability chart of Pettifer & Fookes (1994) is a starting aid for the choice of suitable excavation equipment (Figure 4). The fields in the diagram indicate what type of bulldozer can be used in a rock mass characterised by Point Load Strength ($I_{s(50)}$) and average discontinuity spacing If. (If = (S1 + S2 + S3)/3 ≈ 1/FI; S1, S2, S3,... = average of spacing discontinuity sets 1, 2, 3,...; FI = fracture index = number of discontinuities per metre of drilled core). In the upper right corner of the diagram the massive and strong rock is plotted, the bottom left plots the weak and strongly fissured rock. From the lower left to the upper right the rock increasingly demands more of the excavation tool. In the rippable area, the capacity of Caterpillar bulldozers increases from D6/D7 to D9/D11. It is important to understand that the boundaries of equipment for 'rippable' and 'not rippable' (which implies excavation in relatively strong, massive rock) depend on the cutting mechanism and the spacing of the discontinuities in the rock. Figure 4 is valid for bulldozers with one ripper tooth. Each D-type bulldozer has a larger capacity and is able to exert a higher cutting force on the ripper tooth and is therefore able to handle stronger rock. For each type of equipment such a diagram may be constructed, provided enough cases can be collected. From the bottom left to the upper right, heavier equipment will be needed. Looking at the dredging equipment of Table 1, the backhoe and the hopper dredger resemble the bulldozer ripper when we consider the cutting mechanism used. These also have a linear cutting system using drag teeth. A first guess is that BHD's and TSHD's with rock dragheads of

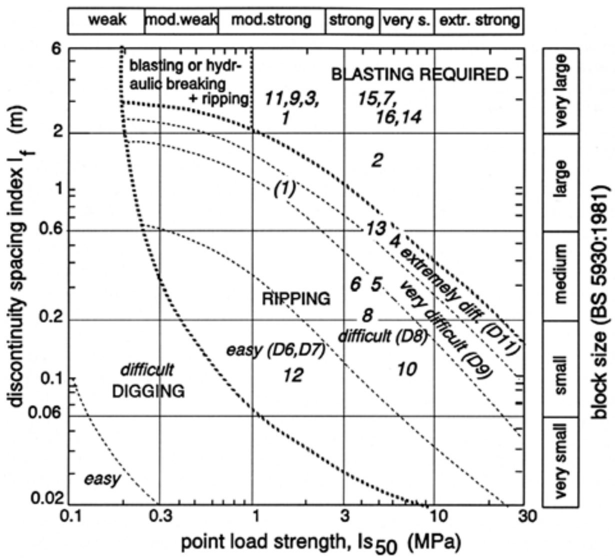

Figure 4 Rippability chart of Pettifer & Fookes (1994). This chart is based on 120 cases of ripping with Caterpillar bulldozers. The numbers in the plot refer to projects with Vermeer T850 trenchers, which are reported in Verhoef (1997).

size L and larger may work in the rippable area (Figure 5). The precise boundaries for these tools should become available when sufficient dredging projects are plotted in the diagram. CSD's of size L and larger are regularly used to cut rock. Cutter diameters of 3 m or more have about 50 teeth or pick points. These cut slices of maximum 250 mm thick and commonly only half of that thickness. This means that a rock mass with an average discontinuity spacing of 250 mm could be experienced by the teeth as being 'massive'. They are forced to cut the rock material. The strength of massive rock is mainly determined by the compressive strength (UCS). From practice it is known that the upper boundary for M-size cutters lies at a UCS of 5 MPa, for XL dredgers in the order of 30 MPa, and for XXL possibly 40 MPa.

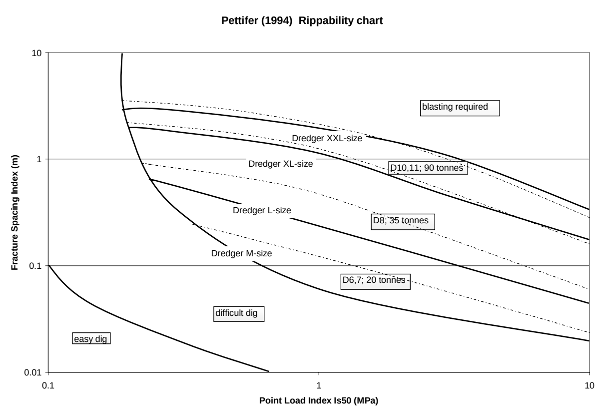

Figure 5 Pettifer diagram with conceptual boundaries for backhoe and hopper dredgers excavating rock. Boundaries have been chosen arbitrarily.

Cutting of massive rock

Dredging contractors use empirical rules to calculate production rates and tool consumption. This is due to the fact that the interaction process between tool and rock is complicated and variable. Cutting theories based on first principles are not successful in predicting cutting performance. Cutting forces needed to excavate rock depend on two important factors: the design and power of the cutting equipment and the strength of the rock. Because the design and power of the cutting equipment varies with type of equipment, the relationship between strength of rocks as determined in the laboratory and the cutting production of a tool is expressed in 'the amount of work done', which is the *specific energy*. We assume here that the work done in the laboratory to crush a rock core (in a strength test) or to cut a rock specimen is related to the work needed to cut the same rock by the draghead of a TSHD or the cutter of a CSD. This implies that for each type of equipment we have to establish the relationship of the specific energy based on laboratory testing with the specific energy determined for the equipment.

Cutting forces

From linear cutting tests performed in the laboratory, relationships have been derived which calculate the cutting force needed to cut intact rock. Three types of failure of rock can be distinguished: brittle failure, ductile failure, and a mixed mode transition between these two. These failure types can be recognised in practice. Figure 6 shows how we can estimate the failure envelope describing rock strength from the index tests UCS and BTS. From Figure 6 we can see that the ratio UCS/BTS (m) describes the directional coefficient of the failure envelope (and is therefore related to the angle of internal friction φ of the rock). From rock cutting practice (tunnelling, mining, and dredging) it is known that for a ratio m>15 the rock cuts brittle and for m<9, ductile.

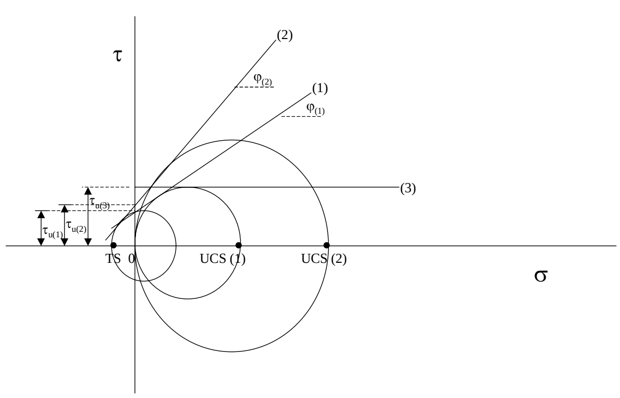

Figure 6 Failure envelope derived from the results of UCS and BTS tests. The ratio m (UCS/BTS) of rock (1)=5, of rock (2)=10. The failure envelope (3) is for porous rock tested under undrained conditions.

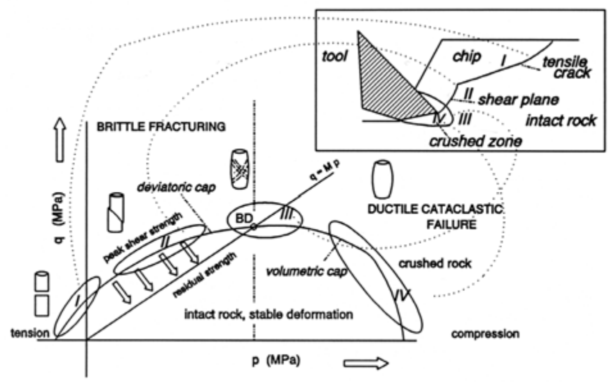

Figure 7 Model for the cutting of drag cutting tools into rock (Verhoef, 1997).

Brittle rock produces fractures and during cutting, rock chips are formed. Therefore, the specific energy for cutting is relatively low. Ductile rock produces much smaller chips. Sometimes only grooves with the shape and size of the cutting tooth or bit are made in the rock. The specific energy needed to cut is relatively high (low production). CSD's and TSHD's use pick points or chisels to cut the rock. Studies carried out at Delft Hydraulics Laboratory show that a whole range of failure mechanisms known from rock mechanics occur during the cutting process (Figure 7):

- Under the tooth the very high compressive stress leads to crushing of the rock. Away from the crushed rock zone, shear and tensile fractures develop in brittle rock. These develop into chips.
- Ductile rock can be seen as a material that is completely crushed during cutting. The tooth is always in contact with crushed rock material.

For *brittle* rock the cutting theory of Evans can be used to calculate cutting forces (Figure 8). The forces are derived from the geometry of the chisel (width, cutting angle, and cutting depth) and the tensile strength (BTS) of the rock.

$$F_c = \frac{2\sigma_t dW \sin\frac{1}{2}(90-\alpha+\theta^{r,t})}{1-\sin\frac{1}{2}(90-\alpha+\theta^{r,t})}$$

$$F_n = \frac{F_c}{2\tan(90-\alpha+\theta^{r,t})} \qquad \text{(Eq.1)}$$

where σ_t = tensile strength; D = cutting depth; W = width of chisel; α = rake angle; $\theta^{r,t}$ = angle of friction between rock and tool. Values for $\theta^{r,t}$ can be found in Verhoef (1997).

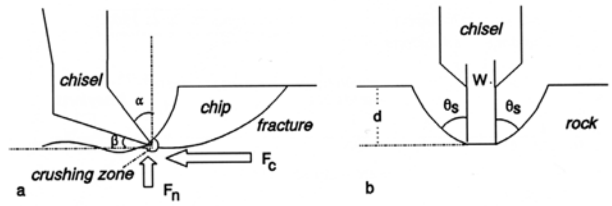

Figure 8 a) Evans' model for the cutting of rock with drag cutting tools; b) This figure shows the outbreak geometry of chips with characteristic outbreak angle θ_s (Verhoef, 1997).

For *ductile* rock we may use the equation of Nishimatsu. This equation describes the cutting force of chisels by failure through shear. Figure 9 gives the parameters needed to calculate the cutting forces. Using the UCS and BTS value the shear strength τ_u can be calculated.

$$F = \frac{2\tau_u dW\cos\vartheta}{(n+1)(1-\sin(\theta^{r,t}+\vartheta-\alpha))}$$

$$F_c = F\cos(\theta^{r,t}-\alpha)$$

$$F_n - F\sin(\theta^{r,t}-\alpha) \qquad \text{(Eq. 2)}$$

where τ_u = rock cohesion (shear strength); φ = angle of internal friction; d = cutting depth; W = width of chisel; α = rake angle; $\theta^{r,t}$ = angle of friction between tool and rock. n is a stress distribution factor determined by linear regression analysis as n = 1.2-0.02α.

The linear failure equation from Figure 9b,

$$\tau = \tau_u + \sigma\tan\vartheta \qquad \text{(Eq. 3)}$$

can be determined from the rock's UCS and BTS value (Vlasblom, 2003):

$$\tau = \frac{\sigma_c}{2\sqrt{m-3}} + \frac{m-4}{2\sqrt{m-3}}\sigma \qquad \text{(Eq. 4)}$$

where τ = shear strength at failure; σ = normal stress at failure, σ_c = UCS and m = UCS/BTS.

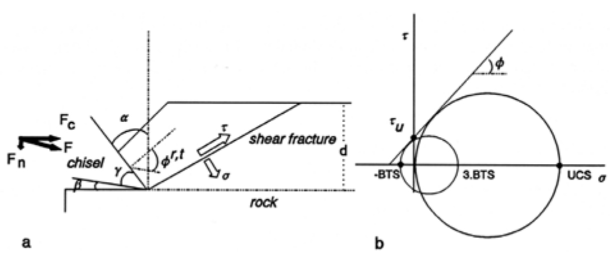

Figure 9 a) Model for shear failure by Nishimatsu; b) Method to calculate τ_u using UCS and BTS (Verhoef, 1997).

Concluding, it can be said that for an approximation of the cutting forces needed we can use the index tests UCS and BTS and the ratio m = UCS/BTS. If the rock is brittle (m>15), failure occurs through tensile fractures and BTS is directly proportional to the cutting force (Evans' equation). If the rock is ductile (m<9) the equation of Nishimatsu can be used using UCS and m.

Specific energy and production

The best way to use laboratory test results for practice is to calculate the specific energy, the work needed to cut a volume of rock, and compare this with the work needed to excavate the same rock with the dredger. The specific energy (SPE) is the work needed to cut 1 m^3 of rock. This is equal to the capacity or power P of the dredging tool to cut 1 m^3 of rock per second. In the cutting equations (Eq. 1 and 2) the volume cut is given by the factor dW (per unit of cutting distance). The power of the cutting tool is expressed as P = Fc.v (where v is the cutting velocity), and the production as Q = dWv, so the production is inversely related to the specific energy (Vlasblom, 2003):

$$SPE = \frac{P}{Q} = \frac{F_c v}{dWv} = \frac{F_c}{dW} \qquad \text{(Eq. 5)}$$

This implies that the specific energy for the cutting of rock is directly related to rock strength and can be expressed using the rock parameters UCS and BTS. For brittle rock with m>15, the SPE can be calculated by dividing the cutting force by the volume of rock that breaks out. This volume consists of dW per unit of length plus the volume of chips breaking out; see Figure 8b, where θ_s is the outbreak angle.

From Evans (Eq. 1) follows:

$$SPE = \frac{2\sigma_t \frac{1}{2}(90-\alpha+\theta^{r,t})}{1-\sin\frac{1}{2}(90-\alpha+\theta^{r,t})}\frac{1}{1+\frac{d}{W}\tan\theta_s} \qquad \text{(Eq. 6)}$$

For ductile rock with m<9, which has no break out of chips due to the ductile cutting process, the volume only consists of material removed by the chisel. From Nishimatsu (Eq. 2) follows:

$$SPE = \frac{2}{n+1}\frac{\tau_u\cos\vartheta\cos(\theta^{r,t}-\alpha)}{1-\sin(\vartheta+\theta^{r,t}-\alpha)} \qquad \text{(Eq. 7)}$$

For rocks in the brittle-ductile transition range of 9<m<15, both Eq. 6 and Eq. 7 can be used to estimate specific energy.

The cutting equations used are based on tests that have been performed under laboratory conditions with sharp chisels on a scale that is an order of magnitude smaller than the chisels which are used in dredging. Blunt chisels result in an increase of the cutting forces and lead to a doubling or more of the specific energy. In practice, chisels or pick points are replaced once worn out. For production calculations it is necessary to have an idea of the rate at which chisels wear down and become blunt. Before we deal with this subject, we will first look at the effect of discontinuities, when these occur within the zone of influence of the cutting equipment.

Cutting discontinuous rock

When discontinuities occur within the zone of influence of the cutting tool, the cutting forces are generally decreasing. There is a transition from cutting rock material to loosening of rock blocks from the rock mass (which is called *ripping*). The in situ block size is going to determine the excavation process. The cutting equations that we use for massive rock are not valid anymore. Using numerical modelling, the cutting process of discontinuous materials can be described, but this has little practical significance for dredging rock at the moment. The transition of 'cutting' to 'ripping' excavation can give problems when the block size of the rock mass is of the order of the opening of the pump passage of the hopper or cutter dredger. In general the productions are significantly higher when ripping excavation occurs. Figure 10 illustrates the transition of cutting to ripping for a rock cutting trencher. This figure has also been chosen to illustrate the importance of this transition for the wear rate occurring during excavation.

Wear

Figure 10 shows two extreme regions of excavation in rock: excavation in *massive* rock and excavation in *broken* rock. In these extreme areas completely different mechanisms take place: either cutting with chisels into rock or the loosening of rock blocks (ripping). In practice it is found that wear of teeth is much higher when cutting and that during ripping the main cause of tool consumption is breakage of teeth (and only when the rock has a high compressive strength).

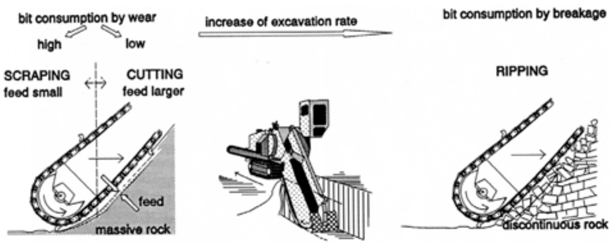

Figure 10 Transition from cutting to ripping for a rock cutting trencher (Verhoef, 1997).

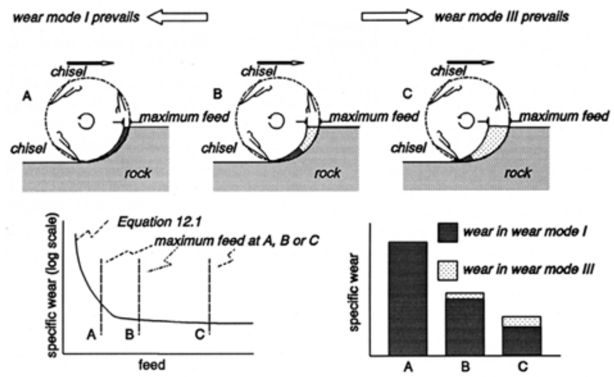

Figure 11 Cutter wear model of Deketh (Verhoef, 1997).

There are two main processes that occur during wear:

- Wear due to the difference in hardness of the mineral substances of the rock and steel: abrasive or erosive wear.
- Wear due to the development of heat on the contact surface of the wearing part (steel) and the rock. If the heat is so high that weakening of the steel occurs, than adhesive wear takes place.

Deketh (1995) distinguishes between three important phases during the penetration of a cutting tool into rock. The model is illustrated in Figure 11, which shows the cutting of the teeth of a CSD. When the rock is too strong for the cutter (the CSD has not enough power) the teeth cannot penetrate and sliding two-body abrasive wear or even adhesive wear occurs. Deketh (1995) calls this mode I wear; situation A in Figure 11. When the rock is ductile (m = UCS/BTS<9) and the UCS is relatively high for the dredger used (for example UCS>30 MPa while using a XL-size dredger, see Table 1), then even in a non-abrasive rock such as limestone significant wear can take place. This is due to adhesive wear with contact temperatures between tool and rock higher than about 700 ˚C, the weakening temperature of hardened steel. When the tooth is able to penetrate deeper into the rock, the crushed rock zone forms (Figure 7). The crushed rock between cutting tool and solid rock has a reducing effect on the wear rate. The process is now a three-body-type of abrasive wear (mode III). The wear rate is significantly lower than during the penetration phase of the cutting process; situation C of Figure 11. Mode II is the transition of mode I to mode III. In mode II the contribution of adhesive wear and two-body abrasive wear to the total wear will decrease (situation B of Figure 11). Due to the cyclic cutting process occurring with a cutter head, at each rotation of the cutter the unfavourable penetration phase with mode I wear occurs. During penetration in mode I, by far the highest wear takes place (Figure 11, see histogram bottom right). The cycloidal penetration of cutting teeth of a cutter is unfavourable from the point of view of teeth consumption. If we compare this with the linear cutting process that takes place while dredging with the dragheads of a TSHD, we see that tool consumption is much lower. The chisels penetrate the rock and can cut for a relatively long linear stretch and the contribution of mode I wear to the total is much lower (provided that the tooth has been able to penetrate the rock deep enough). Figure 12 shows the results of tests by Deketh (1995) to study the penetration of chisels into rock. Increasing UCS gives a proportional increase of the wear. Wear rates appear to be linearly related to the UCS or BTS of rock. Wear factor F, for example, has a good correlation with tool consumption due to wear in mining, tunnelling, trenching, and dredging operations.

$$F = \frac{Qtz_{eq}.\phi.BTS}{100} \quad \text{(N/mm)} \qquad \text{(Eq. 8)}$$

where Qtz_{eq} is the total mineral hardness with respect to quartz (%) and Φ is the grain size of the abrasive minerals (mm). Wear due to cutting is apparently a function of the strength properties of rock (UCS, BTS, and m), but also of the rock's mineral composition, which determines the hardness of the rock. To determine the degree of wear, it is necessary to know the difference in hardness of the rock and the steel of cutting tools, pumps, and pipelines (Table 2). Therefore we need to know the mineral composition of the rock. The relative hardness of minerals is determined with respect to the hardness of quartz. The Equivalent Quartz Hardness (Qtz_{eq}) is obtained by summing the relative hardness's of the minerals proportionally. Significant wear occurs if Ha/Hs>1 (Ha = hardness abrasive rock; Hs = hardness steel). Concluding, wear is a system dependent process that is difficult to predict. Dredging contractors use empirical models to estimate wear.

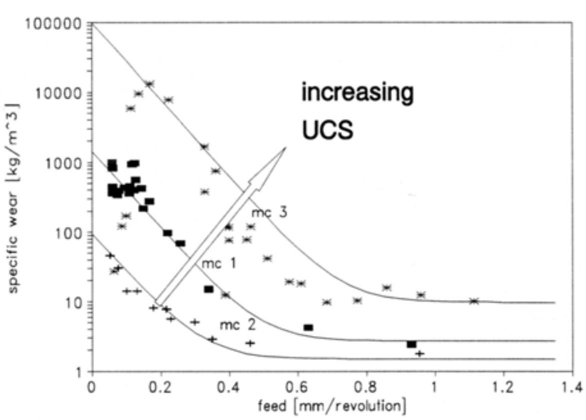

Figure 12 Specific wear against cutting depth (feed) of chisels cutting in artificial quartz concrete with a UCS of 18 MPa (mc 2), 30 MPa (mc 1) and 68 MPa(mc 3); cutting velocity of all tests 0.4 m/s (Deketh 1995).

Table 2 Hardness of minerals and steel. Vickers Hardness is used to calculate the Equivalent Quartz Hardness (Verhoef, 1997).

Mineral	Mohs [-]	HV* [MPa]	RHV** [-]
Talc	1	220	0.02
Gypsum	2	500	0.05
Calcite	3	1,300	0.12
Fluorite	4	1,750	0.16
Apatite	5	5,200	0.47
Orthoclase	6	7,725	0.70
Quartz	7	11,075	1.00
Topaz	8	15,000	1.35
Corundum	9	22,000	1.99
Common minerals	**Mohs [-]**	**HV* [MPa]**	**RHV** [-]**
Feldspars	6	7,750	0.70
Clays and mica's	2.5	1,000	0.09
Carbonates	3	1,500	0.14
Calcite	3	1,300	0.12
Aragonite (shells)	3.5	2,900	0.26
Dolomite	3.8	4,000	0.36
Dredge materials	**Mohs [-]**	**HV* [MPa]**	**RHV** [-]**
Dredge teeth		6,000	0.54
Tungsten carbide		14,000	1.26
Pipe steel		2,000	0.18

**HV = Vickers Hardness; **RHV = Relative Vickers Hardness*

Weathered rock

In the introduction it was mentioned that rock is experienced as difficult when the equipment that is used does not have the capacity to excavate it. Apart from this, there are also typical situations that are difficult. Dredging often occurs in the weathering zone above solid rock. These zones are often very inhomogeneous of composition, resulting in a mix of fresh rock, partly weathered rock, and completely weathered rock to be dredged. The spatial distribution of these soil types is hard to predict. A common weathering product is clay. Presence of clay in the soil-rock mix can have unpleasant consequences during the different phases of the dredging process. Knowledge of the composition and strength properties of the different types of clay is therefore very important. When the Atterberg Limits, the natural moisture content, and the clay content are known, the properties and therefore the expected behaviour of clay during excavation, transport, and deposition can be estimated. The better the geological situation is known in areas underlain by weathered rock, the better the dredging equipment needed can be specified. It helps to use a suite of site investigation

Table 3 Basic parameters for dredging (based on Danson (2005), with additions).

Application	Clay	Silt, sand or gravel	Rock
Excavation (methods and production)	General description	General description	General description
	Grain size distribution	Grain size distribution	RQD
	Organic material content	Angularity	Water absorption
	Gas content	Carbonate content	Total unit weight
	Total unit weight	Maximum/minimum density	Unit weight of solid blocks
	Atterberg Limits	In situ tests: CPT/SPT	UCS
	Moisture content		BTS
	Undrained shear strength		Petrographic analysis
	In situ tests: CPT/SPT		
Transport (methods and production)	Organic material content	Grain size distribution	Unit weight of solid blocks
	Gas content	Maximum/minimum density	UCS
	Particle unit weight	Particle unit weight	BTS
	Atterberg Limits	Mineralogy	Mineralogy
	Moisture content	Angularity	Block size distribution
	Undrained shear strength		
	Viscosity parameters		
Abrasion with excavation and transport	Grain size distribution of coarse grained minor constituents	Grain size distribution	Unit weight of solid blocks
	Mineralogy of coarse grained minor constituents	Particle unit weight	UCS
		Mineralogy	Mineralogy
		Angularity	
Fill material	Clay is only used as fill material for special purposes; required parameters to be derived from special purpose slurries	Grain size distribution	RQD
		Angularity	Water absorption
		Organic material content	Unit weight of solid blocks
		Maximum/minimum density	Mineralogy
			Block size distribution
Dredged slope stability and settlement/ stability of future fill area	Grain size distribution	Grain size distribution	RQD
	Organic material content	Angularity	Water absorption
	Mineralogy	Carbonate content	Total unit weight
	Total unit weight	Organic material content	Unit weight of solid blocks
	Atterberg Limits	Maximum/minimum density	UCS
	Moisture content		BTS
	Undrained shear strength		Mineralogy
			Block size distribution

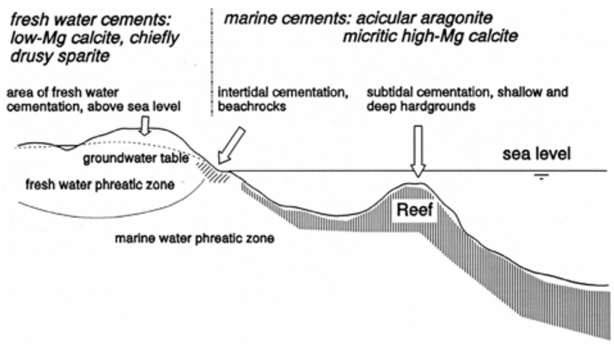

Figure 13 Cementation in coastal areas in the Middle East (Verhoef, 1997).

methods in such areas, like reflection and refraction seismics and resistivity surveys combined with high quality boreholes. Combinations of hopper, cutter, and backhoe dredgers are often used in cases when soft soils, stiff clayey soils resulting from weathering of rock, and variously weathered rock are all present in the project area.

Cemented soils

A second situation which is commonly problematic is the cementation that often occurs in sands when calcium carbonate saturated groundwater is available. This is common along the coasts of the Persian Gulf and other regions with warmer climates. Figure 13 illustrates this situation and indicates that it is possible to distinguish between calcite cement precipitating from fresh (meteoric) water or sea (marine) water, using petrographic techniques. Often, a combination of geophysical techniques as mentioned in the previous chapter is used to help detect variations in degree of cementation in the subsoil. Strength tests (UCS and BTS) should give insight into the range of strengths and degree of ductility of the material.

Necessary data

Summarising, for a dredging project involving rock we need:

- A good spatial model of the geology.
- Sufficient information on the properties of the rock to characterise cutting and wear behaviour.

To describe the rock mass well, information on the degree of fracturing is needed. This can be obtained from a combination of factors describing the discontinuities of the rock such as RQD and Fracture Spacing in cores from boreholes, observations of outcrops of the rock units on shore or offshore (diving), and using indirect indicators such as acoustic velocity obtained by refraction seismic surveys. Regarding the rock material properties, an important item is to have sufficient UCS and BTS test results of the rock materials present. Nowadays often only a few UCS or Point Load test results are available in tender documents. That is not enough. We need sufficient UCS and BTS tests per unit, with the requirement that UCS and BTS are determined on test specimens from the *same sample*. In that way the ratio m between UCS and BTS can be determined on data pairs coming from identical rock material. This ratio m and the variation of it in the rock mass is an important parameter which gives information on both the expected excavation production and wear (tool consumption). Table 3 gives a summary of the properties needed to make a judgment of the dredging process. The table reminds us immediately of the fact that up to now only the excavation process has been discussed. To estimate or predict the behaviour of the excavated material during transport and deposition forms a separate but by no means less interesting and complex subject, which keeps many dredging engineers occupied day and night.

Acknowledgements

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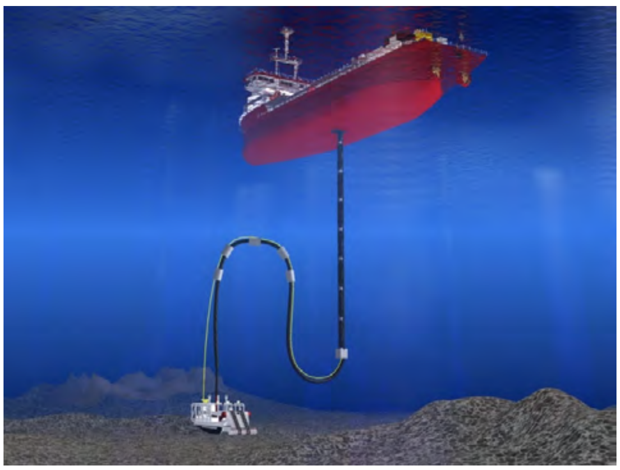
OceanflORE

Advertorial OceanflORE (joint venture IHC Merwede/DEME)

It is expected that the demand for raw materials will double in the coming decades. Existing sources on land will not be able to cater for this demand. Simultaneously, developments in deep sea mining techniques are going fast and it is becoming clear that deep sea mining is technically feasible. These trends have led to a situation in which there is a growing, worldwide interest in the options offered by deep sea mining. In addition, the capital market is becoming increasingly willing to finance this type of projects as a result of this rising confidence.



OceanflORE (a joint venture between DEME and IHC Merwede) is capitalising on the increasing demand for both expertise and exploitation opportunities within the sphere of deep sea mining. OceanflORE (ORE stands for *Ocean Resource Extraction*) has access to DEME company Tideway's high-tech equipment that can be used with enormous accuracy at depths that correspond to deep sea mining activities. Another important asset is the fact that OceanflORE can rely on DEME's extensive experience when it comes to fully processing the extracted materials onboard the mining ship before transporting them to terra firma. These activities encompass onboard integral processing of the reclaimed material and also washing and separating it. Dutch technology innovator IHC Merwede has the right credentials to develop high-end equipment with centuries of experience in dredging technology, unique expertise in the field of mining technology and deep water know-how. Through close cooperation between IHC Deep Sea Dredging & Mining, IHC Marine and Mineral Projects, and IHC Engineering Business, the product portfolio for deep sea mining is extending rapidly. OceanflORE aims to be the preferred partner for mine owners over the world in terms of both feasibility studies and the actual exploitation of deep sea mining projects. Their joint expertise enables feasibility studies to be conducted, after which every significant aspect of deep sea mining can be developed, staffed, and operated. These aspects include underwater mining, vertical transport to the surface, onboard processing on the mining ship itself, and transportation to the mainland. OceanflORE, therefore, is a party that is capable of providing a unique, pioneering, all-in solution for deep sea mining, including all aspects of project financing.

OceanflORE provides the market with an integrated contract mining solution, which focuses on extracting reserves from the ocean floor with their own equipment. OceanflORE focuses entirely on the mining process, allowing the mine owner to focus on his core business and to avoid investment in expensive technology. OceanflORE has the ambition to be the reference company offering an Integrated Contract Mining Solution resulting in a well determined *cost per tonne delivered* for our clients and partners.

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Thesis abstracts

Using 3D terrestrial laser scanning techniques to determine volume changes of a cobble beach

Anton Chan

The use of cobbles as a dynamic revetment is a recent development in coastal protection. Cobbles are placed along the beach and they form a dynamic equilibrium with the waves, thus dissipating the energy from the wave and protecting the soft, native material from eroding. In order to optimise the design and increase the durability of this method, it is important to obtain information on the long term behaviour of cobble beaches in a reliable way. In this research, the efficiency of using terrestrial laser scanning (TLS) to acquire a detailed model of the cobble beach and extracting information from it has been explored. Using a 3D terrestrial laser scanner, a dense point cloud of the cobble beach can be generated. A solid model has been created through scan registration, data processing (e.g. removal of outliers), and surface reconstruction. The volume of the solid can be calculated and compared to the volume of a subsequent scan to identify any changes. Finally, the validity of the model has been verified by comparing roughness profiles obtained from a laser sensor and the profile obtained by cutting the surface of the model.

Shear strength of Bremanger Sandstone rock fill at low stress

Fitsum Yemaneab

Bremanger Sandstone rock fill is used to form a cobble beach, to build an underwater foundation layer of a sea water breaker, and to construct a runway for a large crane during construction of Maasvlakte 2 (MV2), the extension of the Port of Rotterdam. The main purpose of this study is to determine in the laboratory the strength of Bremanger rock fills under low stress. Tests are conducted on rock fills with a finer particle size distribution than that used at MV2. Results obtained cannot be directly used in the design of the MV2 project. The influence of the testing equipment, for a given ratio of the equipment size to the particle size, of the degree of compaction and of the particle strength on the strength of the Bremanger rock fill are investigated. The effect of the test boundary conditions is researched by conducting tests with a triaxial cell apparatus, medium and small scale shear boxes and a tilt apparatus. The movement of particles was also studied during medium scale shear box testing to get a better insight into this effect. Emphasis is put on determining the contribution of dilatancy to the shear resistance of the Bremanger Sandstone rock fills with respect to the contribution to resistance to rolling of particles. For this purpose, dilatancy is measured during testing and the basic friction angle of the Bremanger rock fill is determined on sandblasted rock discontinuities with a Golder shear box. The basic friction angle controls the resistance to rotation of smooth particles. Test results are fitted using empirical models. The performance of these models at predicting the strength of the tested materials is assessed.

Investigation into quantitative visualisation of suffusion

Esther Rosenbrand

Suffusion is the process whereby seepage water removes fine grains from a soil, which can result in failure of the soil body. This poses a risk to structures founded on soils that are subjected to large hydraulic gradients, such as encountered near hydraulic dams or river levees. Currently, most experimental work on this topic is geared towards quantifying both the hydraulic gradient at which suffusion initiates, and the flux of eroded material. The reported values vary widely among experiments. The variation in the results can be explained by taking into account the effect of different experimental conditions. The flux of eroded material is the result of the interplay between particle erosion and filtration within the soil. Visualisation experiments allow for the direct observation of this. The effect of experimental conditions on the individual mechanisms of filtration and erosion, as well as the interaction between these, can be studied. Thereby, visualisation experiments complement methods targeted at quantifying the mass flux leaving the sample. In this work, the movement of fine grains and the resulting change in the structure of the sample are studied. Common laboratory equipment is used to design a visualisation experiment. The acquired images are analysed using three different quantitative image analysis techniques, with the objective of gaining further insight into the mechanism of suffusion. Particle image velocimetry (PIV) is an Eulerian method that is applied to determine velocity fields in fluid mechanics and granular flows. During suffusion, the velocity field is discontinuous; fine grains move whilst the coarse grains form a relatively fixed skeleton. This makes PIV less useful for the study of suffusion. To determine the displacement of individual particles, a Lagrangian method of particle tracking is considered. In the experimental setup used, fine grains are only tracked for a short length of time. This is due to both the large particle displacement between successive images, and the fact that other grains obscure the tracked particles from the camera. These difficulties can be remediated by improvement of the experimental procedure; the former by a higher acquisition rate, and the latter by use of a transparent granular medium where only the tracer particles are visible. With the apparatus used in this work, the temporal resolution is such that particle displacement cannot be studied unambiguously. Instead, a method of image subtraction (IS) is used that is geared towards quantifying the amount of material that moves. This yields data that can be interpreted to study both how much movement occurs, and where the movement occurs. Furthermore, IS is used to quantify the total change in the structure of the soil sample. Tests indicate that the load history plays an important role during suffusion. Erosion and filtration cause the soil structure to change, which has a direct effect on further particle transport in the sample. Therefore, the relation between three parameters (the number of moving particles, the location where they move to, and the progression of the experiment) is key to understanding the process of suffusion. This is studied by plotting the movement in a 1D section of the sample over time. It can be concluded that visualisation experiments complement existing outflow experiments to study suffusion. The results of IS can be related to conceptual models that are currently used to describe erosion and filtration processes; these concepts are applicable also to the process of suffusion. Improvement of the experimental setup is required to establish whether the observations reported in this work have a general validity.

Small and medium scale direct shear testing of Bremanger Sandstone rock fill

Xiochan Sun

This study focuses on the shear strength of the Bremanger Sandstone used as rock fill for a crane walkway. The rock fill was tested in small (100 x 100 x 40 mm) and medium scale (500 x 500 x 400 mm) direct shear boxes to quantify the effect of particle size, packing density and uniformity, specimen size, and normal stress on strength. Laboratory data was fitted with four different models (Mohr-Coulomb Model, Power Curve Strength Model, Hoek-Brown Model, and Barton Model). The Hoek-Brown model, initially developed for rock masses, was found to be suitable for rock fills. Finally, a crane walkway was simulated with Plaxis 10 Beta version to assess its stability.

3D structural and hydrogeological modelling of Metsähovi Research Site

Paul Gerrits

Due to the extreme sensitivity of the superconductive gravimeter GWR T020, based at Metsähovi Research Center in Kirkkonummi, Southern Finland, various local meteorological and hydrological changes influence or disturb its measurements. This study is part of a large research project which aims at identifying the contribution of the local hydrology at Metsähovi to these gravimeter measurements. For this purpose, a study has been done to investigate the geological structure of the subsurface and the hydrological properties of the stratigraphic layers. In order to perform numerical calculations, a digital model is made using digital modelling software. A field investigation is performed consisting of a literature study on the hydro-geological setting in Southern Finland, an extensive fieldwork consisting of boreholes, field observations and various field measurements (among others GPS, GPR, slug tests and monitoring of soil moisture with soil sensors), and laboratory work to obtain necessary soil properties. Main property which was found is the grain size distribution. A large amount of data is gathered and processed in order to integrate the data into a structural model. A method is found to integrate a maximum amount of information into a model and by optimally using the understanding and knowledge of the geological setting. A model with several layers is constructed. The area can be characterised in two hydrogeological domains, one is the higher area where overburden is thin and only till covers the bedrock, the other is a lower area dominated by a low permeable silt and clay layer. The till is low permeable but is still found to infiltrate water considerably. The hydrological setting of Metsähovi is analysed and theories of the hydrogeological processes which govern moisture changes in the area are investigated. Main focus is how these processes can be modelled with their governing laws and parameters. It is recommended to initially make a one-dimensional model with software such as CoupManual.

Conditional simulation for characterizing the spatial variability of sand state

Bram van den Eijnden

Properties of soils are spatially variable and to describe the behaviour of soils as a response to loading, this variability appears crucial in giving the correct range of possible solutions for structure response. As site investigation techniques only provide exact information for a limited part of the site, random field simulations are used to assess this variability over the full test site domain. The random fields use the spatial statistical characteristics that are derived from the site investigation, which in geotechnical applications mainly consists of cone penetration tests (CPT's). To reduce the range of possible solutions to be found for structure response analysis, the random fields can be conditioned by the actual CPT measurements. This report describes the conditioning of the random field in order to generate conditioned simulations of sand state fields. In order to derive the state parameter from the CPT tip resistance, the NorSand constitutive model is calibrated against the results of 55 triaxial tests of test site Maasvlakte 2. Different methods of calibration using triaxial test data are described and the results are discussed. 140 CPT's of the test site are then transformed into state parameter profiles. The statistical characteristics of the profiles are determined to be used for the simulation of the spatially variable fields of sand state. The statistical characteristics of the profiles are used in the conditional simulation of the fields. A conditional simulation algorithm to generate realisations of spatially variable sand state fields is derived and demonstrated. Using unconditioned random fields, generated with the Local Average Subdivision (LAS) method, conditioned simulations of the field around the CPT profiles are generated in a post-processing algorithm. The algorithm uses the geometry-dependent property of the kriging estimation error for the exchange of noise terms between estimation fields. The specific properties of the kriging estimator are demonstrated to be suitable to be used for the conditioning. The decrease in uncertainty by the conditioning with respect to the unconditioned random fields is presented. This decrease in uncertainty is used to demonstrate that the effectiveness of the conditioning is a function of the number and location of conditioning points relative to the scales of fluctuation of the field. It is demonstrated that conditioning reduces the range of possible solutions for the simulation of sand state fields with respect to unconditioned fields. This reduction will lead to a smaller range of solutions to be found when the conditional simulations are used in structure response analysis, leading to less uncertainty in design. The conditional simulation is shown to produce fields that honour the initial distribution function, the correlation structure and the actual CPT profiles in the simulated fields. To demonstrate that conditional simulation can be applied on the test site, a stochastic characterisation of the test site is performed and conditional simulations of the state parameter fields are generated for a small part of the test site.

Design and development of a hazard map for the positioning and siting of large jack-up rigs at the geologically complex areas of the Gulf of Suez

Bert Lietaert

The aim of this research project is to develop a hazard map for punch-through failure during jack-up rig installation in the Gulf of Suez. This map can be used to make an upfront assessment of unfavourable foundation conditions at a proposed installation site. This is not self-evident due to the complex geological setting of the area. Data to complete this research was provided by Fugro Engineers B.V. The available data set contains borehole data, geophysical data, bathymetric data and information regarding the surface sediments in the Gulf of Suez. The borehole data is used to perform bearing capacity calculations for different kinds of spudcan foundations, ranging in diameter between 10 m and 18 m. These calculations resulted in a distinction between locations with a 'safe' profile and locations for which a punch-through profile is generated. At these unfavourable locations, the actual risk of a punch-trough failure will depend on the deployed rig and the corresponding preload. Therefore, a factor of safety is calculated for these risky locations. The data was integrated into an ArcGIS project. Data analysis resulted in the identification of different safe and risky zones regarding punch-through occurrence. The identified zones turned out to be valid for every spudcan with a diameter between 10 m and 18 m. An observed trend is that for a constant preload, the risk for punch-through decreases if the spudcan diameter increases. Generalisation of the identified zones into depositional environments allowed the production of a risk map that also covers these areas in the Gulf of Suez for which no data was available. Two environments turned out to have the highest risk for punch-through failure. The first environment is characterised by a deep bathymetry and fine grained sediments, possibly with coarse grained intercalations. Punch-through in these areas is related to these coarse grained intercalations or to different degree of consolidation inside these fine grained packages. The second environment is related to areas where wadis bring a lot of sediment into the Gulf of Suez and develop an alluvial fan at their mouth. Mixing of this coarse grained input with finer grained deep water sedimentation results in the generation of punch-through profiles. Finally, for two areas with a high borehole concentration, an attempt was made to develop 3D ground models with the SGeMS software package. The bearing capacity inside the grid was predicted by applying ordinary kriging between the boreholes. The accuracy of these models and the practicability of the SGeMS program are thoroughly discussed.

The effect of filter jacket clogging on the performance of prefabricated vertical drains in soft soils

Arjan Kochx

Prefabricated vertical drains (PVD's) are used to accelerate drainage and consolidation of soft sediment. An effective functioning of these PVD's depends on its capacity to drain water from the subsurface. Among other causes, the clogging of filter jackets is thought to be of a considerable negative influence on the functioning of PVD's. Current regulations prescribe a minimum ratio between the filters aperture size and the grain size distribution of the soil to prevent this clogging. The objective of this thesis was to evaluate the effect of filter clogging on the performance of PVD filter jackets, with regards to the soft soil conditions that are encountered in large parts of the Netherlands. A literature review of the principle of vertical drainage, soil types, filter types, clogging criteria, processes and testing methods is carried out. An experimental program was undertaken in the laboratory in which filter jackets were exposed to two different clogging phenomena, i.e. particle clogging and chemical clogging. Also filters from the field were evaluated. All filters were evaluated on their loss of permittivity with constant head measurements on filter pieces of three different filter types: the D165 and HS5417 from the Typar line of DuPont and the 30195 of Freudenberg. Particle clogging was evaluated by two different setups: the oedometer and a long term filtration setup. Within the oedometer tests the load and the moisture content of the clay sample varied. Chemical testing was done briefly, to see to what extent iron precipitation could induce filter clogging and if one filter would be more prone to clogging than the other. Permittivity tests on the filter pieces showed reproducible results. Repeated measurements on the same sample showed an average standard deviation of not larger than four percent of the samples mean. The initial permittivity also corresponded reasonably well with the values provided by the filter manufacturers. With particle clogging, it was seldom observed that clogging ratios were higher than five. Relations between clogging and the applied load on the clay sample could not be found. Increasing moisture content showed a slightly higher clogging ratio. Another clogging indicator is initial permittivity. This is believed to be a better estimator; higher initial permittivity in general gave lower clogging ratios. In the chemical experiment it was observed that on the filter of Freudenberg large amounts of iron oxide precipitated. During filtration the filter seemed fully clogged. Permittivity tests on this filter showed an approximately four times larger clogging ratio than the other filters. Based on literature and observations from both experiments and field data it could be concluded that the permittivity reduction of filter jackets is always less than a factor of ten. Hence filter clogging is not a real issue, since the permeability of soft sediments is several orders of magnitude lower than that of the clogged filters. Hence, the system permeability is mainly determined by the soil behaviour.

Interactions between beach rock formations and shoreline evolution (Togo case study)

Guido Rutten

Beach rocks are lithified coastal sedimentary formations which form inside the beach body on short time scales, possibly within a year. Beach rocks can play an important role in the evolution of a shoreline as they fix the normally loose sediments. On the other hand, the evolution of a shoreline could play an important role in the genesis of beach rock and their preservation. A better understanding of these interactions will aid their use in science, to understand their genesis, and to reconstruct paleo-climates. The knowledge derived is also useful for coastal engineering practice, to evaluate the response of a coastline to beach rock exposure. This study evaluated the interactions between beach rocks and shoreline evolution both from a theoretical point of view as well as in a practical case study in Togo, West Africa. In the theoretical review, the variability of shore and shoreline has been related to the diagenetic environment of a beach rock. Furthermore, existing literature was reviewed in order to assess the existing knowledge on shoreline response to the exposure of beach rock. A two month field campaign was carried out in Togo to gather data on beach rock characteristics and shoreline development. Finally, a synthesis was made of the theoretical findings and the results of the field campaign. The process of beach rock genesis and the controls on this process are not fully clear. Multiple mechanisms have been proposed but cannot be confirmed for all reported occurrences of beach rock. It is thus likely that no single process is responsible for the formation of beach rock. The availability of carbonate and the temperature of the beach water are likely to be among the controlling factors of beach rock formation. It has been proposed that the stability of a beach is another controlling factor, as agitation of beach particles would not allow cementation to develop. This proposition does however not properly consider the variability of shore and shoreline. The results show that short term variations of shore and shoreline (hours to decades) are likely to be a control on beach rock genesis. The variability of the diagenetic environment related to these variations determines the cementation characteristics of beach rock. Shoreline variations thus determine the diagenetic history of a beach rock, which in turn influences the rock mass properties of the beach rock. Longer term trends in shoreline evolution are probably not linked to the genesis of beach rock, thus limiting its paleo-environmental significance. This theoretically derived conclusion needs to be confirmed by more detailed investigation in the field. The analysis of shoreline variability related to beach rock genesis can be used to further evaluate the different mechanisms of formation that have been proposed. Analysis of weathering patterns can be the next step in increasing the paleo-environmental significance of beach rock formations. In the case study of Togo, the impact of a very large outcrop of beach rocks on shoreline evolution was investigated, by defining an approach that could be applied in similar cases. The beach rock in Togo can be divided into two formations: a main formation which shows high continuity and strength, and effectively blocks cross-shore sediment transport along major parts of the coast of Togo. Another less profound formation is found adjacent to the main formation. The beach rock in Togo fulfils a role similar to hard engineered structures that are often used in coastal engineering to protect sandy coastlines. The placement of these engineered structures in the nearshore zone triggers a certain response that is adequately understood and can be predicted using modelling software or engineering guidelines. The retreat of a sandy coastline with buried discontinuous beach rock formations poses difficult questions that have not yet been accounted for in science. The complex coupling between sediment transport and hydrodynamic conditions in the nearshore zone is not well understood, and the activation of non-erodible elements inside or as a continuous part of beaches has received only minor attention. Furthermore, beach rocks differ fundamentally from engineered hard structures as they exhibit a large natural variation in cementation characteristics and thus rock mass properties. Whereas for engineered structures the mechanical properties are known and can be used to calculate the lifetime of the structure, for beach rock a more thorough understanding of the genesis needs to be developed. The dominant mode of weathering of the main formation appears to be undercutting by scouring around the seaward base of the beach rock. The response of a retreating sandy coast to buried beach rock formations can be qualitatively analysed using the Phased Retreat Model presented in this study. This model uses equilibrium shoreline profiles to determine different phases in the process of retreat. Longshore processes play a dominant role in the retreat of a coast behind a beach rock barrier. Starting from locations where the beach rock barrier has been breached, erosion landward of the beach rock travels in the opposite direction of the longshore current seaward of the beach rock. The Phased Retreat Model can be used to further analyse and predict shoreline development along the coast of Togo, using a more data-driven analysis and by investing in structural solutions. The Togolese government can upgrade their shoreline protection policy to the philosophy of Integrated Coastal Zone Management. Furthermore, the presented model could be used in similar cases elsewhere. Lastly, the Togo case can be used for further generic research on morphodynamic response to barriers in the nearshore zone.

A study on monitoring the first field applications of Biogrout in gravel

Werner van Hemert

Biogrout is a new ground improvement method, developed by Deltares, Volker Wessels, and Delft University of Technology. Biogrout involves the injection of liquids, which contain bacteria and salts, in the ground where they induce precipitation of calcium carbonate, which improves the soil strength. This report is the result of a study on monitoring of the first field application of Biogrout, executed for Gasunie and Visser & Smit Hanab. Biogrout is used in gravel layers below the groundwater level to stabilise boreholes during horizontal directional drilling (HDD) at two locations (Beuningen and Slijk-Ewijk). The main research question is: 'how to monitor Biogrout in gravel and gravelly soil layers?' After a thorough literature survey on monitoring methods, the following in situ methods have been applied during the field application of Biogrout: Cone Penetration Test (CPT), Video-CPT, (light and medium), Dynamic Penetration Test (DPT), falling- and constant head permeability tests, and S-wave seismic monitoring to measure the influence of calcium carbonate precipitation on the soil strength and porosity/permeability. Further, sampling by use of trench excavation, sonic vibratory drilling, and other methods was performed together with groundwater sampling to monitor the calcium carbonate precipitation in the subsoil. Also, 2D and 3D geo-electrical monitoring to follow the flow of conductive saline fluids in the subsoil was done. Furthermore, several laboratory methods have been performed to measure Biogrout induced geotechnical parameters for field samples and laboratory prepared gravelly soils: direct shear tests, point load tests, and UCS tests to determine the strength of the samples. Further, measurement of the amount of calcium carbonate present in the samples was carried out and SEM microscopic observations were done to confirm whether calcium carbonate precipitation is induced by Biogrout injection. The only clear increase in in situ strength and stiffness is measured by seismic S-wave: an increase of 245% in shear modulus is measured due to calcium carbonate precipitation. Other in situ tests are too much disturbed by gravel particles to measure the small increase in strength induced by calcium carbonate precipitation. The geo-electric measurements show a clear boundary of the conductive Biogrout injections and the resistive gravel layers filled with fresh groundwater. This indicates that the injection and extraction wells are well designed and keep the injected calcium chloride and ammonium within the intended region. With trench excavation, very weak undisturbed cemented gravel samples were retrieved from the subsoil. With SEM microscopic observations it was confirmed that the origin of cementation is Biogrout injection. Furthermore, gravels with a high calcium carbonate content ($CaCO_3$) were retrieved with (shallow) sampling. In the HDD6 Biogrout injected area an average increase in calcium carbonate of 38.25 kg/m^3/m is measured, which is 89% of the expected value (43 kg/m^3). It must be taken into account that this value is mainly based on sand samples, which show a large fluctuation in natural calcium carbonate content. Therefore also a large difference in $CaCO_3$ content over the sampled soil body is noticed. Laboratory direct shear and point load tests show that with each ratio (%) increase of calcium carbonate content an approximate UCS increase of 0.11 MPa occurs. This can also be expressed as an increase of 0.11 MPa for each 18 kg/m^3 $CaCO_3$ in gravel. After monitoring Biogrout injection at HDD6 and HDD7 it can be concluded that precipitation of calcium carbonate by means of Biogrout in gravelly soil has successfully been applied for the first time in a field application. However, the best suitable monitoring method is difficult to find and will differ for each potential Biogrout application.

The boundary conditions in direct simple shear tests: developments for peat testing at low normal stress

Matthieu Grognet

More than half of the Netherlands is situated below sea level. Therefore, evaluating the safety of dikes is primordial. A specific interest is given to peat dike safety which suffers from of a lack of knowledge manifested recently by some peat dike failures. The behaviour of peat is also of interest to other countries, for instance for assessing peat slope stability. Due to its high anisotropy and fibre content, peat cannot be tested with any device in the laboratory. The direct simple shear test is routinely used since it can simulate several in situ conditions and provides conservative results for peat dike stability evaluation. Furthermore, it does not show the inconvenience of triaxial testing with peat. Larger samples than usual are desirable to investigate the effect of fibres on tests results. The direct simple shear testing devices remain imperfect since no additional shear stresses can be applied to the sides of the specimen. As a consequence, non-uniformities develop on all the faces of the specimen, in particular compression in the obtuse corners and tension in the acute corners. In practice, thin samples are used (height/diameter ratio ca. 0.2-0.3) to limit the non-homogeneities to the sides and leave the major part of the sample in a homogenous state of stress. Testing peat at low vertical stress remains a challenge and asks for the development of adapted devices. A series of test methods have been performed on a wood and sedge peat with the Geonor device in order to compare the effect of two boundary conditions on tests results. The first one is a classical reinforced membrane, the second is a non-reinforced membrane enclosed in a stack of rings. The vertical stresses applied during the tests varied between 10 and 120 kPa. The results show small differences when the Mohr-Coulomb parameters are determined. The comparison is limited considering the variability of the material tested. A more accurate calibration of the stack of rings would be desirable. Some improvements are needed on the actual apparatus to test peat at low vertical stress. Removing the membrane between the soil and the rings would give more accuracy in the results. A direct simple shear prototype has been developed in order to test larger samples (height/diameter ratio 0.5) at low vertical stress. The effect of two innovative rough boundaries on the stress-strain homogeneity of the sample has been investigated. The sidewalls of the device are transparent and make a visual assessment of the deformation of the sample possible. The Particle Image Velocimetry analysis is also considered to assess the shear strain homogeneity in the sample. The results show improving shear strain homogeneity and reduced tension forces in the acute corners. Slippage is also observed between the top cap and the sample whereby the normal load could not be measured. Further research is needed to validate the utility of this prototype. Stress-strain curves obtained from the three boundaries should be compared to quantify the improvement of rough boundaries. A finite element analysis of the prototype boundaries has been performed with two models (Mohr-Coulomb and Soft Soil Creep). The boundaries considered were perfectly rough at the top and bottom and perfectly smooth at the sides. The presence of strips and even more the presence of vanes increase the stress-strain homogeneity inside the sample with both models. Reliable stress-strain curves as measured in classical devices could not be obtained with such boundaries. Interfaces should be preferred to model more realistic conditions.

The deviating interpretation of Almere to the NEN 6743: the consequence of a heterogeneous soil profile, or a result of misinterpreted overconsolidation?

Linda de Vries

Actual bearing capacity of piles in Almere often differs from the expected values as calculated using nearby CPT data; the calculated strength is often higher than can be achieved in the field. To calculate the bearing capacity of foundation piles within Almere an addition has therefore been made to the standard NEN 6743-1. The proposed hypotheses for these deviations include heterogeneous soil behaviour and overconsolidation. These hypotheses are tested by a multiple data acquisition approach in which available data such as geological data and available maps have been used along with newly acquired data including GEM-2, CPT, core logs, and laboratory tests to obtain a better insight in the problem. The investigated maps, Begemann core logs, and CPT data have shown that the subsurface of Almere indeed reflects a heterogeneous character at multiple scales. Comparing CPT soil behaviour type profiles to Begemann-, grain size-, and micro scale CT scan data concludes that the translation from CPT data to in situ soil parameters can be inaccurate; different soil types can show a similar behaviour. Inaccurate CPT interpretations can result in bearing capacity miscalculations, but by itself cannot be the result of the deviating soil behaviour of Almere. Overconsolidation of the soil has been proven by vertical stress calculations and multi-stage triaxial test results of two samples. Overconsolidation may have resulted in an overcompacted soil which has the tendency to dilate when applied to vibratory sources, like the installation of pile foundations, which makes it a more plausible explanation for the deviating soil behaviour. It is recommended to quantify the amount of overconsolidation of the subsurface and its contribution to the reduction of the bearing capacity with an extensive research in which the overconsolidation of the soil is measured before and after installation of the pile.

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