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Research and Development Activities on Pile Foundations in Europe

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ABSTRACT

The paper describes some recent field test programmes performed across Europe. Given the increased need to develop alternative, clean energy sources the paper presents details of R&D programmes with a focus on reducing costs offshore developments and increases safety. Work on the development of design codes for large diameter piles is presented. New areas of research are highlighted in particular ongoing research is needed to look at effects such as cyclic loading and dynamic interaction. In the offshore field the use of piles that reduce environmental impact is an area of ongoing research that is addressed. Interesting full-scale field tests looking at the problem of negative skin friction are also considered.

1. INTRODUCTION

Whilst clean, renewable energy solutions are being developed there is a continued need for offshore oil and gas platforms to provide an important part of our energy mix. Jacket structures for oil and gas platforms are commonly founded on (predominantly axially loaded) open-ended steel tubular piles. The challenge to companies in this sector are to reduce costs and at same time increase safety. New exploitation opportunities are rare and are typically in deep water areas with challenging geology. Reassessment of and life-time extension of old platforms is also becoming more important. One major benefit for developers and operators are that the new CPT based design approaches for axially loaded piles now included in the offshore design standards typically allow for increased axial pile capacities to be adopted, particularly in dense sand deposits. The CPT based design approaches that account for friction fatigue, stress level, interface shearing characteristics and plugging have been shown to be more reliable than traditional design approaches in database assessments by Chow (1997) and Schneider (200) amongst others. However, topics such as the effect of pile ageing and cyclic loading on axial pile capacity remain uncertain. These issues are addressed in Section 2 of the paper.

Many countries around Europe have developed ambitious targets for energy generation from renewable sources. Offshore wind farms are seen as key to achieving this aim. Siting turbines offshore provide several benefits including: (i) availability of   high unrestricted wind speeds, (ii) the ability to use larger turbines and (iii) the ability to develop combined wind and wave/solar energy installations as alternative renewable energy solutions which become economically viable. The majority of turbines constructed to date have been founded on monopile foundations. Because of increased water depths and larger turbines, monopile diameters are increasing, with planned developments having piles of 10 m diameter. The cost of foundations can represent over 30% of the development cost for an offshore wind farm and advances in pile design that allow for cost savings are urgently required. In addition critical issues related to the impact of cyclic loading, soil-structure interaction for dynamic analyses and environmental concerns around the impact of installing large-diameter piles have led to the development of large industry-academia R&D projects. These issues are addressed in Section 3 of the paper. In the final section issues related to onshore piling such as the use of low-displacement piles in addressing environmental concerns and field tests investigating negative skin friction on piles are discussed.

2. FIELD TESTS INVESTIGATING PILE AGEING AND CYCLIC LOADING OF PILES

The positive gain in capacity with time (i.e. set up) of driven piles in clay is well understood due to general acknowledgement of the effective stress changes associated with the dissipation of excess pore pressures induced by driving. Set-up for piles in sand is a less well known phenomenon. A number of field studies of pile ageing in sand have been conducted in recent years. Four major field studies have recently being conducted by researchers at Imperial College London (IC), the Norwegian Geotechnical

Institute (NGI) and University College Dublin (UCD). The work described in detail by Gavin et al. (2015) is summarised herein.

2.1. Imperial College tests at Dunkirk

The Imperial College Dunkirk pile test site at Dunkirk, Northern France is comprised of a 3 m thick layer of hydraulic sand fill over Flandrian marine sand. The ground water table is approximately 4 m below ground level (bgl). The sand is a fine to medium, predominantly silica deposit. The geotechnical characteristics are described in detail by Chow (1997). In summary the deposit is dense to very dense with relative density of $\approx 75\%$ .The sand has a peak friction angle ϕ'_p in the range 35-40°, constant volume friction angle ϕ'_{cv} of 32° and interface friction angle δ of $\approx 27°$. A typical CPT q_c trace showed end resistance, q_c values in the range 10 to 30 MPa over the depth of interest for piling at the site and the presence of a weak organic layer at 8 m bgl.

An investigation of ageing of 325 mm, OD 11 m to 22 m long steel pipe piles (Chow et al. 1998) showed that the tension shaft capacity of a pile rose by 85% between tests performed over a five year period. Jardine et al. (2006) undertook further pile driving and testing at the same site that was designed to specifically investigate pile ageing. They drove six steel tubular test piles (designated R1 to R6) had a diameter, D of 457 mm, a wall thickness which varied between 13 and 20 mm. They were driven to penetrations of between 18.9 and 19.4 m. Monitoring of the soil core during pile installation revealed that partial plugging occurred with plug length ratios (PLR = soil core length/pile penetration length) of approximately 60%. Slow, load-controlled, tension load tests were performed on the piles at various time periods after installation. One of the most striking features of the response was the strong gain in capacity observed during first time load tests on fresh piles which were tested between 9 days and 235 days after installation with the resistance increasing from 1450 kN to 2400 kN, See Figure 1.
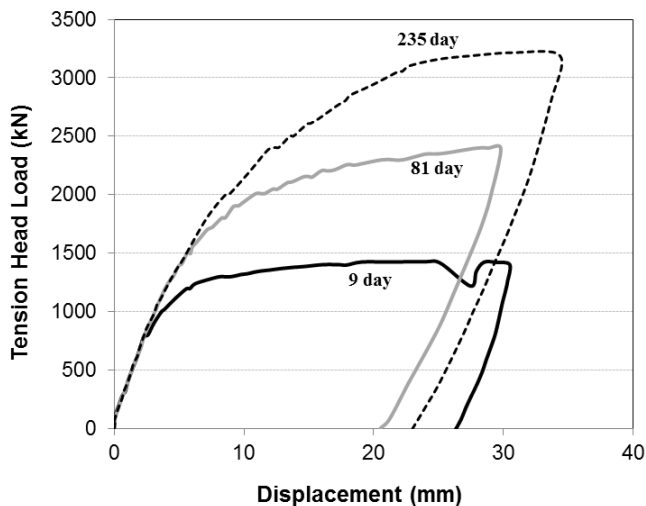

Figure 1: Effect of time on load-displacement response of fresh piles (after Jardine et al 2006)

The data from Figure 1 was added to dynamic End of Driving (EoD) capacity assessments made for the instrumented piles at the same site reported by Brucy et al (1991) and the tension capacity from a pile test on a five year old, 22 m long pile reported by Chow (1997), to propose an intact ageing characteristic line (IAC), see Figure 2. In this plot the shaft capacity, Q_s at a given time, is normalised by the shaft capacity predicted using the ICP pile design approach, Q_{sICP}. The data suggests that the semi-logarithmic rates of capacity increase are modest initially before finally stabilising.
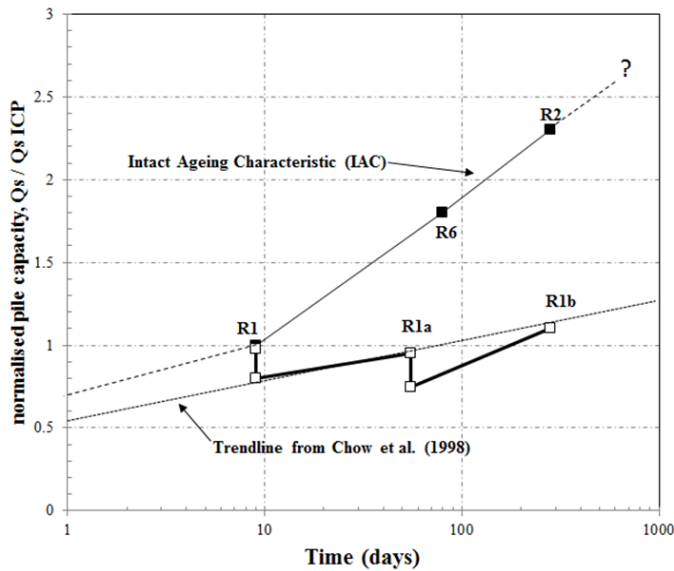
Figure 2: Effect Intact ageing curve for the Dunkirk pile tests

2.2. University College Dublin Pile Tests in Blessington Sand

This study on pile ageing involved load testing of four, 7 m long open-ended steel piles. The test piles were driven into dense sand at the UCD Blessington test site in Ireland. The glacially deposited fine sand has a relative density close to 100% and the ground water table was approximately 14 m below the piling platform level. At each pile test location two CPT tests were performed prior to pile installation. The CPT q_c resistances measured were relatively consistent across the test location, See Figure 3. Additional information on the ground conditions at the site and the sand properties have been reported in Gavin et al. (2009), Doherty et al. (2012) and Kirwan (2015).The piles had identical geometries with an external diameter of 340 mm and a wall thickness, t of 14 mm. One of the piles (designated Pile S5) was instrumented with Kyowa miniature radial stress sensors located at distances from pile toe, h, of 1.5, 5.5, and 10.5 and 17.5 pile diameters. The piles were subjected to maintained tension load tests at time intervals of 1 day, 11 days, 30 days and 220 days after driving respectively. Details of the pile construction, instrumentation and calibration can be found in Kirwan (2015).
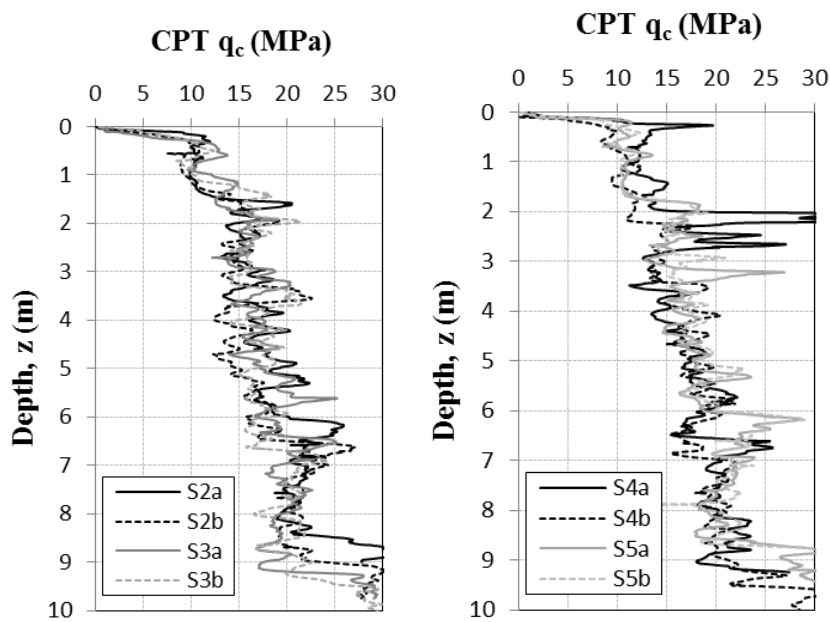

Figure 3: CPT profiles conducted beside piles S2 – S5

Pile driving was performed using a Junntan PM-16 drop hammer with a drop-height of 0.3m. The pile driving records and soil core development (in terms of Incremental Filling Ratio, IFR = incremental change of soil plug length/change in pile penetration) for three of the piles (S2, S3 and S5) were similar. Pile S4 had a similar response in terms of plug development and blow-counts until a penetration depth of 4m. Thereafter, the degree of coring increased and the number of blow-counts for this pile was significantly lower than the other test piles.

The load-displacement response measure during first-time loading on fresh piles is shown in Figure 4. There was a generally a strong trend for the tension capacity of the piles to increase with time. A notable exception is Pile S4. It is possible that Pile S4 had a significantly smaller capacity than piles S2, S3 and S5 at the end of driving due to the much higher IFR and lower installation resistance noted over the last 1 m of penetration and the notably lower blow counts recorded during installation.
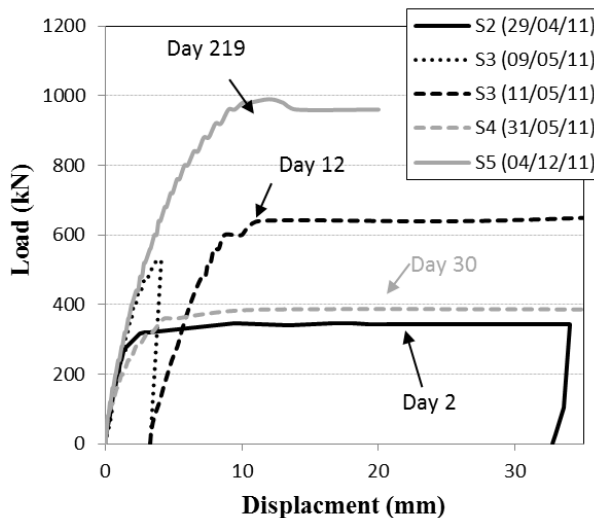

Figure 4: Load-displacement response measured during first time load tests

One of commonly assumed mechanisms for pile ageing is that breakdown of hoop stresses set-up during installation would allow stationary radial stress measured on the pile to increase with time. Such increases have been recorded on closed-ended concrete driven piles by Briaud et al. (1989) and Axellson (2000). The radial stress sensors on pile S5 at Blessington were logged periodically until the load test was performed 217 days after driving. The variation of radial stress over the setup period is shown in Figure 5 together with the stationary radial stress profiles predicted using the IC-05 and UWA-05 design methods. The following points are noteworthy:

1. At the end of installation the radial stresses measured at all sensor locations were much higher than predictions made using either of the design approaches.
2. Rather than increasing with time, relaxation of the radial stress occurred during the set-up period. This relaxation was evident at all sensor levels.
3. At the end of the set-up period, the average stationary radial stresses had reduced significantly. Whilst the values measured near the pile tip were still significantly higher than the values predicted by both the ICP and UWA design methods, it should be noted that both design approaches were developed based on pile test data with a minimum h/D value of 4. At locations further from the pile tip, the radial stresses are much closer to the predictions made using the CPT based design approaches.
4. During static loading, as the test load increased the radial stress sensors at all locations showed a trend for large increases in radial stress due to dilation. Near the pile tip, the radial stress doubled, reaching a value just below the end of installation radial effective stress. Large stress increases were noted at the other two sensor levels.
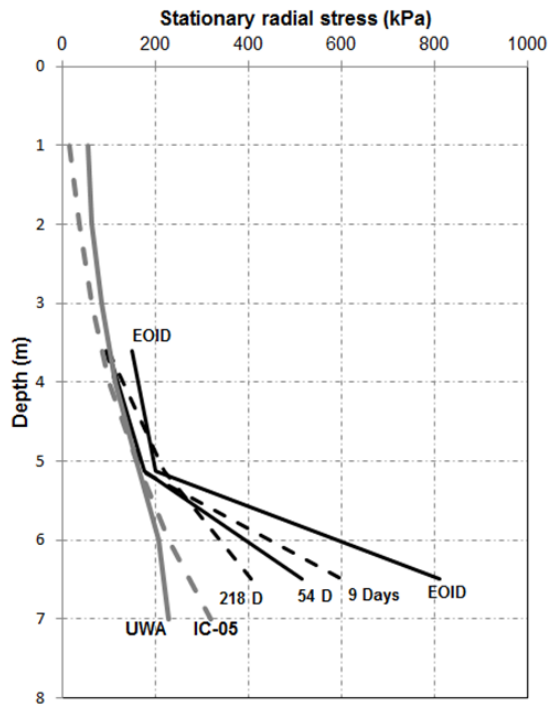
Figure 5: Variation in radial stress measured with time

The study found in general that the axial capacity of the piles increased by 185% over 220 days. However, one pile which experienced less plugging during installation mobilised a lower resistance than the other three test piles. During a 220 day equalisation period, the radial stress measured near the pile toe reduced significantly. During loading, large increases in radial effective stress occurred at all levels of the pile as a result of dilation.

2.3. Norwegian Geotechnical Institute Field Tests at Larvik and Ryggkollen

A major joint industry project (JiP) on pile ageing that included load tests at two sand sites in Norway were reported by NGI (2014) and Karlsrud et al. (2014). Axial load tests were performed on piles at each site where six open-ended piles with diameters of 400-500 mm and driven lengths ranging from 15 to 20.1 m were installed and load tests were performed over a two year period. The sand deposit at Larvik site is a recent, alluvial deposit of loose, fine silty sand with some interbedded layers of clayey silt, constituting 9 to16% of the test bed. The q_c values in the range 1.5 to 10 MPa, with a relative density interpreted to be in the range 20 to 40%. The water table at the site was close to ground level. The sand at the Ryggkollen test site is a gravelly medium sand formed by an end moraine. The site was located in a sand quarry. CPT tests gave q_c values in the range 20 to 30 MPa, which suggested relative density is in the range 50 to 80%. The water table was at 10 m bgl. Detailed soil properties can be found in NGI (2014).

The test piles were driven using a hydraulic hammer. Casings were used to isolate the pile response from overlying soil layers; the casing depth was 1.4 m at Larvik and 5 m at Ryggkollen. The piles installed at Larvik were 508 mm diameter, with a wall thickness of 6.3 mm and were driven to an embedded length of 20.1 m. At the end of installation PLR values were between 83 and 87%. The piles installed at Ryggkollen had an outer diameter of 406.4 mm, with a wall thickness of 12.5 mm, and were driven to an embedded length of 15 m. At the end of driving, PLR values were generally between 63 and 84%. However, one pile (R5) developed a relatively short soil plug (PLR= 32%) after hitting a stone during installation. The load test set-up is shown in Figure 6.

Figure 6: Typical load testing arrangement, NGI tests (after NGI, 2014)

The axial capacity measured during first-time load tests on piles is shown in Figure 7, where the capacity Q_0 is the reference (10 day capacity of the piles). The pile capacities at both test sites show a trend for substantial increase with time. However, comparing these site with pile ageing tests at Dunkirk and in Blessington sand (See Gavin et al. 2013) show the growth rate is quite variable. Also the nine month pile capacity at Larvik was higher than that measured in the 12 and 24 month tests. Karlsrud et al. (2014) attribute this trend to the local variability of soil conditions at the site, which caused variation in both the driving resistance and degree of plugging experienced. This normalisation approach suggests that both the rate effect (i.e. the time when the largest gain in capacity is developed) and the maximum set-up vary from site to site. The piles at Ryggkollen were slowest to show any ageing effects. The interpreted increase in ultimate shaft resistance after 12 months varies from a factor of 1.75 to 2.55, with Larvik giving the lowest and Blessington the highest.
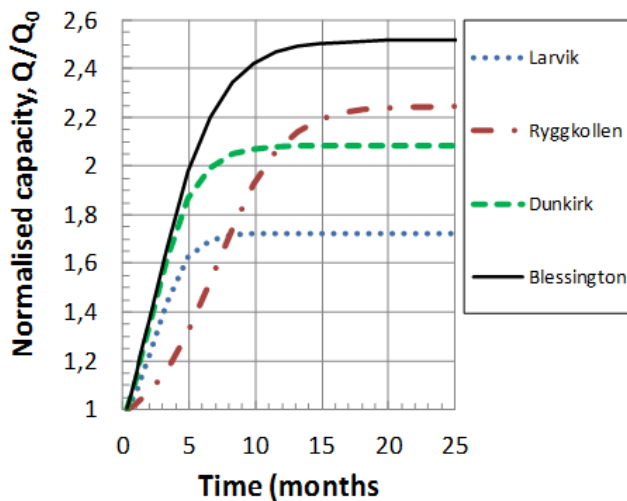

Figure 7: Proposed curve-fitted capacity build-up with time (after Karlsrud et al, 2014)

Repeated load tests on the same pile at both Larvik and Ryggkollen showed a significantly lower capacity at the same time after pile installation than the corresponding fresh tests. Karlsrud et al (2014) therefore warned against using repeated load testing to failure as basis for addressing real time effects. The piles subjected to sustained loading over 12 months before being loaded to failure also showed less gain in capacity than the fresh piles. The gain after 24 months dropped by a factor of respectively 1.42 (Larvik) and 1.16 (Ryggkollen). Karlsrud et al (2014) recommend that this aspect must be considered when applying time effects in design practice. Further investigations are needed to assess how this negative impact of sustained loading depends on the level of sustained loading, and if it is any different for piles loaded in compression as compared to the tension loading applied in NGI's tests.

2.4. The Effect of Cyclic Loading

The cyclic response of piles in sand has been shown by Chan and Hanna (1980) to be affected by the number (N) of load cycles, the mean shaft load (Q_{mean}) and the shaft cyclic amplitude (Q_{cyc}) among other factors, as shown in Figure 8. While a number of model laboratory scale cyclic pile studies have been conducted, the dearth of field cyclic tests on driven piles in sand has been highlighted by Tsuha et al. (2013). Limited suites of field cyclic load tests on instrumented piles have been conducted by Imperial College at Labenne and Dunkirk in France (Lehane 1992 and Chow 1997) and by University College Dublin (UCD) at the Blessington test site (Gavin and O'Kelly, 2006, Igoe et al. 2011).
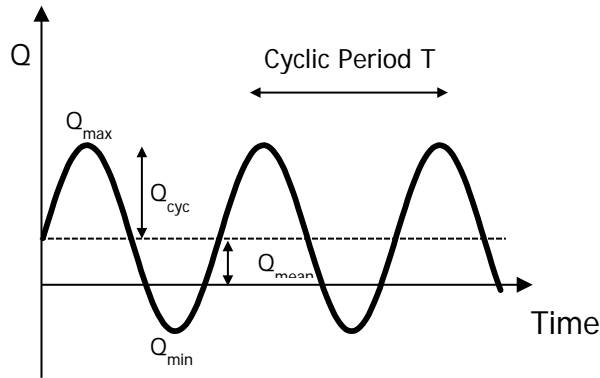

Figure 8: Definition of cyclic loading parameters (after Tsuha et al. 2012)

A range of static and dynamic load tests were undertaken by Jardine et al. (2008) in order to evaluate the effects of prior cyclic or static failure on the ageing process. The effects were striking with the authors concluding that any test to static or cyclic failure degrades the pile capacity and can lead to a brittle load-displacement response. While the pile capacities recovered with time, the growth rates were lower than with fresh piles. Data from retests performed on pile R1 shown (as the lower line) in Figure 2 reveal that;

(i)   The capacity at any time for a reloaded pile is lower than the capacity of a fresh pile.
(ii)   Performing a subsequent reload test on the same day verified the reduction in the pile capacity caused by the preceding failure, explaining the zigzag pattern in Figure 2.
(iii)   The capacity of the statically pre-tested piles increased with time. However, the rate of set-up was lower than for fresh piles.
(iv)   The trend for capacity variation for time for the reloaded pile was similar to the average trend line proposed by Chow et al. (1998) from their database study. Most of the entries in the latter databases had involved multiple re-tests on the same piles.

Comprehensive testing on multiple piles by these author showed that cycling above certain combinations of mean and cyclic loads led to changes in capacity;

-    Low level stable cycling, where the pile head displacements accumulate slowly over hundreds of cycles, was shown to accelerate the ageing process resulting in increased capacities. High level unstable cycling, where head displacements develop rapidly leading to failure within 100 cycles, was shown to degrade the capacity of aged piles. Metastable load cycling, where head displacements accumulated at a moderate rate leading to failure between 100 – 1000 cycles, also resulted in significant capacity reductions.
-    Cyclic interaction diagrams were proposed to express how cyclic loading parameters N, Qmean and Q_{cyc} affect the piles cyclic response relative to the max static shaft capacity, $Q_{s,max}$ as seen in Fig. 9. Boundaries for stable, metastable and unstable behaviour were proposed and it was noted that the boundaries did not appear to be sensitive to whether the piles had experienced prior cyclic or static failure.
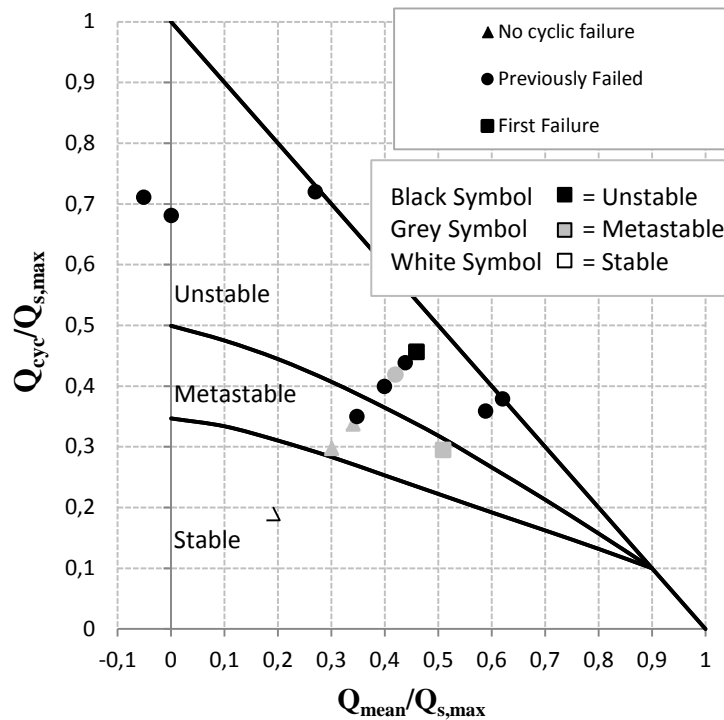
Figure 9: Cyclic Interaction Diagram based on Dunkirk cyclic tests (after Jardine and Standing 2012)

3.   MONOPILES FOR THE OFFSHORE WIND INDUSTRY

3.1. Background

The limitation of the current design codes (that were developed for long-slender piles) to the design of laterally loaded monopiles used in the offshore wind sector are well known (See Doherty and Gavin 2011).

3.2. PISA Project

3.2.1.   Background

The PISA (PIle Soil Analysis) project is a JiP established to develop new design methods for the very large diameter, relatively rigid monopiles with diameters up to 10 m used to support turbines. The project involves: (i) numerical modelling using 3D finite element analysis; (ii) the development of a new design method, and (iii) field testing at two separate sites, dense sand at Dunkirk and stiff clay at Cowden. The work is being directed by an Academic Work Group (AWG), led by the University of Oxford and comprising researchers from Imperial College and UCD in collaboration with the Geotechnical Consulting Group and supported by the larger PISA industrial consortium. The comprehensive 3D finite element analyses that were completed for the development of the design method and to inform the field test programme are described by Zdravkovic et al. (2015).

3.2.2.   Design Models

A new design approach that draws on the existing p-y method and is based on 1D finite element model of a monopile has been developed. In addition to the standard distributed load (p) curve See Figure 10, the model includes a distributed moment curve (associated with the vertical shear stresses on the pile shaft), a base shear curve, that defines the relationship between the shear force, S and lateral displacement, and a base moment curve, M_b considering the rotation at the pile toe. Two approaches are considered for determining the soil reaction curves; (i) a rules based method with recommended parameters determined from basic strength and stiffness parameters and (ii) a more detailed approach using bespoke finite element analyses to derive site specific soil response functions.
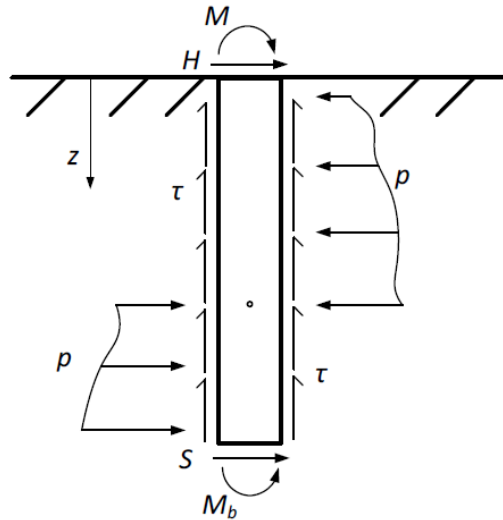
Figure 10: Soil reaction components applied to a monopile (Byrne et al 2015a)

The field test programme had a vital role in the development of the new design methods, by validating both the numerical models and the parameterisation of the soil response.

3.2.3. Field Tests

The major field test programme undertaken at the two sites performed between October 2014 and July 2015, summarised below is described in detail in Byrne et al (2015a). The field tests were performed in an onshore setting to allow for multiple tests to be performed that investigate a range of the parameters that govern pile performance and to investigate the effects of scale. The test piles were loaded at a height h, above ground level of between 5 and 10m in order to consider normalised Moment, M, horizontal load, H and pile diameter, D ratios representative of those arising from wind and wave loads applied to offshore wind turbines. The piles were instrumented with fibre optic strain gauges, inclinometers and extensometers in the buried section (piezometers were placed in the soil at Cowden). In addition above ground instruments included inclinometers, LVDT's and load cells, See Figure 11.
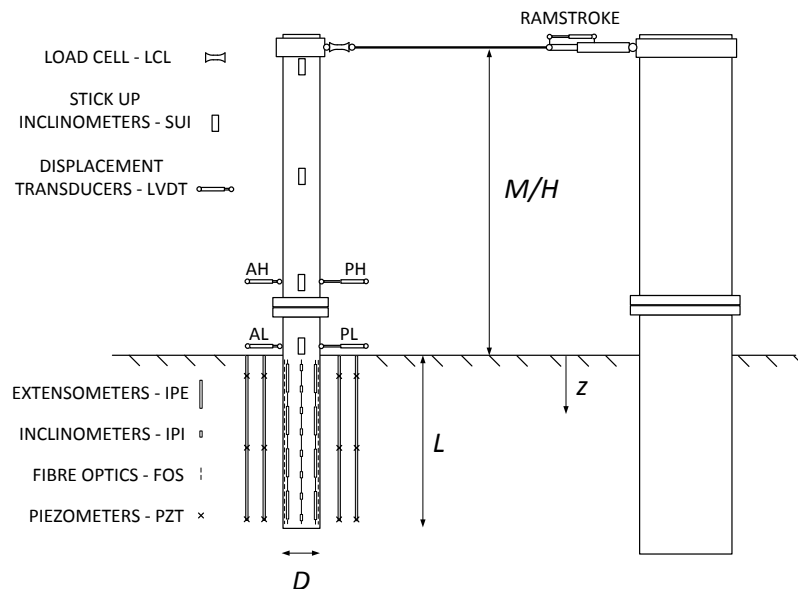

Figure 11: Instrumentation used in the PISA test programme (McAdam pers com.)

In total there were 14 test piles at each site. Three pile diameters were tested, 0.273m, 0.762m and 2m. The majority of tests were performed on the mid-size 0.762m diameter piles. The focus of the PISA project was on monotonic loading and tests were carried out as a series of maintain load steps, which

targeted specific displacements and included a main unload-reload loop after the 3rd load step. A limited number of additional load tests were performed to explore rate effects, cyclic loading and damping.

3.3. REDWIN Project

The REDWIN (REDucing cost in offshore WINd by integrating structural and geotechnical design) aims to bring together the structural and geotechnical analyses performed for offshore wind turbines. The project will develop integrated soil-structure interaction models, See Figure 12 that account for stiffness, damping, drainage, degradation and long-term behaviour. The foundation models being developed based on the concept of macro-elements, and soil interface models are based on the concept of distributed foundation springs such as generalized p-y models.
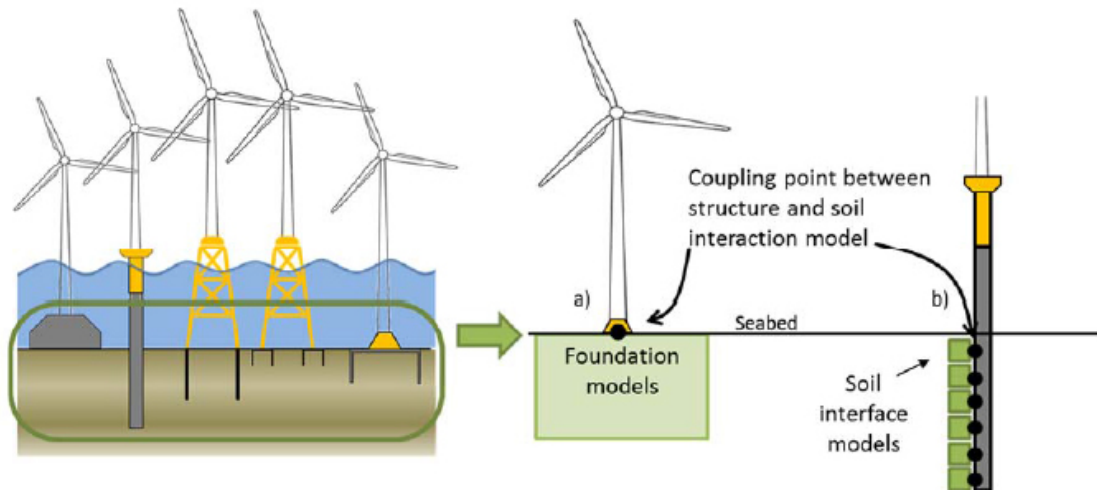

Figure 12: Illustration of soil interaction models: a) foundation model, and b) soil interface model (from www.ngi.no/en/Project-pages/REDWIN)

The project is led by the NGI and partners including research and industrial organisations; NTNU, Statoil, Statkraft, IFE and Olav Olsen AS. Technical work packages will include a work package aimed at compiling measurement data such as the natural frequency response of turbines in operation and data from model and full-scale pile tests from the literature. A work package will be dedicated to development of a library of 3D soil models capable of converting raw soil date into input data for advanced soil-structure interaction models. Work package four concentrates on implementing the 3D Soil models into a time domain aero-elastic code.

3.4. Vibro Project

Environmental concerns regarding pile driving is driving research into alternative installation methods as the noise associated with driving large diameter piles has been shown to affect sea mammals (Igoe et al 2013). Whilst vibratory installation is relatively common in the offshore setting, uncertainties regarding the potential effect of installation method on the capacity of monopiles were the driver for the Vibro project. The project was led by RWE and the consortium included major utilities; engineering consultants, pile manufactures, authorities, certifying body's universities and research institutes. The project involved a major field trial where six 4.3 m diameter, 18 m long, instrumented monopiles were installed in medium-dense sand at Cuxhaven, See Figure 13. Three of the piles were driven and three were vibrated in place. The piles were statically load tested in pairs (i.e. a driven pile and vibrated pile were loaded of each other) to assess the effect of installation on the lateral capacity and stiffness of the piles. An interesting feature of the project was that CPT tests were performed before and after pile installation.

Figure 13: Artist impression of test site in Cuxhaven. ©VIBRO-project by RWE

4.   ONSHORE PILING PROJECTS

4.1.   Introduction

In addition to new markets in offshore engineering a number of interesting research topics are ongoing in the onshore field. These relate for example to different pile installation methods with screw piles becoming increasingly used and problems such as the development of negative skin friction. Another topic that is currently being studied extensively are thermally activated pile foundations (so-called energy piles) for heating and cooling purposes. Finally, the mechanical behaviour of micropiles, a foundation system with a large potential in a wide range of applications, is also subject of research. Following sections discuss briefly the ongoing research on these topics.

4.2.   Screw piles

4.2.1.   Background

Screw piles are finding increased market share due to several advantages including; (i) the low-displacement nature of their installation, (ii) low noise and (iii) their installation speed. A major research project was performed by the Belgian Building Research Institute (BBRI) organized, with the financial support of the Belgian Federal Ministry of Economic Affairs (BBRI, 1998-2000 & 2000-2002) to investigate the behaviour of screw piles and to compare this to pre-cast piles. The research was performed at two sites. At the first site located in Sint-Katelijne-Waver, five types of screw piles and driven precast piles were installed in over-consolidated tertiary Boom clay. In total thirty pile load tests (including static, dynamic and Statnamic loading) were performed. A similar test campaign was performed at the second site in Limelette where the subsoil consists of quaternary silty layers (loam) and tertiary Ledian-Bruxellian sand. This summary of the work concerns results from the first test site.

4.2.2.   Site description and pile details

A major field investigation programme include in-situ CPT, pressuremeter, SPT, DMT and geophysical testing was performed. This was augmented with laboratory tests including triaxial strength tests with bender elements. Huybrechts (2001) gives details of the installation procedure for the six different types of piles were installed and tested, five cast-in-place screw piles and one pre-cast driven pile;

- Atlas pile, installed by the company Franki Geotechnics B
- De Waal pile, installed by the company De Waal Palen
- Fundex pile, installed by the company Fundex
- Olivier pile, installed by the company Olivier
- Omega pile, installed by the company Socofonda

4.2.3.   Load Tests

For each test site a series of static, dynamic and kinetic pile load tests were performed that included:

- Instrumented static load tests (two tests per pile type at each site);
- Dynamic pile load tests (two tests per pile type on each site and a second series of twelve pile load tests after one year on the Sint-Katelijne-Waver test site, in order to study the time effect);
- Statnamic load tests (one test per pile type on each site)

The results of the static pile load tests are summarized in figures 14, where the measured loads are normalised by estimates of the pile capacity made using CPT based design methods and the vertical displacement is normalised by the pile diameter. The load test programme proved that the screw piles had comparable capacities (considered at normalised displacement of 10%) to the driven pre-cast piles. Given the environmental and time savings achieved by screw piles this was a very positive outcome.
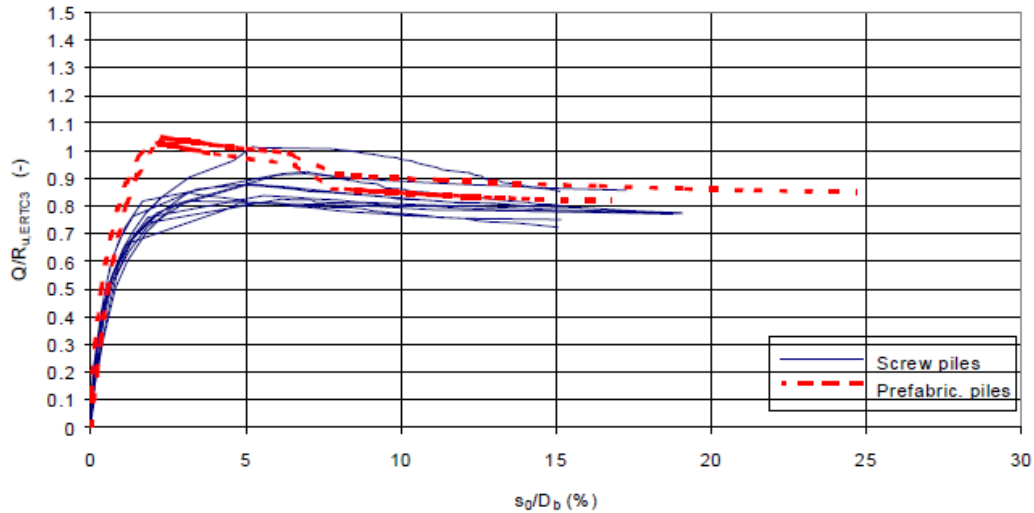

Figure 14: Normalised load settlement curves static pile load tests Sint-Katelijne-Waver

4.3.   Development of negative skin

4.3.1.   Background

In 2013 a research and development project was initiated by the Engineering College, Aarhus University in collaboration with Per Aarsleff A/S with the aim to further document the influence of bitumen coating and the development of negative skin friction on precast concrete piles installed in soft soils. As part of the project a novel full-scale pile test setup, involving instrumented piles and a ground monitoring program was established at Randers Harbour, Denmark.

4.3.2.   Site conditions and test setup

The test setup consists of four test piles (T) and five reaction piles (R) installed in a row with 3 m centre-centre spacing, as illustrated in Figure 15. Figure 15b shows details of the pinning of the test piles to the HE 280 M crossbeam, which enables measurement of the total down drag force on the pile using a standard load cell. The toes of the test piles are located in the soft organic post glacial marine Gytja, while the reaction piles are embedded into the underlying layers of marine sand. All piles are precast reinforced concrete piles with a square cross-section (0.25 m width). Bitumen coating was applied to test piles T1 and T3, while Piles T2 and T4 are uncoated.
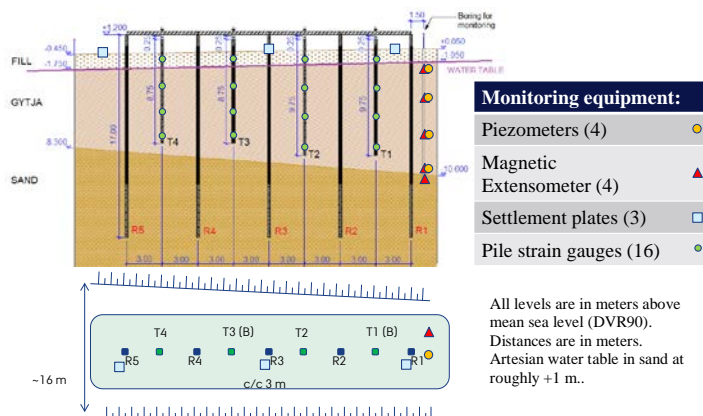

Figure 15a. Longitudinal section view and situation plan of the pile test setup before terrain loading (after Ventzel and Jensen,

Figure 15b. Photo showing pinning of test pile to cross-beam.

Multipoint vibrating wire piezometer (4 piezometers), settlement plates (3) and a magnet extensometer (4 probes) were used to monitor pore water pressures and settlements, while a total of 16 embedded vibrating wire strain gauges were cast into the test piles to measure internal pile strains. In addition to CPT and field vane testing carried out as part of the site investigation, laboratory testing (oedometer tests, interface shear tests) have been carried out to obtain soil and interface parameters, while the pile stiffness has been estimated from in-situ loading of the test piles in addition to tension tests on representative pile sections.

After pile installation and a period of rest (~ 200 days), 0.8 m of fill was placed around the piles to initiate settlement in the soft soil and hence to generate negative skin friction on the piles.

4.3.3. Preliminary key findings

Figures 16a shows the observed development in the total down drag force on the test piles until 315 days after ground loading (fill placement), while Figure 16b shows the relationship between measured total down drag forces and changes in the average excess pore water pressure in the top layers. Based on the preliminary observations the following key findings have been made.

- Significant reduction seen in total down drag force due to bitumen coating. A reduction greater than 80% is supported by the findings, cf. Figure 16b.
- The study has indicated that the negative skin friction may be significantly overestimated if calculation is carried based on an undrained analysis (α-method). The results suggest that a drained analysis may be more appropriate.
- The mobilisation of down drag forces on both coated and uncoated piles is seen to be proportional to changes in excess pore water pressure / effective normal stress on the pile.
- The on-going study has furthermore highlighted a high degree of uncertainty in the estimation of stress distribution on the test piles. The uncertainty stems from the use of a limited number of single point measurements of strains in addition to a significant influence of concrete shrinkage on the strain measurements and uncertainties about the concrete stiffness.
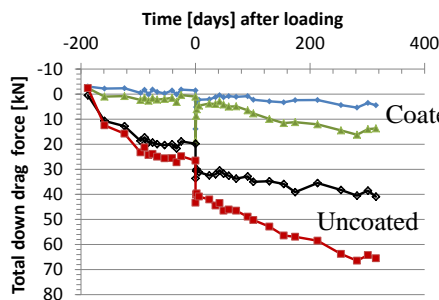

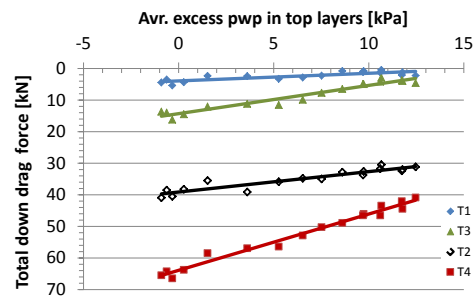*Figure 16a. Development of total down drag forces on test piles after loading*

Figure 16b. Changes in total down drag forces on test piles vs. changes in the average excess pore water pressure

Monitoring of the field test is still on-going, and the results are yet to be published. Preliminary and selected results have been presented by Sorensen (2015) and in this paper. To verify if the full down drag forces have been mobilised on the test piles further ground loading is planned.

4.4. Thermo-mechanical behaviour of energy piles

4.4.1. Background

Although the integration of heat exchangers in foundation piles to thermally activate them as shallow geothermal source has become common practice, the thermal and thermo-mechanical behaviour remains a topic of discussion. Former and current research work focuses on the short and long term energetic performance and thermo-mechanical behaviour in terms of bearing capacity and settlements.

4.4.2. Energy pile testing projects

Based on three studies in Austria (Brandl, 1998), Switzerland (Laloui et al., 2006) and the UK (Bourne-Webb et al., 2009) including in-situ tests, a simplified descriptive framework regarding the thermo-

mechanical performance of energy pile foundations has been developed (Bourne-Webb et al., 2013). This framework takes into account the effect of heating and cooling, pile shaft – soil interaction, mechanical loading and end-restraints. An example is shown in Figure 17, illustrating the influence of heating and cooling of a mechanically loaded, at both ends restrained foundation pile on the vertical load distribution and shaft friction.
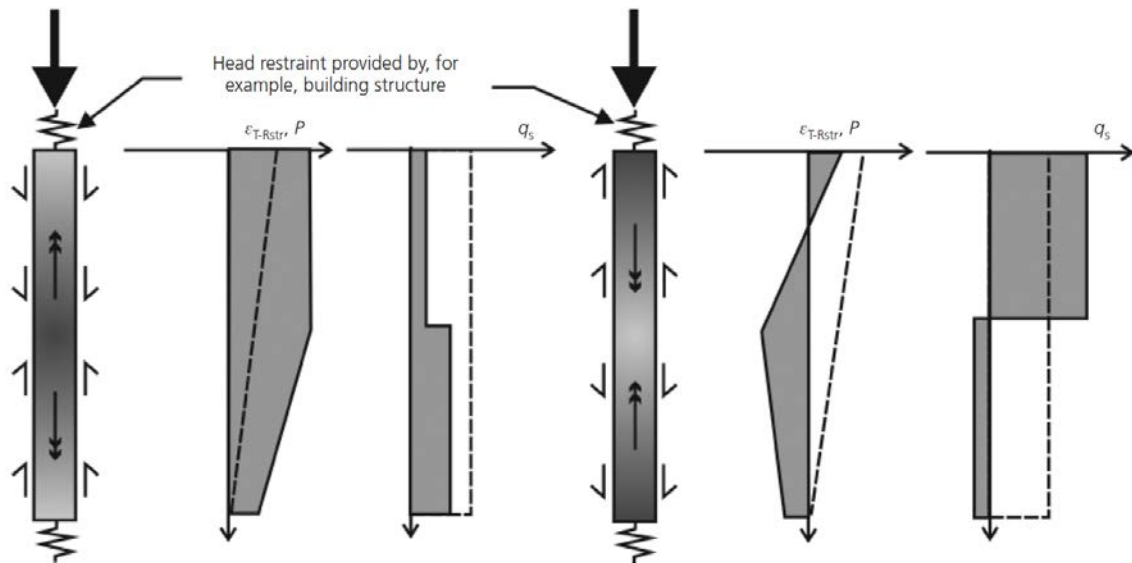

Figure 17: Impact of end-restraint on thermal load effects (in dashed line the effect of pure mechanical loading). (Left) Heating and both end restraints; (Right) Cooling and both ends restraints (from Bourne-Webb et al., 2013).

Meanwhile, several other energy pile test programs have been and are being carried out, for example in the USA, Australia, UK, France, Spain and Belgium. Figure 18 shows some energy piles before and after installation, as well as the test setup for mechanical loading in the framework of a research program in Ostend (Belgium). Small-scale and centrifuge testing and numerical modelling (Finite Element, Finite Difference, …) are performed as well aiming to improve the fundamental understanding of energy pile thermo-mechanical behaviour. An extensive overview of on-going research is given by Olgun and Loveridge (2015).

Figure 18: Energy piles installed for a BBRI test program in Ostend (Belgium). The heat exchangers are attached to the reinforcement cage prior to installation. The energy piles are fully monitored with strain and temperature sensors.

In all research projects the influence of several parameters is studied more detailed such as pile type, pile diameter, soil type and end-restraints. In future work, these results will be related to the existing framework, for example within the scope of the European COST Action TU 1405 *'European network for shallow Geothermal energy Applications in Buildings and Infrastructures (GABI)'*. This project also aims to propose design rules for energy piles, based on a benchmark about energy pile modelling and design

throughout Europe and the world. The workgroup meetings are open to everyone who is interested. In 2012, the Ground Source Heat Pump Association (GSHPA) already published a Thermal Pile Standard (GSHPA, 2012). This standard was based on the GSHPA borehole loop guide, but expanded to energy pile applications and already contains some design considerations.

4.5. Micropiles

Micropiles are widely used in areas that are difficult accessible for large drilling rigs. Typical applications are underpinning works, extension or reinforcement of existing infrastructure, stabilisation of slopes, tension piles for the lateral support of retaining structures, etc. Many installation methods and materials are available on the market and the bearing capacity of these elements depends strongly on the installation method, including grout injection parameters.

In order to tackle this problem of the bearing capacity of isolated micropiles a load testing program is going on in Belgium BBRI (2014-2016). The main objective of this research program is to link the execution and grout injection method to the bearing capacity of the micropile systems, to categorise these foundation elements and to translate the results in the framework of the Belgian semi-empirical design method according to Eurocode 7. Some first results were published in the Volume 3 of BBRI (2008).

Besides the behaviour of isolated micropiles, the group effect and the effect of cyclic loading is assessed in the research program by an experimental test set up as well as by 3D FEM numerical modelling (see figure 19). The publication of the results of this research are expected by the end of 2016 – begin 2017.
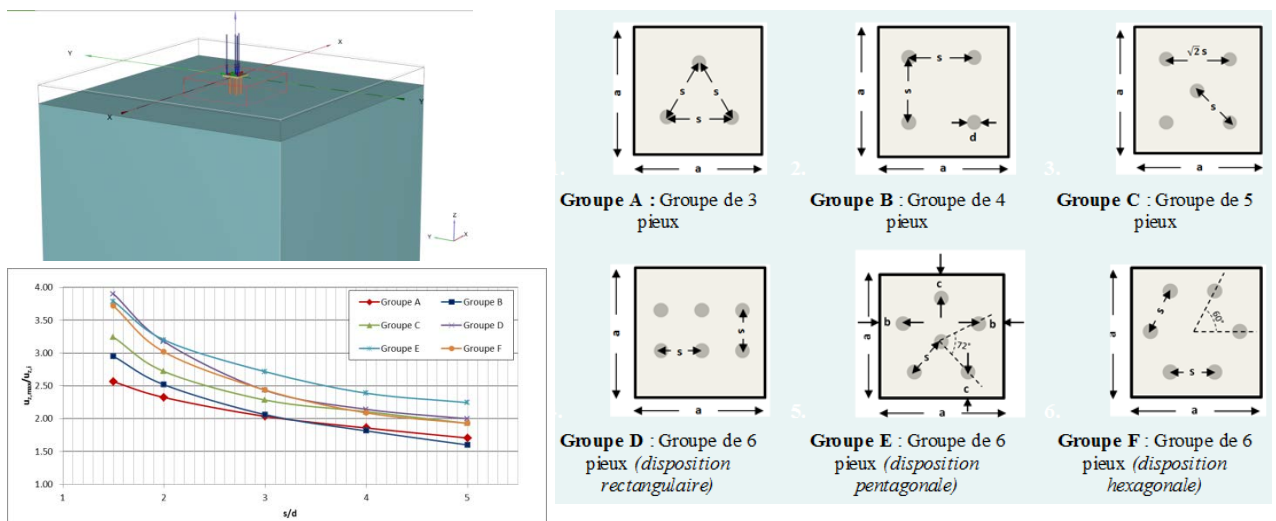

Figure 19: Group effect on micropiles - numerical 3D FEM simulations..

5.   CONCLUSIONS

Despite the relatively advanced state of the European piling industry many critical issues remain and these are being addressed through research and development projects. Issues such as new applications of existing technology, e.g. offshore wind developments, climate change and environmental concerns are the drivers for this research need. This paper provides a brief overview of projects been undertaken by members of ETC3. The positive collaboration between industry and academia in many JiP projects is an indicator of the importance of this ongoing work and many challenges remain.

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