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Field testing of axially loaded piles in dense sand

Essais sur le terrain des pieux dans du sable dense

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ABSTRACT: Large areas of the Netherlands are dominated by deep, soft soil deposits, posing a challenge to engineers with respect to the design of axially loaded foundations. The design of these foundations is primarily based on methods which use cone penetration test (CPT) parameters, such as that outlined in the Dutch national standard NEN 9997-1. A recent update to this standard included revised reduction factors for the pile base resistance. However, it is believed that this update is overly-conservative in certain design situations, leading to increasing cost, environmental impact and difficulty of installation. As a result, a national research project was initiated to enhance the understanding of pile-soil interaction effects and their influence on pile design methods. As part of this project, a series of large-scale field tests on fully instrumented piles in dense sand has been executed and are being analysed with a view to refining the national standards and contribute to the existing knowledge base on piles worldwide. This paper shall provide an overview of two pile test sites that have been developed as part of the programme, along with the design of the testing programme and an overview of the results.

RÉSUMÉ : Des vastes étendues des Pays Bas sont dominées par des dépôts des sols profonds et mous, ce qui posent un défi aux ingénieurs concernant la conception des fondations sous des charges axiales. La conception de ces fondations est basée sur des méthodes qui utilisent les paramètres de l'essai de pénétration au cône (CPT), tels que ceux décrits dans la norme nationale néerlandaise NEN 9997-1. Une récente version de cette norme a inclus des coefficients révisés pour la résistance de pointe des pieux. Cependant, on pense que cette version est trop prudente dans certaines situations de conception, entraînant une augmentation des coûts, de l'impact environnemental et des difficultés d'installation. Un projet de recherche national a été lancé pour améliorer la compréhension des effets de l'interaction entre les pieux et le sol, et leur influence sur les méthodes de conception. Dans le cadre de ce projet, une série d'essais sur le terrain à grande échelle sur des pieux entièrement instrumentés dans du sable dense a été réalisée et est analysée en vue d'affiner les normes nationales et de contribuer à la connaissance existante sur les pieux dans le monde entier. Cette article offrira un aperçu de deux terrains d'essai des pieux qui ont été développés dans le cadre de cette programme, ainsi que la conception du programme d'essai et des résultats.

KEYWORDS: pile design, fibre optics, field testing, cone penetration test, load testing

1 INTRODUCTION

The deltaic nature of the Netherlands has resulted in the prevalence of thick, highly compressible soil deposits across much of the country. In the major Dutch cities, such as Amsterdam or Rotterdam, most structures require long piled foundations extending through these soft soil deposits and terminating in a dense to very dense sand formation deep below the subsurface.

To assure the safety and reliability of these foundations, pile design in the Netherlands is guided by the national standard NEN 9997-1 (Netherlands Standardisation Institute 2017a). The standard prescribes a CPT-based pile design method and uses the cone tip resistance q_c to derive the shaft and base capacities of the pile. The reduction factors α_p and α_s are then applied to obtain the base resistance q_b and shaft resistance q_s respectively. The basic form of this method can be summarised as follows:

$$q_b = \alpha_p q_{c,avg} \tag{1}$$

$$q_s = \alpha_s q_c \tag{2}$$

For the calculation of the base resistance, $q_{c,avg}$ is determined using a weighted averaging method proposed by van Mierlo & Koppejan (1953), frequently referred to as the Dutch method. This averaging method takes into account the cone resistances over a zone of 0.7*D* to 4*D* below the pile tip and 8*D* above the pile tip (where *D* is the pile diameter or the pile equivalent diameter). For the shaft resistance, an arithmetic mean is taken of all q_c values measured across the soil layers in question. The reduction factors α_p and α_s are largely dependent on the pile's interface and method of installation (e.g. soil-replacing, soil-displacing or a combination thereof). An example of these for a range of pile types is provided in Table 1.

Table 1: Reduction factors for selected pile types from NEN 9997-1

Pile type	α_p	α_s
Bored pile	0.35	0.006
Continuous flight auger	0.56	0.006
Driven cast-in-situ	0.70	0.014
Driven precast	0.70	0.010
Screw injection	0.63	0.009
Steel tubular (open-end)	0.7	0.006

A recent update to NEN 9997-1 resulted in a 30% reduction to α_p across all pile types. This has caused debate within the Dutch foundation industry due to the limited amount of pile failures observed in practice. Subsequently, it is believed that the design method is overly conservative for many different geotechnical conditions and results in a foundation design that is economically inefficient and environmentally unsustainable.

This has resulted in the formation of the InPAD (**In**vestigation of the Axial Capacity of **P**iles in S**and**) project. This project aims to investigate a number of hidden safety factors pertaining to design methods such as NEN 9997-1, in addition to enhancing the wider understanding of pile behaviour. The project commenced in October 2019 and is scheduled to run for four years. It is focused on investigating several aspects of pile behaviour including, but not limited to:

(i.) The impact of friction fatigue on the distribution of α_s with depth for soil-displacing piles.
(ii.) Determination of accurate pile base resistances that include the effect of residual loads and therefore allow consistent α_p factors.
(iii.) Whether limiting q_c values are necessary for estimating the shaft resistance.
(iv.) If limiting values on the pile base resistance are necessary.
(v.) Appropriateness of averaging methods for obtaining the design cone resistance $q_{c,avg}$ and correspondingly, the pile base resistance.
(vi.) Assess the impact of ageing on pile capacity.

The output of this project will be used to refine the Dutch pile design method and provide an impetus for the improvement of pile design methods worldwide. As a result, it is envisaged that this project will give rise designs which are more economically and environmentally sustainable, whilst maintaining a high level of safety and reliability.

1.2 *Research Approach*

Three work packages form part of the InPAD project. The first work package involves the execution of full-scale axial pile load tests on at least three different pile types: driven precast, driven cast-in-situ (DCIS) and screw injection (SI) piles. Thus far, tests on all three of these pile types have been carried out in the Netherlands at Maasvlakte and Delft. An overview of both test sites, test setups and initial findings shall be provided in this paper.

The second work package focusses on the physical modelling of piles in the laboratory. One aspect of this work involves centrifuge testing to investigate the behaviour of a model pile across a wide range of cone resistances. The aim of these tests is to delineate the effects of particle crushing underneath the pile base along with the influence of the chosen loading scheme and load-holding periods of a pile test. Another series of centrifuge tests are being executed in layered soils using a variety of different pile diameters in order to analyse the performance of different CPT averaging methods, including a new averaging method proposed by TU Delft (de Boorder 2019). Another part of this work package is also being carried out using a calibration chamber to compare the effects of pile jacking and pile driving.

The third work package involves the use of numerical and statistical models. As part of this, the effect of layering on the pile base capacity is being investigated using finite element modelling, with the aim of resolving the appropriateness of new or existing averaging methods. Particular emphasis is also being placed on the use of reliability-based or probabilistic models for assessing pile capacity in order to better account for uncertainty present in pile design and pile testing. In this regard, a comprehensive investigation into the effect of inherent soil variability on pile design is being carried out. This is crucial for situations where information is limited, for instance with respect to the number of CPTs, pile tests or boreholes performed at a site. To support this analysis, a database of existing pile tests throughout Netherlands and worldwide is being compiled.

This paper shall discuss the first work package involving the field tests on full-scale piles. As a background, a brief overview of Dutch geology will be given, followed by a factual description of the test sites in Delft and at Maasvlakte. In both cases, a synopsis of the results will be given, although it should be noted that the detailed assessment of the results is still ongoing at the time of writing.

Figure 1. Distribution of the Kreftenheye Formation across the Netherlands. Adapted from Tolsma (2018).

2 DUTCH GEOLOGY & PILE DESIGN

A large amount of the Netherlands' geomorphology has been dictated by the Rhine—Meuse—Scheldt delta. Traversing the central to southern part of the country, this river system has deposited a variety of soil formations that have had a significant influence on the geotechnical design of foundations across the country.

Of these formations, a fluviatile late-Pleistocene to early-Holocene era sand formation known as the Kreftenheye Formation comes to the fore with respect to pile design. The formation is practically ubiquitous across western and central Netherlands (Figure 1), particularly in the Randstad area – a megalopolis where almost half of the Netherlands' population reside. The area includes major cities such as Amsterdam, the Hague, Rotterdam and Utrecht, in addition to vital infrastructural centres such as the port of Rotterdam or Schiphol Airport. As a result, the area is under persistent development and is highly dependent on the economical and efficient design of piled foundations.

In general, the Kreftenheye Formation is a coarse sand, frequently gravelly, varying from medium dense to very dense across its distribution. Notably high cone resistances in the formation are prevalent in the far west of the country towards the coast. For example at Maasvlakte, cone resistances typically range between 40-60 MPa, with peaks of up to 80 MPa (e.g. Figure 2). Moving inland, towards the centre of the country, these resistances reduce to between 10-30 MPa. Throughout both of these regions, the upper boundary of the formation is frequently threaded with channel belt deposits (Hijma 2019) which may locally increase or decrease the cone resistance depending on the filling material. Progressing further east across the country, towards the city of Nijmegen, the upper boundary of the Kreftenheye approaches ground level, with cone resistances generally between 20-40 MPa.

Given the critical nature of this formation for foundation design, pile testing within the InPAD research programme has been carried out on piles founded in this Kreftenheye Formation. The first pile test site at Maasvlakte targets the extremely high cone resistances which are characteristic of the area. The second test site at Delft, targets a region of lower cone resistances, providing insights into the variability across the formation and its effect on piles and pile tests. A laboratory investigation is also underway with regards to the formation's characteristics, such as the particle morphology and mineralogy.

The choice of these test sites is also in line with the stipulations of the Dutch guidance document for pile tests, NPR 7201 (Netherlands Standardisation Institute 2017b), with regards to adjusting the reduction factors, the number of test sites required and the variations between these sites.

3 MAASVLAKTE PILE TESTS

The tests at Maasvlakte were the first series of field tests executed as part of the InPAD programme. Carried out in late-2019 and early-2020, tests were performed on three driven precast piles, four DCIS piles and four SI piles, outlined in Table 2. The tests form part of the design of a large quay wall development around the harbour of Amaliahaven. The quay wall is to consist of an anchored combi-wall with between 2000-3000 foundation piles, comprising of a mix of both DCIS and SI piles. The site-specific design method which has been developed from the pile test results is envisaged to save up to 50% in both carbon emissions and financial cost for the project's foundation design and installation.

An extensive ground investigation programme has been carried out at the pile test site, with one CPT executed on the central axis of each test pile and at least three CPTs within a radius of two metres from each of these piles. In addition, three CPTs were executed after installation around one of each pile type. As outlined before, the CPT profile of the test site (Figure 2) is particularly distinct due to the extremely high peaks of cone resistances throughout the Kreftenheye Formation, whose upper boundary is located at a depth of twenty-eight metres below the surface. Across this formation, numerous weak lenses are present and in general, the soil variability across the site appears to be relatively high. The overlying formations consist of intermixed clays and sands between 17-28m depth, with the layer above consisting of dredged sand, deposited from the extension of Maasvlakte ten years ago.

2.1. *Pile Geometry & Installation*

Throughout the installation process, measurements were recorded for each pile, such as blow count, torque or rate of fluidisation. The 400 mm square precast piles were driven to depths of 31.25 m to 31.75 m. During installation, water fluidisation was carried out to a depth of approximately 25 m to reduce the number of blows imparted on the piles and consequently, reduce the risk of damaging the piles. A jetting tube placed along the central axis of each pile facilitated this.

The DCIS piles were installed using a reusable steel auxiliary tube, 380 mm in external diameter. A 480 mm diameter sacrificial base plate was fitted to the bottom of the auxiliary tube, creating an offset of 50 mm between the outer edge of the auxiliary tube and the base plate. This prevented the influx of soil and water inside the tube during the installation process. Once the target depth was reached, a steel reinforcement cage was placed within the empty auxiliary tube and then the tube was withdrawn using a reverse hammering action, leaving the base plate in place at the bottom of the pile. Just before withdrawal of the tube, the tube was partially filled with concrete, with more concrete being poured during the withdrawal of the tube. All four DCIS piles tested were approximately 32.5 m in length.

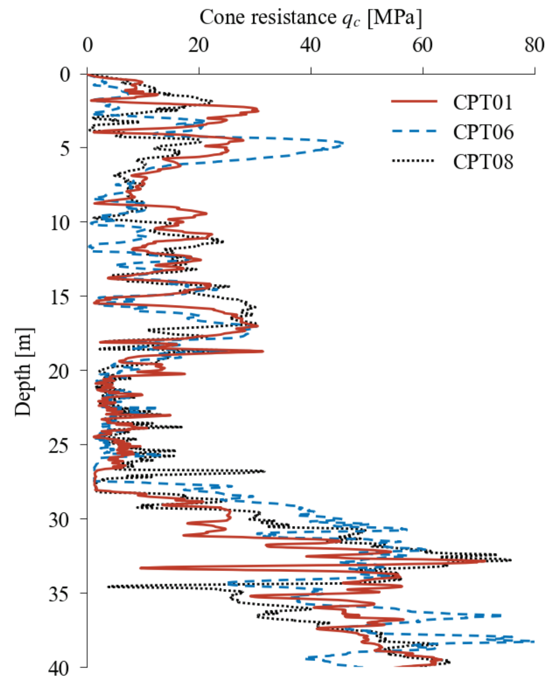

Figure 2. Selected CPT profiles from across the Maasvlakte test site

Lastly, the SI piles consisted of a steel tube with a screw tip welded to the bottom of the tube. Across the full installation depth, grout was jetted through outlets at the screw tip, the flow and water-cement composition of which was controlled and monitored during installation. Once the target depth was reached, the steel tube was filled with concrete. Two of the SI piles (SI1 and SI2) were 37 m in length, with SI3 and SI4 founded at 35 m and 34 m depth respectively. The different lengths were chosen in order to investigate a region of low cone resistance and provide a lower bound estimate of the pile capacity for the piles at the quay wall.

Further details of these test piles are shown in Table 2.

3.1. *Load Frame & Instrumentation*

Critical to the planning of the test programme, was the design of a test frame which could develop loads of up to 25 MN to bring the piles to their failure capacity. As a result, a custom load test frame was designed for the Maasvlakte site. The frame, shown in Figure 3, consisted of a steel reaction frame tied in by up to twelve grout anchors, each inclined 8° away from the pile to avoid imparting extraneous stresses on the pile. Six hydraulic jacks used this frame as a reaction force, allowing them to generate compressive force on the pile. Six load cells were placed on top of these jacks to measure the forces applied to the pile. Four linear variable differential transducers (LVDTs) were placed around each pile, measuring the pile head settlement with respect to a ladder-type reference frame.

Strains in the pile were measured using two different fibre optic analysis techniques: Brillouin Optical Frequency-Domain Analysis (BOFDA) and Fibre Bragg Grating (FBG). The former provided a continuous strain profile along the pile shaft over a time interval of four minutes whilst the latter provided discrete strain measurements along the pile shaft, with a time interval of every ten seconds.

Figure 3. Pile test frame and the reaction anchors used at the Maasvlakte test site

In the case of the driven precast piles, both fibres were installed within the pile during pile fabrication. Temperature change within the pile was measured using Raman spectroscopy, allowing for the temperature compensation of the fibre optic readings so that appropriate residual load readings could be obtained. These residual loads were measured at the start of the pile test, with a reference measurement made as the piles lay horizontally on the site prior to their installation.

For the SI piles, the fibres were placed within two small reservation tubes after installation. These tubes were welded to the inside of the steel tube of the SI pile during pile fabrication. After the placement of the fibres within the reservation tubes, the tubes were filled with grout to ensure the full transfer of strain between the soil, grout and steel of the SI piles. Likewise, the instrumentation of the DCIS piles was also installed on-site by affixing the fibre optic cables directly to the reinforcing cage. Measurements of the DCIS and SI piles were taken from the start of the pile test itself, with the readings being zeroed at this point.

3.2 *Test Procedure*

The pile test was carried out in line with the stipulations of NPR 7201. Each test comprised of at least eight load steps to the predicted pile capacity. During each load-holding period, a set of creep criteria dictated the duration of the load-holding period and the magnitude of the subsequent load step.

With the exception of DP1, which featured an unload/reload cycle after each load step, unload/reload cycles were not regularly incorporated into the test schedule. However, a cycle was occasionally implemented in an attempt to mobilise further base resistance of the piles whilst remaining within the permissible capacity of the load test frame. A cycle was also carried out in the event of an unsafe situation, for example, if the

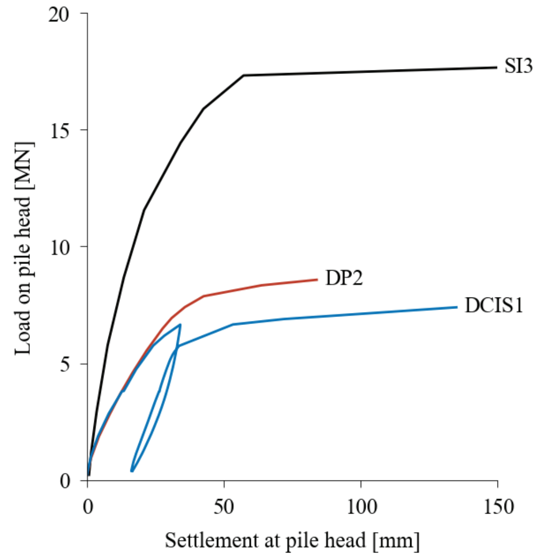

Figure 4. Load-settlement response of selected piles at the Maasvlakte test site

test frame was excessively inclined and the frame or the instrumentation needed adjustment at zero-load.

The failure criterion for all piles was defined as when the pile base displacement reached 10% of the (equivalent) pile diameter.

3.3 *Results*

An overview of the results is shown in Table 2, in addition to selected load-settlement curves from each pile type in Figure 4. At the time of writing, the strain readings are still being assessed to quantify the distribution of load with depth.

In the case of the driven precast piles, tests on two piles (DP1 and DP2) were carried out within a couple of days of each other, one month after installation. The third and final test on the driven precast piles was carried out on pile DP3, executed two and a half months after installation to investigate the effect of aging on pile capacity. No ostensible difference in the failure capacity is evident between the three piles, however all three exceed the design capacity derived by the design method in NEN 9997-1.

The SI piles developed the highest capacities of all three pile types due their size and length, with SI2 reaching the highest capacity of the four SI piles at 21.2 MN. As expected, the shorter piles SI3 and SI4 developed the lower capacities, albeit in excess of the design capacity.

Table 2. Synopsis of the results from the Maasvlakte test site. Limiting resistances on the piles' base and shaft capacities have been included in the design capacity and is based on the CPT along the central axis of the pile

Pile	Days between installation & testing	Pile length (m)	Outermost diameter or edge length (m)	$q_{c,avg}$ (MPa)	Design capacity (NEN 9997-1) (MN)	Measured capacity (MN)
DP1	28	31.74	0.40	33.75	7.27	7.9
DP2	30	31.29	0.40	31.86	6.98	8.6
DP3	78	31.80	0.40	30.14	7.15	8.3
SI1	43	37.02	0.85	24.78	17.53	16.3
SI2	49	37.06	0.85	36.44	19.24	21.2
SI3	78	34.98	0.85	16.76	16.33	17.3
SI4	50	34.06	0.85	11.79	14.33	16.4
DCIS1	59	32.54	0.48	18.53	9.56	8.3
DCIS2	34	32.49	0.48	38.25	9.56	8.7
DCIS3	50	32.50	0.48	30.29	8.99	6.9
DCIS4	52	32.50	0.48	37.79	8.79	8.0

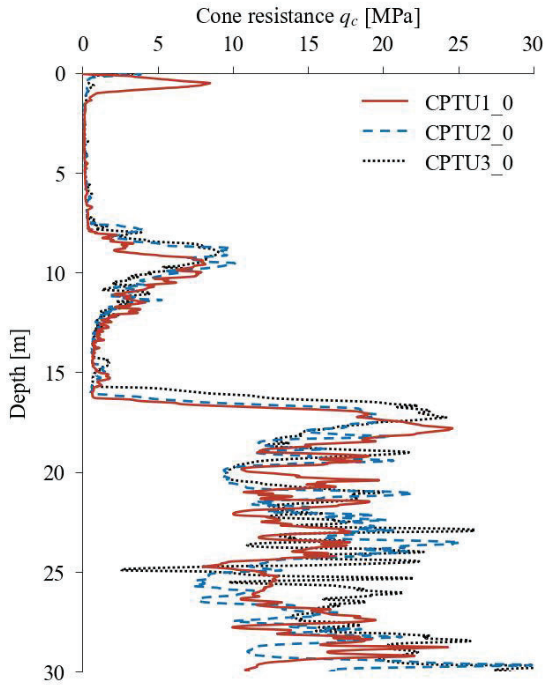

Figure 5. CPT profile at the Delft test site. The three piles are founded at approximately 20.5 m depth

Throughout all four DCIS piles, a significant amount of scatter in the strain readings was observed in both the BOFDA and FBG strain readings across the bottom four metres of each pile. It is postulated that this is as a result of concrete segregation in the piles, that is, the separation of certain aggregate sizes from the cement mix. Nonetheless, none of the piles were extracted following the Maasvlakte test programme and the precise cause of this segregation is still being investigated. With respect to the piles' total capacities, all four of the piles had a measured capacity less than their design capacity, with DCIS1, DCIS2 and DCIS4 reaching a capacity of around 8-9 MN. DCIS3 had the lowest capacity of all four piles at 6.9MN.

4 DELFT PILE TESTS

In the latter stages of 2020, a pile test site in Delft was created for the second series of tests on driven precast piles. These piles were each approximately 21 m in length and 350 mm square. Further details of these piles are shown in Table 3.

Similar to the Maasvlakte test site, four CPTs were executed at each pile location prior to installation (Figure 5), in addition to a series of CPTs executed after installation. Immediately underlying the site are soft clay and peat deposits, characteristic of western and central Netherlands. Underneath this, lies an intermediary sand layer between 8-12 m depth, followed by a bed of interlaminated silt and clay deposits. The Kreftenheye Formation starts at 16 m. The upper part of this formation, within which the piles were founded, features cone resistances of 15-20 MPa, trending higher with depth. The pile tips were located in a slightly weaker zone of sand at a depth of 20-21m, with a cone resistance of approximately 10MPa. As a result, this site provides a similar geological formation as the Maasvlakte test site albeit with much lower cone resistances in the founding layer.

4.1 *Load Frame & Instrumentation*

A deadweight load test frame system was used at the Delft test site. This comprised of a steel beam upon which concrete blocks were placed (Figure 6). One hydraulic jack was used to develop the force on the pile head, coupled with one load cell placed between the jack and the frame. The resulting pile head settlement was then measured by four LVDTs placed around the pile head, measuring with respect to a reference frame. Furthermore, a direct measurement of the pile base settlement was also facilitated for through the use of a small steel rod known as a telltale, running all the way down a reservation tube placed within the pile itself.

Fibre optic cables were installed on the central reinforcing bar of each face of the pile, allowing for the assessment of bending effects in the piles during the load test. Similarly to Maasvlakte, temperature compensation was also carried out on the strain readings, albeit with loose-tube telecommunication cables. These cables theoretically isolate the optical fibre from mechanical strain so that the fibre is only influenced by changes in temperature. The change in thermal strain can then be assessed using the same BOFDA data logger as that used for the strain readings. To support these measurements, thermistor modules were also placed within the pile.

In addition to the BOFDA measurement system, multiple data loggers with different interrogation techniques (e.g. BOTDR, BOTDA and COTDR) were also experimented with to offer a comparison of the different systems and analyse their benefits and drawbacks for the purpose of strain monitoring in piles.

Figure 6. Deadweight test frame system used at the Delft test site

4.2 *Test Procedure*

A similar loading procedure to that at the Maasvlakte test site was followed, that is, in line with the stipulations set out in NPR 7201. Similarly, the failure criterion was defined as when the pile base settlement exceeds 0.1*D*.

At the end of each test, up to five unload/reload cycles to the failure load were executed to investigate the effect of load cycling on pile shaft friction. In the case of piles P2 and P3, a retest was also carried out a number of weeks after the initial test to investigate the effect of aging on the capacity of the pile. These retests loaded the pile incrementally to the failure capacity attained in the initial tests, with the test concluding when excessive levels of creep were experienced in the load-holding periods.

4.3 *Results*

Testing of the three piles was completed in February 2020. A brief synopsis of the results is shown in Table 3, in addition to load-settlement curves for the first test on each pile, shown in Figure 7. The first pile to be tested, pile P3, was tested three days after installation and reached the failure criterion at a load of 1.58 MN. In contrast, the subsequent retest carried out two weeks after the initial test, reached a capacity 0.17 MN greater than the initial test. Pile P2, which took place three weeks after installation, measured the lowest capacity of all three piles with just 1.3 MN attained during the initial test. Similar to pile P3, the retest on pile P2 developed an additional capacity of 0.2 MN between pile tests. Nonetheless, it should be noted that in the case of both

Table 3. Brief synopsis of the driven precast piles and the results from the Delft test site. Limiting resistances on the piles' base and shaft capacities have been included in the design capacity and is based on the CPT along the central axis of the pile

Pile	Days between installation & testing	Pile length (m)	Outermost diameter or edge length (m)	$q_{c,avg}$ (MPa)	Design capacity (NEN 9997-1) (MN)	Measured capacity (MN)
P1	91	20.32	0.35	11.36	2.06	1.80
P2	1st test: 22 Retest: 26	20.75	0.35	9.39	1.88	1st test: 1.30 Retest: 1.50
P3	1st test: 3 Retest: 19	20.65	0.35	9.58	1.90	1st test: 1.58 Retest: 1.75

piles, approximately 60% of the failure load was sustained on the piles in order to minimise the settlement of the test frame footings.

The last pile to be tested, pile P1, was designed as an aging test and was tested three months after installation. The resulting capacity was higher than that of the other piles, with 1.8 MN being applied to the pile at failure. No retest was carried out on pile P1.

It appears aging has occurred on the pile capacity between installation, testing and retesting and this is currently a key focus of the data analysis. It is expected that the strain readings will give a detailed insight into the mobilisation of the shaft and base resistances in the piles, particularly with regards to their development over time and across which sections of the piles these strains develop.

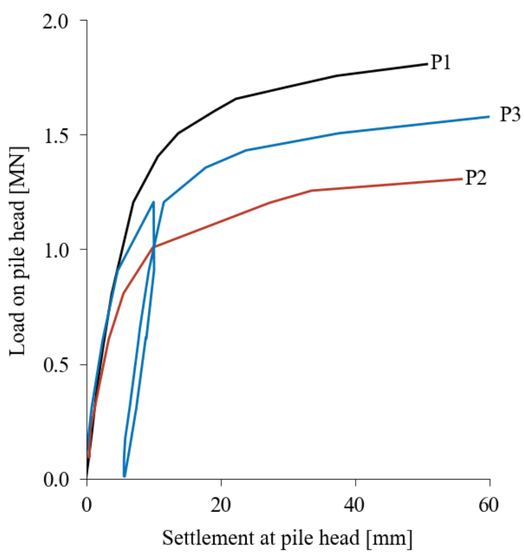

Figure 7. Load-settlement response of piles at the Delft test site during the first test

6 CONCLUSIONS

This paper has discussed a method of assessment for the refinement of design methods for piles founded in dense to very dense sand. This is being carried out using full-scale field tests, laboratory testing and advanced statistical techniques. So far, a series of pile tests at two different test sites have been carried out in the Netherlands. Three different pile types have been tested and every pile has been extensively instrumented in order to assess their response to axial compressive loading. The results are currently being compiled and analysed, with a view to enhancing the understanding of numerous aspects associated with pile behaviour.

These results are expected to give extensive insights into the response of the piles. For instance, these results will help delineate the effects of phenomena such as friction fatigue, pile aging and residual load development. Furthermore, the results will also provide a means of making adjustments to the existing Dutch pile design method, particularly with regards to the appropriate reduction factors, limiting resistances and averaging method. It is hoped that this will provide guidance into creating a design method which is economically and environmentally sustainable, whilst maintaining a high level of safety and reliability.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

De Boorder, M. 2019. *Development of a new CPT averaging technique and review of existing CPT based methods for the calculation of total pile capacity*. MSc Thesis. TU Delft.

Hijma, M. P. 2019. *From river valley to estuary: the early-mid Holocene transgression of the Rhine-Meuse valley, the Netherlands*. PhD Thesis. Utrecht University.

Netherlands Standardisation Institute 2017a. *NEN 9997-1+C2:2017 Geotechnisch ontwerp van constructies - Deel 1: Algemene regels*. Delft, The Netherlands, (in Dutch).

Netherlands Standardisation Institute 2017b *NPR 7201:2017 - Geotechniek - Bepaling van het axiaal draagvermogen van funderingspalen door middel van proefbelastingen*. Delft, The Netherlands, (in Dutch).

Tolsma S.P. 2018. *Kreftenheye Formation: Distribution map v1.0*. TNO, Utrecht, The Netherlands.

Van Mierlo, W. and Koppejan, A. 1953. Lengte en draagvermogen van heipalen. *Bouw* 3, (in Dutch).