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Application of axial load tests in the Netherlands to offshore pile design

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ABSTRACT: This paper describes axial load tests on three full-scale driven precast piles in the Netherlands. The piles were founded in dense to very dense river-deposited sands, a soil that is widespread across the Dutch North Sea sector. The deposit is characterised by cone penetration test (CPT) tip resistances of up to 90 MPa and offers a detailed insight into pile response in realistic offshore conditions. Each test pile was incrementally loaded to failure under compression, while fibre optic sensors measured the changing deformation of the pile. The analysis and interpretation of the load test data focussed on how the three slender piles behaved at large shaft and base resistances. Notably, the piles mobilised base and shaft resistances greater than currently prescribed limiting resistances in design standards, thereby highlighting some overconservatism present when designing piles in dense sand.

1 Introduction

The new ISO/API design method (Lehane et al., 2020) has changed how driven piles under axial loading are designed offshore. To calibrate the design method, high quality static load tests were compiled (Lehane *et al.*, 2017), building on previous databases such as the ZJU-ICL database (Yang et al., 2015) and the UWA database (Schneider et al., 2008). However, these databases still lack data from very dense sands, primarily because of the difficulty in safely and economically mobilising the required failure loads in these conditions.

This data gap has created uncertainty regarding pile behaviour in very dense sand. In response, many design methods cautiously limit the amount of resistance that a pile can mobilise. While this can be a pragmatic response to the unknown, it can also introduce overconservatism into the design. This in turn, increases the financial and environmental cost of pile fabrication whilst increasing the likelihood of pile damage and pile refusal during installation.

To investigate pile response in very dense sand, a test site was established at the port of Rotterdam in the Netherlands. Three driven precast piles were installed at least eight pile diameters into a dense to very dense sand layer. Each pile was instrumented along their full length with fibre optic sensors, giving a detailed insight into the pile shaft and base response. This paper presents the load tests performed on these piles, bringing more certainty to designers and contractors when designing piles in dense onshore and offshore sand deposits.

2 Limiting resistances in design

Divergence in national and international design standards has led to different calculation approaches for dense to very dense sand. In general, many design standards limit the calculated base resistance q_b and shaft resistance q_s, (e.g. Table 1). Some standards, such as in the Netherlands, impose a strict limitation. Other design methods, like in China or the older ISO/API method, adjust the limiting resistance based on the relative density or the CPT cone tip resistance q_c. Furthermore, Table 1 does not convey some of the implicit limitations in design methods. For example, a weighted average of cone resistances around the pile base is used to determine the design cone resistance $q_{c,avg}$ and correspondingly, the pile base resistance. Many different algorithms have been proposed to determine this $q_{c,avg}$ (e.g. van Mierlo and Koppejan, 1953; Boulanger and DeJong, 2018). However, each algorithm treats the high cone resistances differently and often limits the base resistance in a way that is not explicitly described in design standards.

Figure 1: Lifting the load test frame into place at the Amaliahaven test site

The origins of limiting resistances are partially rooted in the critical depth theory (Poulos, 2001). In a set of full-scale (Vesic, 1970) and model pile experiments (Kerisel, 1961; Robinsky and Morrison, 1964; Vesic, 1965), it appeared that the average shaft resistance reached a limiting value for depths more than twenty pile diameters from the top of the pile. However, the theory has since been heavily refuted (Kulhawy, 1984; Kraft, 1991; Fellenius and Altaee, 1995), including by the original author themselves (Kulhawy, 1996). This was because installation-induced residual stresses and scale effects of the model tests were not fully assessed at the time. Other uncertainties, such as apparent diameter-dependent scale effects (Chow, 1997; White and Bolton, 2005), contributed to the apprehension over pile response at high resistances.

In the Dutch design code, alternative reasons were cited for implementing limiting resistances. One concern was that pile driving could reduce the high horizontal stresses in overconsolidated soils and therefore reducing the amount of base resistance available (te Kamp, 1977). At the time, precast piles were also not prestressed during manufacturing. This meant that piles had to meet minimum size requirements to avoid damage or buckling during transportation, staging and installation. Because of these requirements, reaching test loads beyond the prescribed limiting resistances was practically inconceivable at the time (te Kamp, personal communication).

Indeed, the geotechnical and structural design of piles has since progressed significantly and modern design approaches are gradually shifting away from strict limiting resistances (Fleming et al., 2008). However, the dearth of high quality full-scale load tests has meant that designers and contractors must approach these dense to very dense soil conditions with caution.

Table 1: Limiting resistances for driven precast piles in CPT-based design codes. Where the limitation is dependent on the relative density or q_c, the value in very dense sand has been presented

Location	Standard	Base [MPa]	Shaft [kPa]
Belgium	NBN-EN 1997-2	None	150
China	JGJ 94-2008	Non-linear reduction	125
France	NF P 94-262	None	150
Netherlands	NEN 9997-1	15	150
Offshore	API RP 2A	12	115
Offshore	ISO/API	None	None

3 Amaliahaven pile test site

Dense sands have been a persistent challenge in the port of Rotterdam (de Gijt and Broeken, 2013). In 2013, the port was extended into the North Sea by the creation of the Maasvlakte II peninsula. The extension opened a large amount of land for development and many kilometres of deep-sea quay walls are now being designed and constructed. These large earth-retaining structures require thousands of foundation piles and so even minor adjustments to their design can be hugely beneficial, both financially and environmentally.

To investigate the pile behaviour in these dense sands, a test site was established at the harbour of Amaliahaven (Figure 1). Eleven piles were installed: three driven precast piles, four driven cast-in-situ piles and four screw displacement piles. These tests meet the requirements for high quality test databases such as the ZJU-ICL (Yang et al., 2015) or the ISO/API databases (Lehane et al., 2017).

This paper focusses on the load tests of the driven precast piles. More information and discussion on all test piles is to be disseminated into journal papers in due course.

3.1 *Local geology*

The river Maas is the focal point of Rotterdam. It is a part of the Rhine-Maas-Scheldt delta system, a delta which has shaped the Netherlands not just culturally and economically, but also geologically. The dynamic nature of the delta and the Dutch coastline has largely dictated the geological depositional processes, resulting in a range of marine, lagoonal and fluvial soils across the region (Hijma et al., 2012).

Around Rotterdam, newer Holocene soils are present to a depth of around 20m. In the eastern part of the port, these soils consist of thick layers of soft clay. Moving towards the western coast, on the other hand, interlayered clays and sands tend to dominate. These deposits include formations commonly found in the North Sea, such as the Naaldwijk, Nieuwkoop and Southern Bight Formations (Rijsdijk et al., 2005).

Under the Holocene soils is a Pleistocene epoch sand known as the Kreftenheye Formation. The formation was deposited by the Rhine-Maas river system and can be described as a medium dense to very dense poorly sorted calcareous coarse silica sand, frequently gravelly. The upper boundary of the formation is often capped by a bed of stiff clay known as the Wijchen Member (Autin, 2008).

The Kreftenheye Formation is found not just in Rotterdam but also across much of the western Netherlands, such as under the Hague, Utrecht and parts of Amsterdam. In these cities, the formation is the primary load-bearing layer for many pile foundations and so it is of substantial engineering importance. The outwash of the Rhine-Maas river system has also meant that the formation is present throughout the Dutch North Sea sector (Rijsdijk et al., 2005; Hijma et al., 2012), generally found 5 to 10m below seabed level with thicknesses of around 10m.

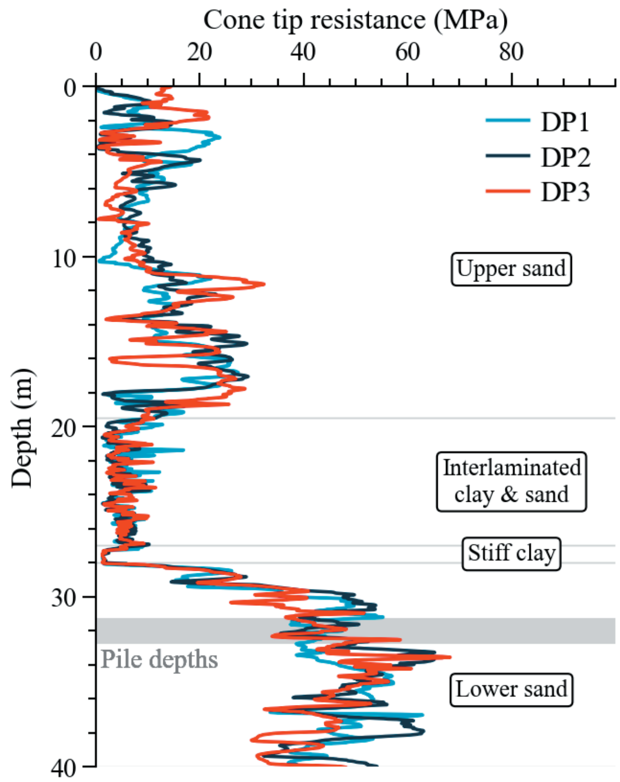

Figure 2: CPTs performed at each driven precast pile prior to installation

3.2 *Site investigation*

Before installation, at least three CPTs were performed two metres away from each test pile location, including one CPT on the location itself (Figure 2). Three CPTs were performed 1.5 metres away from pile DP3 after installation, although no significant variation was observed compared to the pre-installation CPTs. Boreholes were also performed across the entire harbour and a large amount of laboratory tests were carried out on the retrieved samples.

Dredged sands are present down to 14m depth. Underlying these sands is a naturally deposited sand layer, followed by interlaminated clays and sands belonging to the Naaldwijk formation. The Kreftenheye Formation begins at a depth of 28m. The formation is first capped by 1m of stiff clay with q_c equal to around 1.5MPa. The resistance of the sand layer then builds up to an average of around 50MPa that is relatively constant with depth. Some areas of the site feature thin weak laminations within this sand layer, although none of these were evident in the vicinity of the driven precast piles.

3.3 *Geometry of the test piles*

The driven precast piles (DP1, DP2 and DP3) measured 400mm square (equivalent diameter D_{eq} = 450mm) and with a length of 32m, giving a slenderness ratio L/D_{eq} of 70. To minimise the number of hammer blows on the pile, water jetting was performed in the upper layers during pile driving. This jetting stopped at least two metres above the lower sand layer to avoid any adverse effects on the primary load-bearing layer. The three piles were installed at least $7D_{eq}$ into this lower sand layer (Table 1).

In terms of the soil surrounding the pile base, all three piles have design cone resistances $q_{c,avg}$ of between 33 and 38MPa when using the 4D/8D Dutch averaging method (van Mierlo and Koppejan, 1953). The range of this $q_{c,avg}$ is much narrower when using an update to the Boulanger and DeJong (2018) filter method (de Boorder et al., 2022) with $q_{c,avg}$ around 44–46MPa.

Table 2: Test pile properties

	DP1	DP2	DP3
Side length [m]	0.4	0.4	0.4
Embedded length [m]	31.74	31.29	31.80
Age at pile test [days]	28	30	78
$q_{c,avg}$ (Dutch method)	38.1	33.5	34.0
$q_{c,avg}$ (Filter method)	45.5	44.0	45.3

3.4 *Instrumentation and load test procedure*

During each load test, deformation of the pile was measured to help distinguish between the pile base and shaft resistances. This was done using two types of fibre optic sensing techniques: Brillouin Optical Frequency Domain Analysis (BOFDA) and Fibre Bragg Grating (FBG). The BOFDA system provided distributed strain readings, whereas the FBG system provided discrete strain readings. Raman sensing was also used to apply temperature compensation between the reference measurement and the residual load measurement, as described in Duffy et al. (2022).

The piles were tested under axial compression using a test frame tied in by grout reaction anchors. Incremental loading was applied, with the duration of each increment determined based on the creep rate of the pile. Failure was defined as when the pile base displacement reached at least 10% of the pile's equivalent diameter.

4 **Results**

All three piles reached peak loads of around 8MN during testing. Piles DP2 and DP3 were loaded directly to this peak load, whereas pile DP1 was tested with load cycles after each of the first eight load steps. Nevertheless, Figure 3 shows that the three piles responded identically in terms of their initial stiffness. All piles experienced plunging failure within a pile base displacement of 10% D_{eq}.

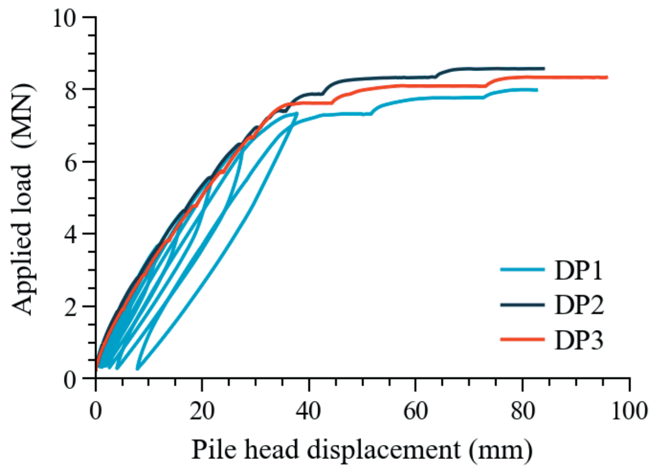

Figure 3: Response of the pile head during load testing

4.1 *Mobilised base resistances*

As with the overall load-displacement response, the piles behaved very similarly at the pile base (Figure 4). Locked-in residual stresses were already present at the start of load testing, measuring 10MPa at the pile base. This is already a substantial contribution to the total base capacity, that is, 20–25% of the design cone resistance $q_{c,avg}$ when determined by the filter method (Table 2). From this initial stress, a further 20MPa of base resistance was mobilised during testing, reaching a total base capacity of 30MPa, 66% of $q_{c,avg}$.

Figure 5 compares the measured resistances to those in the ISO/API database, without residual stresses included. The Amaliahaven pile tests reach base stresses 10MPa higher than the existing maximum base stress in the database. A clear linear relationship is seen across all cone resistances measured, well beyond any of the limitations provided in Table 1.

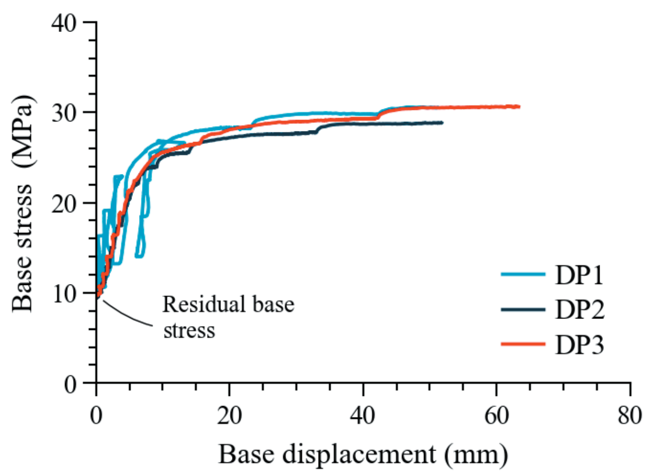

Figure 4: Mobilised base resistance, including residual stresses

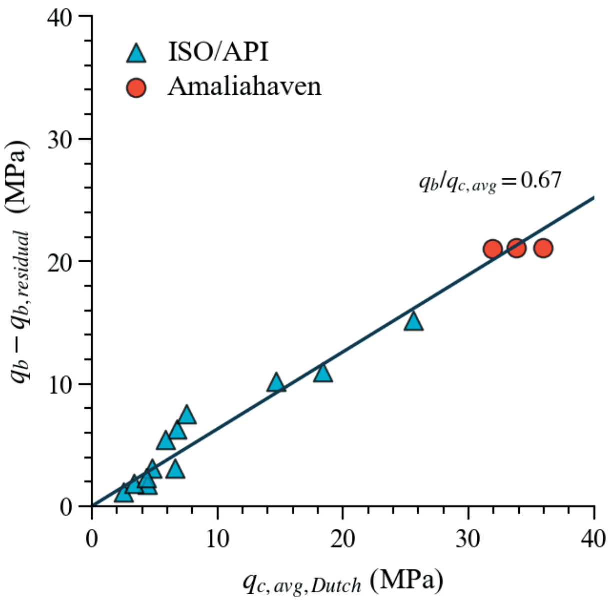

Figure 5: Comparison of the Amaliahaven results with the ISO/API database. The Dutch 4D/8D averaging method was used to determine $q_{c,avg}$

4.2 *Mobilised shaft resistances*

Taking the average shaft resistance $q_{s,avg}$ for each layer, some variation can be seen between the soil layers during each load test (Figure 6). Negative shear stresses were mobilised before load testing, acting in equilibrium with the 10MPa of residual base stress. These negative shear stresses are most significant in the lower sand layer, ranging from −40 to −60kPa.

The load tests mobilised very low resistances of around 20kPa in the upper sand layer, despite average q_c values of around 15MPa. In the interlaminated layer, the mobilised resistance was at least twice as high, measuring peak resistances of 50 to 120kPa. These results come with the caveat that the shaft resistance in the two layers was likely affected by both water fluidisation and friction fatigue. However, discerning these phenomena within the constraints of field testing means that it is difficult to quantify the individual impact of each on the piles.

Across the lower sand layer, much higher shaft resistances were reached. Mobilisation of the peak resistance rapidly occurs, within a displacement of around 1% of the equivalent diameter. A relatively brittle failure is then seen at loads between 200–250kPa. Interestingly, strong dilatancy is exhibited in pile DP3, tested two months after DP1 and DP2. This dilatancy reduced the shaft resistance from a peak of 260kPa to just under 220kPa.

Considering the shaft responses in the context of the design, the founding sand layer mobilised resistances twice as high as some of the prescribed limiting resistances (Table 1). This comes despite the negative shear stresses that had to be overcome at the start of the load test. In a similar fashion to the measured base resistances, the results suggest that imposing limiting resistances would lead to unnecessary overconservatism when designing for the pile shaft resistance.

5 Conclusions

This paper presents the outcomes of high-quality axial load tests on driven precast piles at a nearshore site in the Netherlands. The piles were founded in dense to very dense sand and extensively instrumented with fibre optics, providing a detailed insight into the mobilised base and shaft resistances.

In the load-bearing sand layer, high base and shaft resistances were mobilised, surpassing the limitations present in several design codes such as the API RP 2A design method. In the context of quay walls at the port of Rotterdam or offshore structures in the North Sea, for example, the limiting resistances would introduce a great deal of overconservatism into the foundation design. Indeed, while it is preferable to include a factor of safety, appropriately delineating where these factors of safety are to be applied is crucial for efficient design. By introducing overly conservative design components, piles that are larger than necessary are developed. This is turn, creates more risk during installation and increases the financial and environmental impacts associated with pile fabrication, transport and installation.

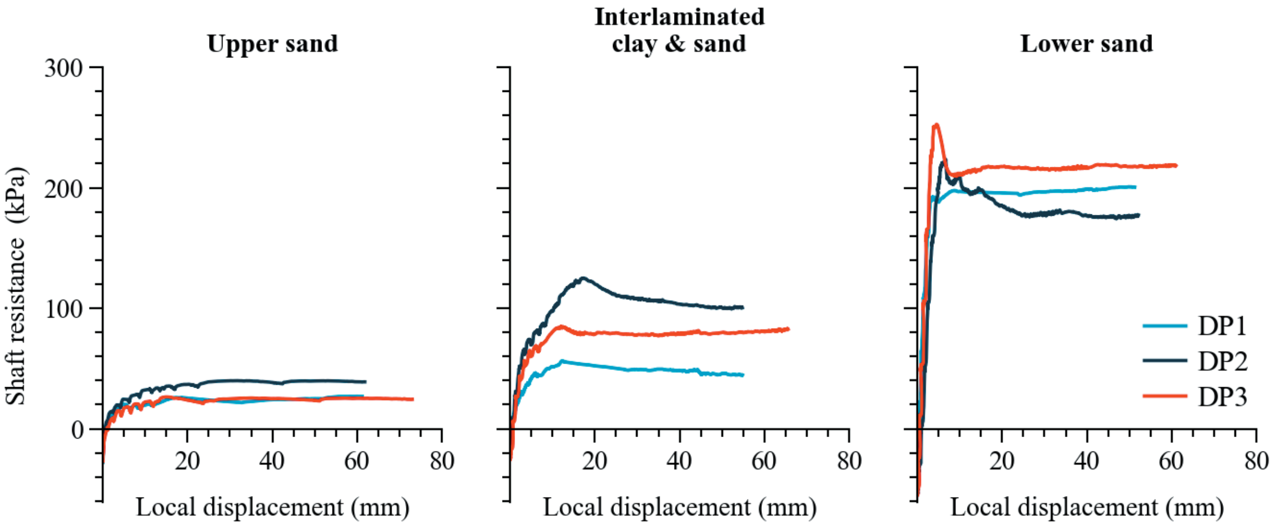

Figure 6: Shaft resistances mobilised in each soil layer. The stiff clay has been incorporated within the interlaminated clay and sand layer

5.1 *Acknowledgements*

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