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SERVICEABILITY LIMIT STATE DESIGN: BASE RESPONSE OF SCREW INJECTION PILES

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ABSTRACT

Screw injection piles are a type of screw displacement pile that use grout injection to displace soil around the pile tip and to reduce the installation resistance on the pile. The process results in lower noise and vibrations compared to a driven pile, whilst creating displacement mechanisms considered beneficial to the pile's capacity. Yet while these benefits have increased the uptake of screw injection piles in the industry, divergences in design standards means there is no consensus on the axial response of these piles, particularly with regards to their load-displacement response for serviceability limit state design. This paper takes two settlement prediction methods for sandy soils and compares their performance to instrumented load tests on screw injection piles from two different sites. In summary, the results suggest that screw injection piles develop little to no prestressing around the pile base during installation into sand, and a stiffness-based formulation could accurately estimate the pile displacement at both test sites. The formulation presents an effective way of predicting pile displacement, allowing for more benefit to be gained out of proof load tests and providing an efficient means of validating the in-situ pile capacity after installation.

Keywords: screw displacement, drilled displacement, cone penetration testing, pile testing, serviceability limit state

INTRODUCTION

Design according to Eurocode 7 must comply with both ultimate limit state (ULS) and serviceability limit state (SLS) criteria. ULS pertains to the pile's maximum bearing capacity which, if exceeded, may lead to severe economic losses, environmental damage, and human casualties. On the other hand, SLS focusses on the pile's displacement before ULS is reached, ensuring deformations remain within acceptable limits throughout a structure's service life.

While ULS design of screw displacement piles is generally well-defined, SLS design is often not codified (Allani and Huybrechts 2020). Engineers therefore face considerable uncertainty in predicting the load-displacement response of a screw displacement pile, compounded by the lack of fully instrumented field tests. Moreover, recent research (Duffy et al. 2024b) suggests that screw displacement piles mobilise base capacities more comparable to a soil-replacing, bored pile as opposed to a full-displacement, driven pile. Given the fundamental differences in how bored piles and driven piles affect soil stresses around the pile base, the findings can have important implications for SLS design.

To consider this further, this paper looks at a subset of screw displacement piles known as screw injection piles. Screw injection piles combine a push-in force and torque with fluid injection (typically grout) from the pile tip, reducing the installation resistance and increasing the pile stiffness upon curing. The pile's adaptability to different soil conditions and the low noise and vibrations it generates during installation, means they are becoming increasingly common in the deep deltaic soil deposits of the urbanised lowlands of the Netherlands and Belgium (Bottiau and Huybrechts 2019).

As part of a Dutch research programme on the axial capacity of piles in sand, two full-scale field tests were performed at Amaliahaven in the Port of Rotterdam and in Delft. Using the base resistance measurements, this paper evaluates the performance of the Dutch code and compares it to an alternative stiffness-based formulation to reduce the uncertainty designers face with SLS design.

SERVICEABILITY LIMIT STATE DESIGN

Pile settlement is often assessed using the load-transfer method, modelling the pile-soil interaction as a series of one-dimensional springs. These springs, known as *t-z* and *q-z* curves, describe the shaft and base resistances as a function of the local displacement. Their shape depends on the installation method and soil conditions, making them virtually unique to every pile.

A variety of approaches are available to estimate the *t-z* and *q-z* springs (Allani and Huybrechts, 2019; Bohn et al. 2019; Bateman, 2022). One of the few codified methods for screw displacement piles appears in the Dutch National Annex to Eurocode 7 (NEN 9997-1): the method prescribes "type-curves" (Fig. 1), where the curve is selected based on the pile type and soil conditions. For screw injection piles, a stiff response akin to a full-displacement pile is assumed and so Curve 1 is used. For soil-replacement piles, like bored or auger piles, Curve 3 is usually used and so a softer base response is modelled.

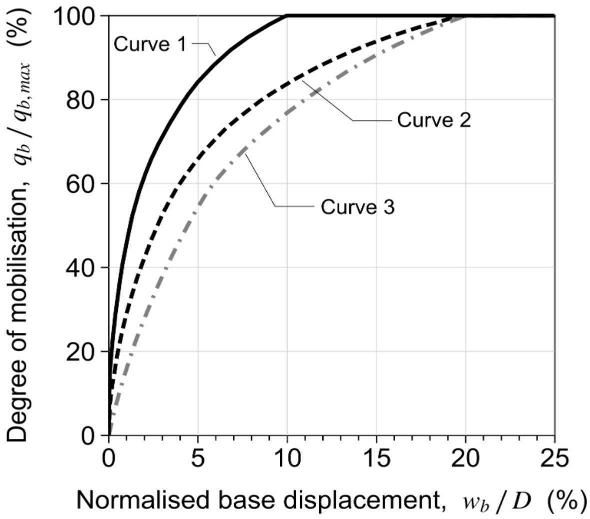

Fig. 1. Type-curves prescribed by the Dutch design method NEN 9997-1

Nevertheless, the rate at which a pile mobilises its full capacity is fundamentally dependent on the in-situ soil stiffness (Fleming, 1992; Atkinson, 2000). Recognising this, Fleming (1992) proposed a stiffness-based equation to evaluate the pile base displacement w_b:

$$\frac{w_b}{D} = \frac{q_b(1-v^2)\left(\frac{\pi}{4}\right)}{E_{b,eq}} \qquad [1]$$

where D pile diameter, $E_{b,eq}$ is the equivalent stiffness beneath the pile base and v is the Poisson's ratio which is approximately 0.2 for drained loading in sand (Mitchell and Soga 2005).

This same formulation was later adapted by Gavin and Lehane (2007) to assess both driven and bored piles, and Flynn (2014) later extended this to driven cast-in-situ piles (colloquially known as "vibro piles"). The adaptation assumes a linear stress-strain response upon initial loading of the pile. After reaching a certain yield displacement, w_{by}, the base response is expressed as a parabola of the form:

$$q_b = k\left(\frac{w_{by}}{D}\right)^{1-n}\left(\frac{w_b}{D}\right)^{n} \qquad [2]$$

where n is related to the base capacity at $w_b = 0.1D$, and k represents the initial stiffness response of the piles. A detailed description of the approach is given in Gavin and Lehane (2007).

PILE TEST SITES

At Amaliahaven (Duffy et al. 2024a), three screw injection piles with a permanent casing (*Tubex* piles, maximum diameter of 850 mm) were installed, as well as three 400 mm square, driven closed-ended piles. At Delft (Duffy et al. 2024b), two screw injection piles with a removable casing (*Fundex* piles) were installed along with three Tubex piles (Fig. 2), with both sets of piles having a maximum diameter of 470 mm.

The base mobilisation of all piles was relatively comparable at both sites and so for brevity, this paper presents just three screw injection piles:

1) Pile SI1: Amaliahaven, with permanent casing (D = 850 mm; L = 37 m).
2) Pile F1: Delft, with removable casing (D = 470 mm; L = 20 m).
3) Pile T2: Delft, with permanent casing (D = 470 mm; L = 20 m).

To compare the mobilisation rate of a screw displacement pile to a full-displacement pile, one of the driven closed-ended piles at Amaliahaven (DP2; L = 32 m) has also been included.

Fig. 2. The conical tip of a screw injection pile with a permanent casing (*Tubex* pile)

Ground conditions

The test piles at both sites were founded at least five pile diameters into a sand formation known as the Kreftenheye Formation. The Kreftenheye Formation is a fluvially deposited coarse sand found across all of the western Netherlands, colloquially known as “the Pleistocene sand”. At Amaliahaven, the formation was characterised by its very high relative density, with CPT cone resistances q_c measuring 40 MPa on average (Fig. 3) and occasionally reaching peaks of 80 MPa. At Delft, the cone resistances were much lower, averaging around 12 MPa.

Ideally, the initial in-situ stiffness modulus E_0 of sand is derived from shear wave velocity measurements, such as geophysical surveys or seismic CPTs. In the absence of such data, E_0 can also be approximated by correlations to CPT q_c data, such as that defined by Rix and Stokoe (1991) :

$$K_G = \frac{G_0/q_c}{Q_{tn}^{-0.75}} \qquad [3]$$

where Q_{tn} is the stress-normalised cone tip resistance and G_0 is the initial shear modulus, related to E_0 by the equation $E_0 = 2G_0(1+\nu)$.

At Amaliahaven, a geophysical survey (multichannel analysis of surface waves, MASW) was performed alongside the test piles, penetrating three metres into the Kreftenheye Formation. The survey yielded mean G_0 values of 125 GPa which, when compared to the CPT data (Fig. 4), showed that a K_G value of 215 gave a good fit to the measurements and consistent with values for young, uncemented sand (Schneider and Moss 2011). No shear stiffness measurements were made beyond three metres into the Kreftenheye Formation at Amaliahaven nor at the Delft test site. However, since the sand formation at both sites is geologically identical, the same K_G was assumed for both sites. Within two pile diameters from the pile base, G_0 ranged from 125−200 MPa at Amaliahaven and from 70−100 MPa at Delft.

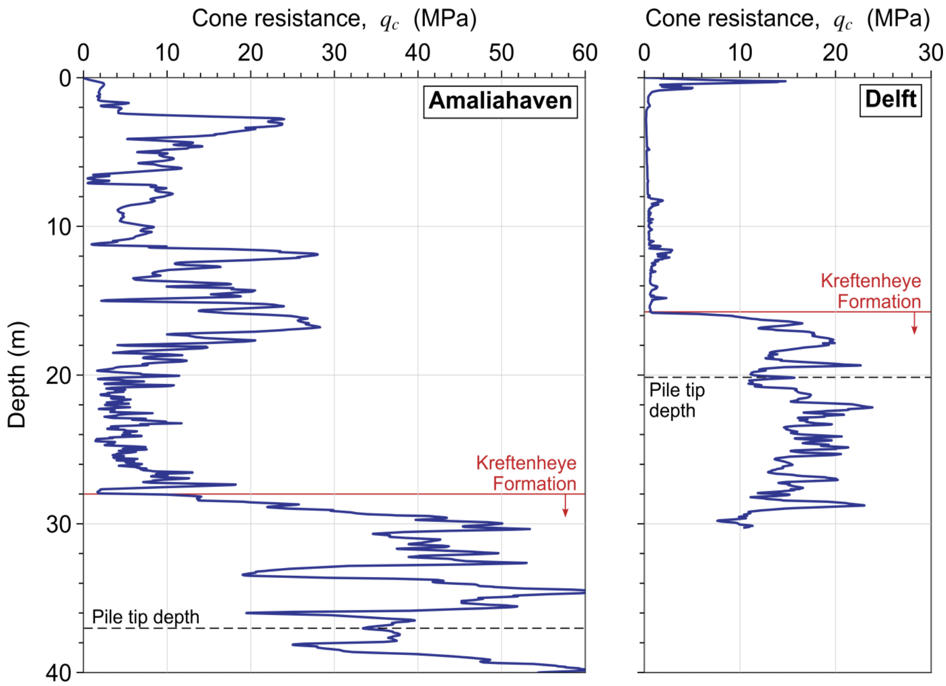

Fig. 3. CPT cone resistances at Amaliahaven and Delft

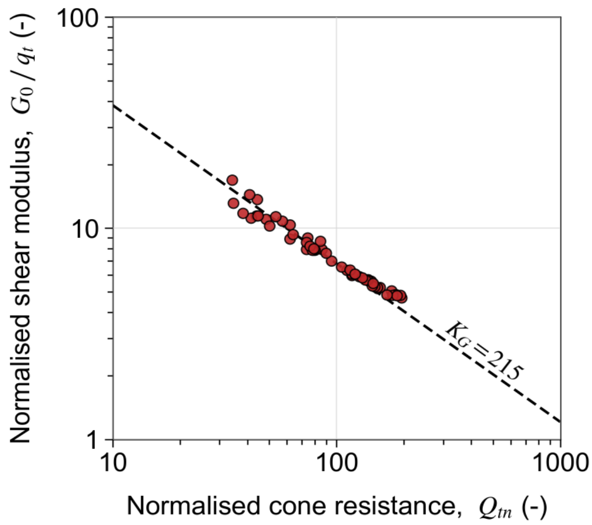

Fig. 4. Results of the MASW survey at Amaliahaven compared to CPT measurements

Pile installation

Compared to the Delft site, the penetration speed was much slower in the very dense sand of Amaliahaven. This is shown in Fig. 5 by the advancement ratio, colloquially known as the "scrape factor". The advancement ratio normalises the vertical displacement in one full pile rotation by the helical pitch of the screw tip. The pullup force was not measured at Delft and so the pulldown force in Fig. 5 can be seen as the maximum downward force on the pile. The pulldown force varied across all three piles, because of the different installation procedures for each pile as well as differences in relative densities at the two sites.

Grout with a water-cement ratio of 1.5 to 2.0 was injected at 200 L/min from the pile tips for most of the installation. Towards the end of installation, the piles were installed "in-the-dry"—a typical practice where the grout injection is turned off towards the end of penetration to avoid destructuration of soil around the pile base. At Delft, the piles penetrated an additional 25 cm without grout injection whereas at Amaliahaven, the very dense sands meant only one additional rotation of the pile was possible without grout injection.

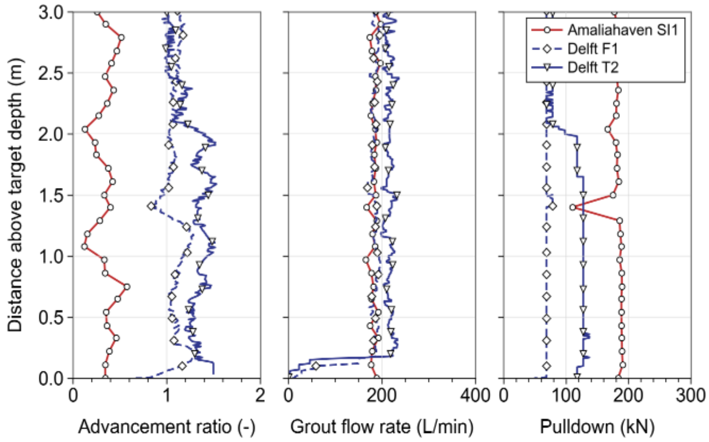

Fig. 5. Comparison of installation performance towards the end of installation

STATIC LOAD TEST RESULTS

All piles were instrumented with distributed fibre optic sensors along their entire length, affixed to either the permanent casing or to the reinforcing cage. Each pile was loaded incrementally under axial compression until a base displacement of around $0.1D$, giving clear insights into the shaft and base capacity of each pile.

Measured base mobilisation curves

Fig. 6 compares the mobilised base resistances at both test sites. To account for the variation in soil strength around the pile base, the base resistances have been normalised by $q_{c,filter}$ using the spatial filtering algorithm by Boulanger and DeJong (2018). At Amaliahaven, SI1 mobilised its full resistance much more gradually compared to DP2, with the base stress continuing to rise as it approached a normalised displacement of 10%. In contrast, DP2 mobilised most of its resistance within a displacement of just 3%. In the medium dense sand of Delft, piles F1 and T2 behaved much stiffer than SI1: both piles mobilised most of their resistance within a displacement of 5%, showing a clear plunging failure beyond this point whereby further displacement mobilised little additional base stress.

At a base displacement of $0.1D$, all three screw injection piles mobilised normalised base stresses $q_b/q_{c,filter}$ of around 0.28. By comparison, the driven closed-ended pile DP2 mobilised twice as much base capacity, including the residual stresses developed during the process of pile driving.

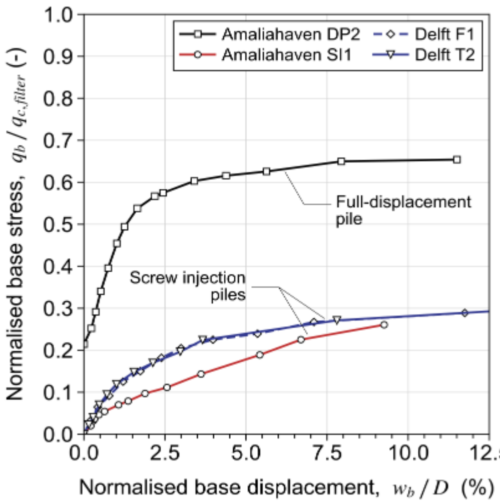

Fig. 6. Mobilised base resistances at Amaliahaven and Delft

Equivalent linear base stiffness

Using Equation 1, the base response can be interpreted as a stiffness degradation response (Fig. 7). Since field tests focus on large-scale movements, very small displacements (< 0.5mm) could not be accurately resolved with the fibre optic instrumentation. Notwithstanding, the initial stiffnesses of the screw injection piles generally aligned with the in-situ G_0 values. By contrast, the driven closed-ended pile DP2 showed much higher initial stiffnesses, likely because of the large amount of prestressing created by the installation process.

This same prestressing has also been shown (Gavin and Lehane, 2007) to affect when stiffness degradation occurs (i.e. w_{by} from Equation 2). For the screw injection piles, stiffness degradation begins at normalised displacements of 0.3−0.6%. By comparison, stiffness degradation occurs later in DP2, at a displacement of around 0.8%. From this point, the stiffness degrades much more rapidly in DP2 compared to the screw injection piles.

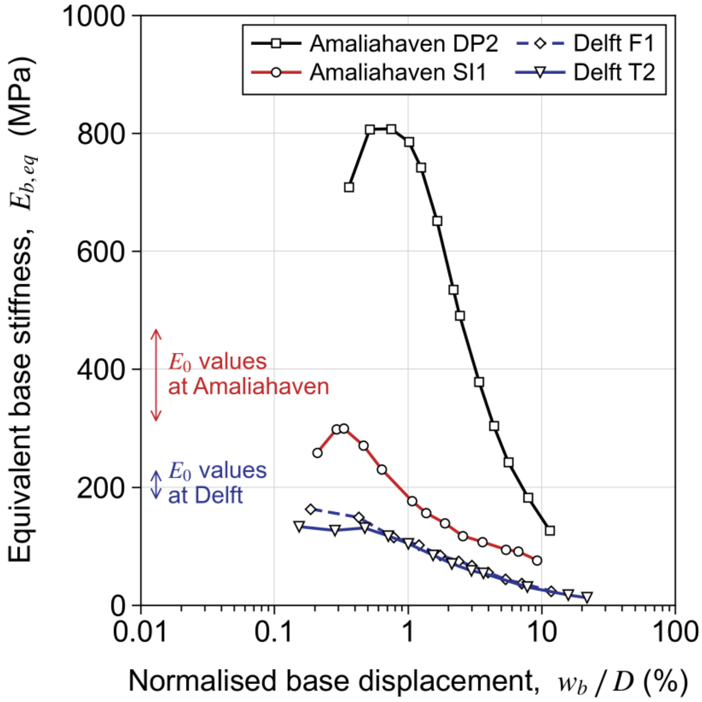

Fig. 7. Stiffness response of the test piles under static loading

Prediction of the base mobilisation curves

Fig. 8 compares the predictions of Equation 2 to the NEN 9997-1 type-curves. Both methods were calibrated using a normalised base stress $q_b/q_{c,filter}$ of 0.28, as indicated by the measurements in Fig. 6. The methods therefore converge to the same base stress at a normalised displacement of 10%, that is, the pile base capacity for ULS design.

The type-curve predictions align well with the measurements at Delft—a site whose geotechnical conditions (soft clay overlying medium dense sand) are relatively representative of most of the western part of the Netherlands. The stiffness-based method performs similarly to the type-curves, accurately predicting the base mobilisation for pile F1 and slightly underestimating the displacement for pile T2.

However for Amaliahaven SI1, the type-curve overestimates its initial stiffness by nearly a factor of two at a normalised displacement of 3%. In contrast, this response is captured well by the stiffness-based approach. This agreement comes in spite of the observed installation differences shown in Fig. 5, suggesting that the in-situ soil stiffness governs the base response of screw injection piles over other installation effects, within the range of installation parameters considered.

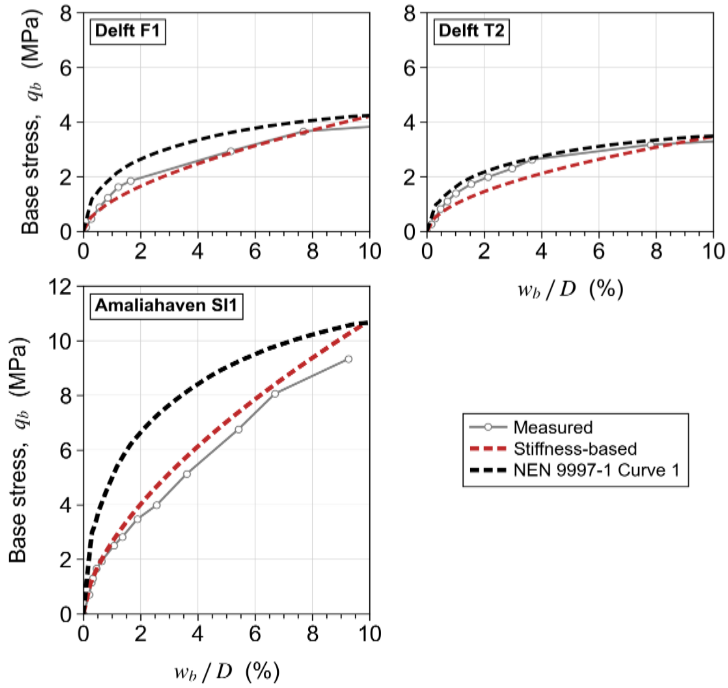

Fig. 8. Prediction of the base mobilisation curves

CONCLUSION

Measurements from screw injection piles at two test sites have been used to investigate how screw injection piles mobilise their base resistance in sandy soils. Considering the measurements in terms of an equivalent base stiffness, it was shown that stiffness degradation in screw injection piles begins at much lower displacements compared to an equivalent driven closed-ended pile. Comparing this equivalent stiffness to the in-situ soil stiffnesses before pile installation suggests that screw injection piles create little to no prestressing around the pile base, leading to a softer response under loading compared to a driven pile.

This behaviour was well captured by a load-transfer method that directly incorporates the in-situ soil stiffness into the formulation. By defining the input parameters directly from site investigation data, the approach is more generalisable to sand layers of different relative densities, improving on existing formulations prescribed in design standards for screw displacement piles. Ongoing research is focussing on understanding the site-specificity of CPT-based small-strain stiffness correlations, although it's recommended that geophysical measurements are used directly in order to give more confidence in stiffness-based SLS design.

ACKNOWLEDGEMENTS

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