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CPT-based axial design of screw displacement piles in granular soils

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Screw displacement piles are designed to maximise soil displacement, while minimising the noise and vibration generated from installation. While these benefits have increased the uptake of screw displacement piles in the industry, divergences in design standards means there is no definitive consensus on the axial response of these piles. To address this, a high-quality database of load tests on screw displacement piles is presented in this paper – the largest of its kind in the published literature. Both instrumented and uninstrumented records are used to compare four different cone penetration test-based design methods for screw displacement piles in granular soils. The instrumented data show that existing design methods tend to overestimate the base capacity of screw displacement piles yet underestimate their shaft capacity. A linear best fit to the instrumented data shows improved predictions of the total pile capacity when compared to both instrumented and uninstrumented records. However, variability in predictive performance suggests that installation-related mechanisms may influence the overall capacity of these piles, particularly piles with an oversized displacement body.

Keywords: cone penetration testing/drilled displacement/piles & piling/screw displacement/testing & evaluation

Notation

D_{meas}	measured outermost diameter
D_{nom}	nominal outermost diameter
d	casing diameter
L	length of the pile
p_{a}	atmospheric pressure
Q_{calc}	calculated failure load on the pile head by a design method
Q_{meas}	measured load on the pile head at failure
Q_0	load applied on top of the pile
$Q_{0,0.1D}$	load applied on the pile head at a base displacement of $0.1D_{\text{nom}}$
$Q_{0,1\text{inch}}$	load applied on the pile head at a head displacement of one inch (= 25.4 mm)
$Q_{0,\text{max}}$	maximum load applied on the pile head
q_{b}	base capacity
$q_{\text{b,lim}}$	limiting base capacity
q_{c}	cone penetration test (CPT) cone tip resistance
$q_{\text{c,avg}}$	design cone tip resistance for base capacity design
$q_{\text{c,tip}}$	cone tip resistance at pile base
q_{s}	shaft capacity
$q_{\text{s,lim}}$	limiting shaft capacity
s_{b}	displacement of the pile head
s_0	displacement of the pile head
w_{s}	correction factor in the Federal Highway Administration (FHWA) method for the pile shaft resistance
w_{t}	correction factor in the FHWA method for the pile base resistance
α_{p}	pile class factor for determining the pile base resistance
α_{s}	pile class factor for determining the pile shaft resistance

1. Introduction

Screw displacement piles, also known as drilled displacement or auger displacement piles, combine soil removal and displacement mechanisms to improve installation efficiency. Screw displacement piles therefore have some of the benefits of bored and auger piles – like the lower installation resistance and reduced noise – while the soil displacement mechanisms can create higher radial stresses and shaft capacities compared to an equivalent bored pile (Basu *et al*., 2014; Knappett *et al*., 2016; Pucker and Grabe, 2012; Slatter, 2000). Screw displacement piles are therefore an attractive alternative to bored or driven piles, constituting roughly 9% of the worldwide piling market (Bottiau, 2015) and are also used as rigid inclusions for ground improvement (Suleiman *et al*., 2016; Varaskin *et al*., 2016).

Axial design methods for screw displacement piles often relate the base capacity q_{b} and shaft capacity q_{s} to cone penetration test (CPT) parameters like the cone tip resistance q_{c}. These correlations are the basis for many national design standards, like in France (French Standardisation Association; AFNOR, 2018), Belgium (Belgian Bureau of Normalisation; NBN, 2022), the Netherlands (Dutch Standardisation Institute; NEN, 2017) and the USA (Brown *et al*., 2007). Yet research into CPT-based design continues to evolve. Recent studies (Bittar *et al*., 2022; Chai *et al*., 2025; de Lange *et al*., 2018; Duffy *et al*., 2024b; Tian and Lehane, 2025) have shown that the filtering algorithm by Boulanger and DeJong (2018) gives improved predictions of the pile capacity, particularly when weak zones are nearby. Moreover, many design methods limit the base and shaft resistances, an approach that has been shown to introduce substantial over-conservatism in dense to very dense sands (Duffy *et al*., 2024a).

This research has been accounted for in new design methods for driven precast (Lehane *et al*., 2020) and bored piles (Doan and Lehane, 2021; Niazi and Mayne, 2016), but design methods for screw displacement piles remain underexplored. While recent analyses of screw displacement pile design methods have been performed (Figueroa *et al*., 2022; Moshfeghi and Eslami, 2019; Rad *et al*., 2023), the studies highlight the lack of full-scale instrumented tests which reach the failure criteria set out by design standards.

Therefore, this paper presents a new compilation of screw displacement pile load tests in granular, coarse-grained soil – all satisfying the requirements of a high-quality pile test database. Using the database, four CPT-based design methods were analysed: first in terms of the measured base and shaft resistances, and then in terms of the total capacity. As a comparison, a best-fit method to the instrumented data was also derived, and in the process of doing so, showing the current state-of-the-art in screw displacement pile design.

2. Categorising screw displacement piles

For most design methods, a 'screw displacement pile' is a relatively broad description, encompassing a range of installation techniques that balance both soil removal and soil displacement mechanisms. In the Netherlands alone, for instance, there are over 20 proprietary names for screw displacement piles, often distinguished by just minor variations in installation procedures or drilling tool geometries. For simplicity, this paper has categorised screw displacement piles into four types (Figure 1). To distinguish these piles from screw piles or screw anchors (e.g. Bittar *et al*., 2024; Cerfontaine *et al*., 2023; Mohajerani *et al*., 2016), these four types of screw displacement piles have a larger stem or casing diameter d_{nom} – with the aim of maximising the displacement mechanisms that may govern the axial pile capacity – and the resulting pile is a cast in situ concrete pile that is usually designed for compressive loads.

2.1 Type 1: smooth shaft

Type 1 piles use a sacrificial screw tip with a diameter larger than that of the steel casing. Once the screw tip reaches the target depth, concrete is poured into the casing and the casing is extracted, leaving the screw tip behind. The finished pile therefore has a relatively smooth shaft with a screw tip at the base. Alternatively, some Type 1 piles use a thin-walled casing welded to the screw tip, both of which remain in situ after installation.

2.2 Type 2: helical shaft

For Type 2 piles, a helical screw is integrated into a hollow drilling tool. A sacrificial base plate is fitted to the base of the drilling tool, sealing the tool during installation and providing a base resistance. Once the target depth is reached, the drilling tool is filled with concrete and extracted at a rate matching the pitch of the helical screw – creating a helical concrete shaft. The Atlas pile is one of the pioneering pile types in this regard (Imbo, 1984), installed using a purpose-built rig. Olivier piles are another common Type 2 pile, particularly for fixed leader piling rigs.

2.3 Type 3: with displacement body

Type 3 piles use a drilling tool with screw flights above and below an oversized displacement body. A sacrificial base plate is fitted to the bottom of the drilling tool, which is filled with concrete and withdrawn once the target depth is reached, creating a relatively smooth-shafted pile. Initial development of Type 3 piles began with the De Waal pile in the late 1980s, later evolving into more geometrically complex drilling tools like the Omega pile (Bottiau and Cortvrindt, 1994).

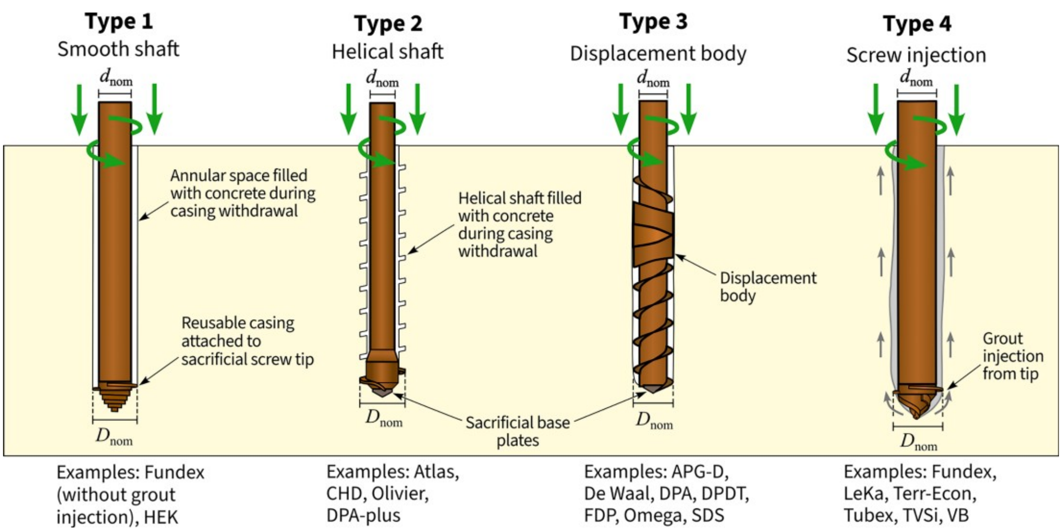

Figure 1. Four different types of screw displacement piles, indicating the nominal, outermost diameter D_{nom} and the diameter of the casing or stem d_{nom}

2.4 Type 4: screw injection

Screw injection piles use an injection fluid, such as grout, in tandem with a push-in force and torque, and are often just a minor modification to Type 1 piles. The injected fluid flows up through the annular space around the pile shaft and exits at the ground surface, lubricating the pile shaft and reducing the resistance on the pile during installation. Screw injection piles are becoming increasingly prevalent in the Netherlands and Belgium, particularly for dense sands and alternating loose and dense sands (Bottiau and Huybrechts, 2019).

3. Design methods for screw displacement piles

CPT-based methods for screw displacement piles often use empirical coefficients (e.g. α_p and α_s) to relate the pile shaft resistance q_s and base resistance q_b to the CPT cone tip resistance q_c:

1. $$q_b = \alpha_p q_{c,avg} \leq q_{b,lim}$$

2. $$q_s = \alpha_s q_c \leq q_{s,lim}$$

where $q_{b,lim}$ and $q_{s,lim}$ are limiting resistances to the base and shaft capacities and $q_{c,avg}$ is a weighted average of CPT tip resistances in a diameter-dependent zone around the pile base, determined using 'q_c averaging methods'.

Both Equations 1 and 2 determine the capacity at failure. However, precisely defining the moment of failure is a point of contention (NeSmith and Siegel, 2012; Stuedlein *et al*., 2014), particularly since the mobilisation rate of the base resistance depends on the soil stiffness and pile diameter (Gavin and Lehane, 2007; White and Bolton, 2005). For design purposes, the pile capacity is often just defined at a displacement where most (if not all) of the capacity is expected to be fully mobilised.

The performance of four design methods has been analysed in this paper, chosen based on the popularity of screw displacement piles in their respective countries.

- *AFNOR (France)*. Also referred to as the LPC or LCPC method, this approach was developed using a database of 174 load tests on different pile types, drawing on research by Bustamente and Gianeselli (1998). In this paper, the description in Frank (2017) and Verheyde and Baguelin (2019) has been taken.
- *NBN (Belgium)*. This was developed from mainly unpublished tests, as well as extensive test campaigns at Sint-Katelijne-Waver (Huybrechts, 2001) and Limelette (Maertens and Huybrechts, 2003a). The 2022 update of the NBN method reduced α_p by around 40% for screw displacement piles (WTCB, 2020). An outline of the design method prior to the 2022 update can be found in Huybrechts *et al*. (2016).
- *NEN (The Netherlands)*. This method was developed on mainly unpublished tests (Stoevelaar *et al*., 2014). A 2017 update to the design code led to a 30% reduction in α_p across all pile types. More details on the design method can be found in Gavin *et al*. (2021).
- *FHWA (USA)*. This is the recommended design method by the Federal Highway Administration (FHWA). It was developed based on a database of 40 tests on pressure-grouted Type 3 piles, as presented in Brown *et al*. (2007) and NeSmith (2002).

The three European design methods, AFNOR, NBN and NEN, are all country-specific complements to Eurocode 7, first published in 2007.

3.1 Coarse-grained soils

Table 1 presents a simplified overview of each design method, highlighting some key differences.

- Each method uses its own q_c averaging method from which its respective α_p factors are calibrated – meaning the zone of influence around the pile base is also different for each method.
- The FHWA method defines failure as the load $Q_{0,1inch}$ at a pile head displacement s_0 of 25.4 mm (1 inch), while the three other design methods define the failure load $Q_{0,0.1D}$ at a base displacement s_b of $0.1D$.
- The NBN and NEN methods include a shape factor to account for the influence of an oversized pile base on the pile response, potentially reducing q_b by up to 30%.
- The AFNOR and NBN methods use a non-linear q_s–q_c relationship, which varies depending on the soil type. This non-linearity implicitly limits the maximum shaft resistance to 200 and 90 kPa, respectively.

Table 1. Summary of design methods in granular soils

Country	Method	Pile type	α_s	$q_{s,lim}$: kPa	α_p	$q_{b,lim}$: MPa	Averaging method	Failure criterion
Belgium	NBN	All	0.0060[a]	90	0.50	None	De Beer (1971)	$s_b = 0.1D$
France	AFNOR	Temporary casing, concrete shaft	0.0145[a]	200	0.50	None	LPC 2012 Verheyde and Baguelin (2019)	$s_b = 0.1D$
	—	Permanent casing (Type 1)	0.0040[a]	90	0.50	None	—	—
Netherlands	NEN	All	0.0090	135	0.63	15	Dutch 4D/8D van Mierlo and Koppejan (1952)	$s_b = 0.1D$
USA	FHWA	All	0.0100	160–210	0.40	7.2–8.62	Fleming and Thorburn (1983)	$s_0 = 25.4$ mm (1 inch)

[a]Includes an additional non-linear dependency between q_s and q_c depending on the soil type.

- The FHWA method adds a factor w_t to Equation 1 and w_s to Equation 2, which depend on soil gradation and angularity. For example, in uniform, rounded sands with a fines content of up to 40%, w_t is taken as 0 MPa and $q_{b,lim}$ is set at 7.2 MPa. In well-graded, angular sand with less than 10% fines, w_t is taken as 1.35 MPa with a $q_{b,lim}$ of 8.62 MPa. Interpolation is used between these two bounds.

3.2 Fine-grained soils

Only one design method was used for all fine-grained soils (clay, silt or mixed soil) so that the estimation error in fine-grained soils was the same across all design methods. Therefore, the CPT-based approach by Bustamente and Gianeselli (1998) was used for clay, a method which also forms the basis of the AFNOR design method and was recommended by Larisch (2014). The method is a graphical approach (Figure 2), where the curve is chosen based on the soil and pile type. For example, a screw displacement pile with a temporary casing uses curve Q1 for clay (q_c <1 MPa), Q3 for silty clay (q_c >1.5 MPa) and Q4 for sandy clay (q_c >3 MPa). A screw displacement pile with a permanent casing uses curve Q1 for clay and Q2 for both silty clay and sandy clay.

4. Screw displacement pile test database

Sixty load tests on screw displacement piles were compiled (Table 2). Most tests come from public literature and the archives of Delft University of Technology and Deltares, with data from three of the sites (Amaliahaven, Delft and Werkendam) that were collected first-hand by the authors. All tests meet the following requirements.

- At least one CPT was performed beside each pile, to a depth at least four pile diameters below the pile base.
- The tests were first-time static compression tests, reaching a pile head displacement s_0 of at least 25 mm (the failure criterion of the FHWA method).
- The pile's base was located in a sand layer and derived at least 50% of its capacity from granular soil.

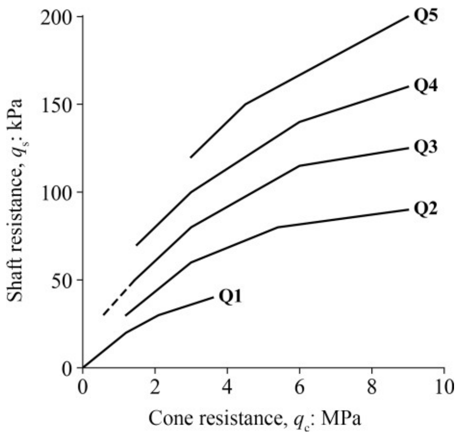

Figure 2. Design chart for fine-grained soils, adapted from Bustamente and Gianeselli (1998)

Type 3 piles are the most common in the database (Figure 3), representing 38% of all piles. Type 4 piles are the least common, although their increasing use in industry has led to more recent tests, often with state-of-the-art fibre optic instrumentation. Most piles have a nominal outermost diameter D_{nom} ranging from 360 mm to 560 mm (Figure 1). The only exceptions are the 850 mm dia. screw injection piles at Amaliahaven, where the cone resistance at the pile tip $q_{c,tip}$ was around 50 MPa. These piles also have the highest capacity of the database piles, ranging from 18 MN to 24 MN. The rest of the database piles have capacities of 2–3 MN, with $q_{c,tip}$ ranging from 10 to 25 MPa.

4.1 Pile shape and diameter

For design and load test evaluation of cast in situ piles, assumptions are needed regarding the realised diameter and cross-sectional stiffness. The test piles at Limelette were the only piles from the database that were extracted and none of the database records included measured cross-sectional stiffnesses. However, extraction records from piles that did not meet the database requirements (Figure 4) suggests that the measured diameter D_{meas} tends to be equal to the nominal, outermost diameter of the drilling tool D_{nom}, with variation of up to ± 20% from the average measured diameter. The extracted piles also showed a range of tip shapes – even within the same site (Figure 5) – which is likely to be as a result of differences in soil conditions and installation procedures. Based on these observations, the nominal, outermost pile diameter D_{nom} was used to derive the base and shaft capacity of each pile. For the Limelette piles, the average measured diameter was used.

5. Instrumented test results

In total, 25% of the tests were instrumented, allowing for the base and shaft capacities to be clearly distinguished from the total capacity. Most of these tests used long-gauge extensometers placed in a reservation tube after pile installation. The only exceptions were at Amaliahaven and Delft – where both discrete and distributed fibre optic sensors were used – and Terneuzen, where vibrating wire gauges were used. Where possible, the force distribution was derived from the response of the strain gauges at the pile head (Fellenius, 2001; Flynn and McCabe, 2021b) and compared to the theoretical composite stiffness. None of the Type 1 piles were instrumented and the FHWA method was not considered because of its substantially different failure criterion compared to the other methods.

5.1 Shaft capacity

At cone resistances less than 10 MPa (Figure 6), the AFNOR method predicted the highest shaft resistances, whereas NBN and NEN predicted shaft resistances almost half that of AFNOR. Across this same range, Type 3 piles tended to mobilise higher resistances than the Type 2 and Type 4 piles. Beyond cone resistances of 15 MPa, the limiting resistances in the AFNOR, NBN and NEN methods led to substantial underestimation of the shaft resistance. This is particularly evident for the Type 4 piles tested at Amaliahaven – which

Table 2. Database for assessing screw displacement pile capacity in granular soils

Site and pile	D_{nom}: mm	D_{nom}: mm	L: m	$q_{c,tip}$: MPa	$Q_{0,max}$: MN	$s_{0,max}/D$: %	Reference
Type 1: Smooth shaft							
Almere SBP4	360	430	13.1	12.4	1.8	8	Schokking and IJnsen (2014)
Almere SBP5	360	430	10.2	4.4	1.0	8	Schokking and IJnsen (2014)
Almere SBP6	360	430	10.2	2.9	1.0	6	Schokking and IJnsen (2014)
Limelette A1	380	450	9.6	20.5	3.1	17	Maertens and Huybrechts (2003a)
Limelette C1	380	450	9.6	21.7	1.6	7	Maertens and Huybrechts (2003a)
Seildeich P1	440	560	14.8	15.8	2.6	5	de Cock (2001)
Seildeich P2	440	560	13.8	18.3	3.0	5	de Cock (2001)
Terneuzen P02	460	560	21.4	12.7	6.0	10	van Baars *et al.* (2018)
Terneuzen P04	460	560	20.0	16.2	5.3	13	van Baars *et al.* (2018)
Terneuzen P06	460	560	19.9	14.8	4.4	14	van Baars *et al.* (2018)
Zimmerplatz P1	420	560	14.0	17.0	3.0	6	de Cock (2001)
Type 2: Helical shaft							
Berlin P2	460	560	14.2	12.4	3.5	5	Brieke (1993)
Gent P1	460	510	13	8.1	2.8	9	de Cock (2001)
Gent P2	460	510	13.5	9.7	3.3	15	de Cock (2001)
Limelette A2	360	510	9.2	21.0	3.1	5	Maertens and Huybrechts (2003a)
Limelette B3	360	510	9.4	22.8	3.6	13	Maertens and Huybrechts (2003a)
Limelette B4	360	510	9.4	23.1	3.4	11	Maertens and Huybrechts (2003a)
Limelette C2	360	510	9.1	22.0	2.7	5	Maertens and Huybrechts (2003a)
Oldenburg P1	510	560	9.5	21.2	2.3	5	de Cock (2001)
Oldenburg P2	510	560	7.0	7.5	1.8	6	de Cock (2001)
Oostwoud P27	340	410	14.2	14.0	1.3	14	Private database
Oostwoud P28	340	410	13.8	15.9	1.5	12	Private database
Zeeland O1	310	460	9.1	31.5	1.4	16	Private database
Zeeland O2	310	460	8.0	23.6	1.6	12	Private database
Zeeland O3	310	460	9.5	20	1.8	16	Private database
Werkendam P5	410	410	16.6	11.9	1.7	8	Private database
Werkendam P6	410	410	16.6	12.1	1.8	6	Private database
Werkendam P8	410	410	16.1	12.1	1.7	7	Private database
Type 3: Displacement body							
BEST C2	260	450	9.5	7.4	2.4	28	Fellenius *et al.* (2017)
Elblag P9	n/a	400	7.5	16.6	1.1	7	Krasiński and Wiszniewski (2021)
Gdansk P809	n/a	400	12.9	9.7	1.7	11	Krasiński *et al.* (2023)
Grottgera P600	n/a	400	11	12.6	1.2	11	Krasiński *et al.* (2023)
Grottgera P602	n/a	400	10.8	15	1.5	10	Krasiński *et al.* (2023)
Hamburg PA	318	510	10.4	11.9	3.2	8	Busch *et al.* (2013)
Hamburg PB	n/a	440	8.4	12.5	2.2	13	Busch *et al.* (2013)
Limelette A3	410	410	9.4	22	2.9	12	Maertens and Huybrechts (2003a)
Limelette A4	410	410	9.5	22.6	2.6	16	Maertens and Huybrechts, (2003a)
Limelette C3	410	410	9.4	20.7	2.9	14	Maertens and Huybrechts (2003a)
Limelette C4	410	410	9.5	19.4	2.5	20	Maertens and Huybrechts (2003a)
Loenhout S2	n/a	410	9.5	13.2	1.2	14	Theys *et al.* (2003)
Lomme P1	273	360	8.5	23.8	1.6	14	Bustamente and Gianeselli (1997)
Oostende P1	n/a	460	21.6	29.5	3.6	16	Van Impe *et al.* (2013)
Oostwoud P1	360	360	16	16.8	1.6	17	Private database
Oostwoud P22	360	360	18	12	1.8	15	Private database
Oostwoud P6	360	360	16.5	14	1.1	19	Private database
Vilvoorde P5	n/a	410	8.2	13.7	1.9	14	Bottiau (1995)
Werkendam P1	410	410	17.4	9.6	2.4	11	Private database
Werkendam P2	410	410	17.8	5.4	2.4	8	Private database
Werkendam P3	410	410	17.4	10.9	2.3	9	Private database
Werkendam P4	410	410	17.8	8.7	2.2	10	Private database
Zulawy P6293	n/a	400	12.6	14.1	0.9	11	Krasiński and Wiszniewski (2021)
Type 4: Screw injection							
Amaliahaven SI1	610	850	37	36.5	20.3	14	Duffy *et al.* (2024a)
Amaliahaven SI2	610	850	37.1	49.4	23.3	7	Duffy *et al.* (2024a)
Amaliahaven SI3	610	850	35	22.5	18.6	18	Duffy *et al.* (2024a)
Amaliahaven SI4	610	850	34.1	54.7	19	15	Duffy *et al.* (2024a)
Delft F1	315	470	19.8	12.7	2.7	42	Duffy *et al.* (2024b)

(continued on next page)

Table 2. Continued

Site and pile	D_{nom}: mm	D_{nom}: mm	L: m	$q_{c,tip}$: MPa	$Q_{0,max}$: MN	$s_{0,max}/D$: %	Reference
Delft F2	315	470	19	13.6	2.6	41	Duffy *et al*. (2024b)
Delft T1	382	470	19.9	11.3	2.6	46	Duffy *et al*. (2024b)
Terneuzen P03	460	560	20.2	13.5	6.1	13	van Baars *et al*. (2018)
Terneuzen P05	460	560	20.2	15.3	6.4	15	van Baars *et al*. (2018)

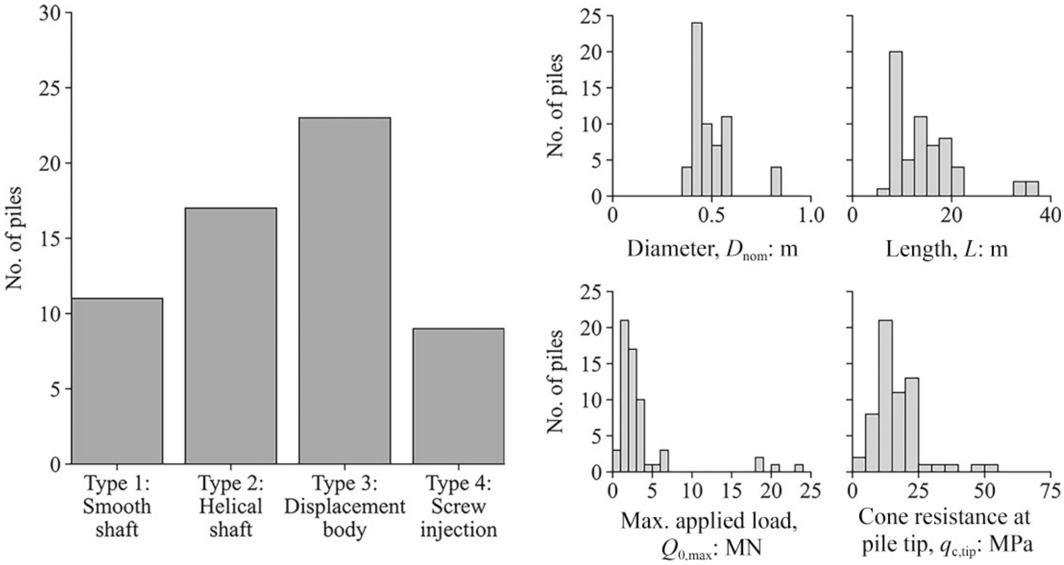

Figure 3. Visual summary of the database

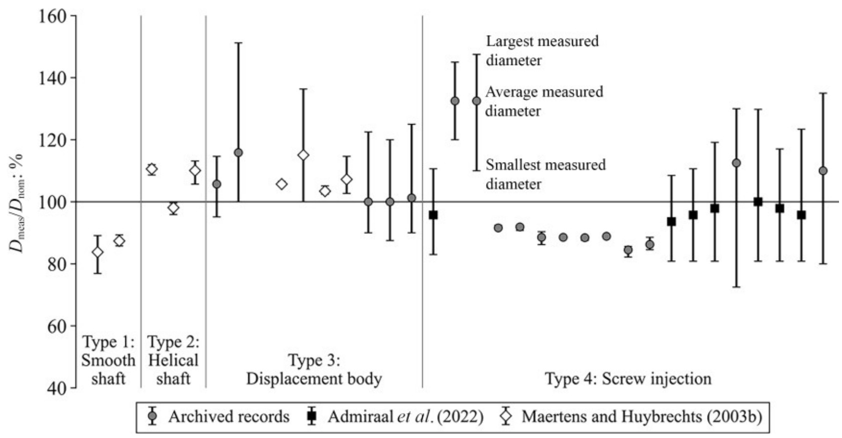

Figure 4. Measured shaft diameters of extracted piles. The nominal outermost diameter D_{nom} ranges from 400 mm to 550 mm

ultimately failed due to structural debonding – although it should be noted that these piles had a diameter and length much larger than any of other database records (D_{nom} = 850 mm; L = 35–37 m).

The results also suggest that the Type 3 piles mobilised the highest normalised shaft resistances ($\alpha_s = q_{s,avg}/q_c$). Nevertheless, each pile type shows considerable variation, which is likely because of variations in installation procedure, curing conditions or cross-sectional stiffness (Flynn and McCabe, 2021a). As a result, a best-fit α_s was applied across the measurements from all three pile types. This yielded a best-fit α_s of 0.012 (both with and without the Amaliahaven test piles), corresponding to a coefficient of variation (COV) of 37%.

Figure 5. Base of four extracted piles, all installed at the site

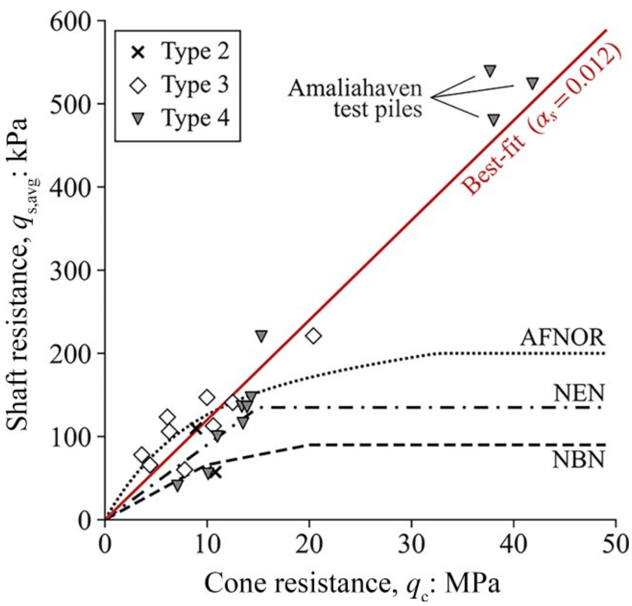

Figure 6. Variation in q_s with q_c in granular soils. No instrumented records on Type 1 piles were available

5.2 Base capacity

Base capacities were derived from the lowermost strain gauge measurement, extrapolated to the pile base. The AFNOR, NBN and NEN methods tended to overestimate the base capacity (Figure 7), in some cases by a factor of four. Both NBN and NEN also exhibited a high COV in their estimations. Part of this variation can be attributed to their respective q_c averaging methods, which were shown to introduce bias in the derived α_p when the pile base was only embedded at a shallow depth into the load-bearing sand layer on the same database (Duffy *et al.*, 2024b). Therefore, for the best-fit method, $q_{c,avg}$ was determined using an adapted version (de Boorder *et al.*, 2022) of the filter method by Boulanger and DeJong (2018), resulting in a best-fit α_p of 0.25 and a lower COV of 31%. When the original filter method was used, α_p decreased slightly to 0.22 with a COV of 33%. No distinct trend was evident across the three pile types, with a constant α_p fitting well across the entire range of cone resistances.

6. Total capacity prediction

To compare the design methods to the measured total capacities, several procedures and assumptions were made, as described below.

- The failure criterion specific to each design method was used (Table 1). Furthermore, none of the load–displacement curves was extrapolated and therefore more database entries were available for the FHWA method because of its lower displacement criterion (s_0 = 25.4 mm) compared to other design methods ($s_b = 0.1D_{nom}$).
- A mechanical CPT was used for nine tests and so its measurements were equated to that of an electronic CPT using the correlation by Kulhawy and Mayne (1990):

3. $$\left(\frac{q_c}{p_a}\right)_E = 0.47\left(\frac{q_c}{p_a}\right)_M^{1.19}$$

where p_a is the atmospheric pressure (=101.3 kPa).

- The groundwater level was not reported for 24 tests and was therefore assumed to be 1 m below the surface.
- If the base displacement was not measured, a load-transfer analysis was used to estimate the pile's elastic shortening and base displacement.
- CPTs were available in digital format for 43% of the sites. CPT results were digitised for the remaining sites, which introduced some pseudo-measurement error, especially in low-q_c regions.
- When determining w_s and w_t in the FHWA method, half of the sites did not indicate soil angularity and gradation. For these sites, w_s and w_t were set to that of a well-graded, angular material with a fines content less than 10% (w_s = 160 kPa and w_t = 1.34 MPa).

6.1 Overall performance

The ratio of the calculated to measured capacity Q_{calc}/Q_{meas} was used to evaluate the performance of the design methods (Figure 8, Table 3), where values greater than 1.0 indicate an overestimation of the pile capacity.

The best-fit parameters (α_s = 0.012 and α_p = 0.25) gave the best agreement on average but with a higher variability, with a mean Q_{calc}/Q_{meas} of 0.93 suggesting a tendency for the method to underestimate the total capacity. NBN and NEN also tended to underpredict the total capacity, yielding mean Q_{calc}/Q_{meas} ratios of 0.86 and 0.88, respectively. By contrast, the AFNOR method tended to overpredict the total capacity, consistent with the results from the instrumented records at cone resistances of less than 15 MPa (Figures 6 and 7). The FHWA method gave the lowest accuracy and usually overestimated the pile capacity, often by at least 50% (Figure 8). The results from the FHWA method were also highly dependent on the choice of w_s and w_t (factors representing the soil gradation and angularity), which were both subjective to determine within the criteria given by Brown *et al.* (2007).

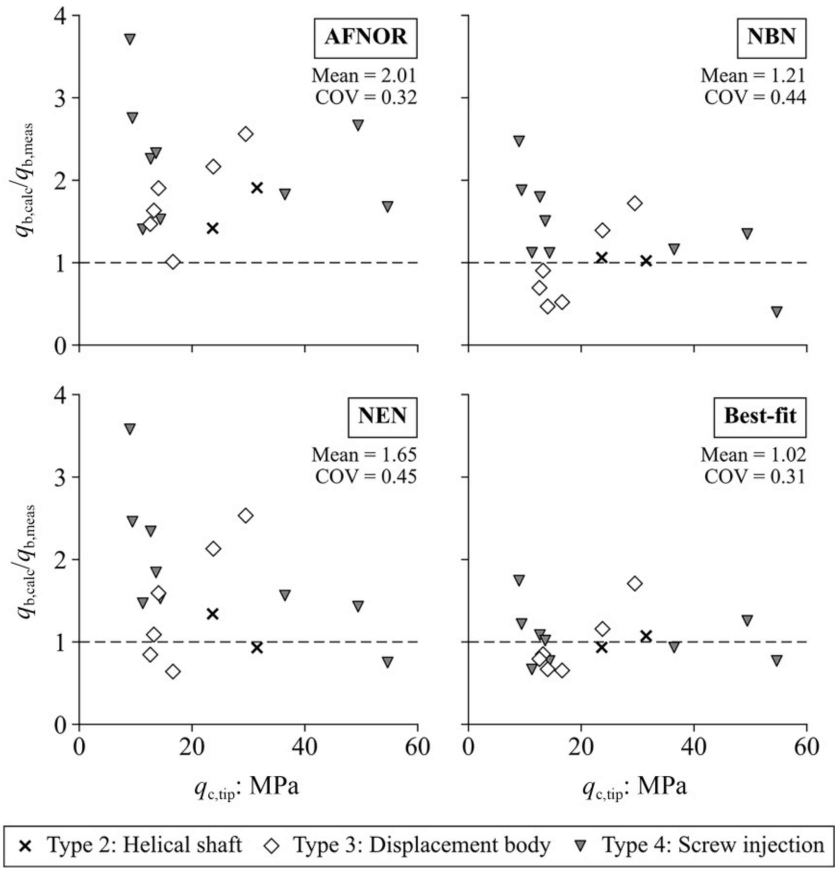

Figure 7. Predictions of the pile base resistance at $s_b = 0.1\ D_{nom}$. No instrumented records on Type 1 piles were available

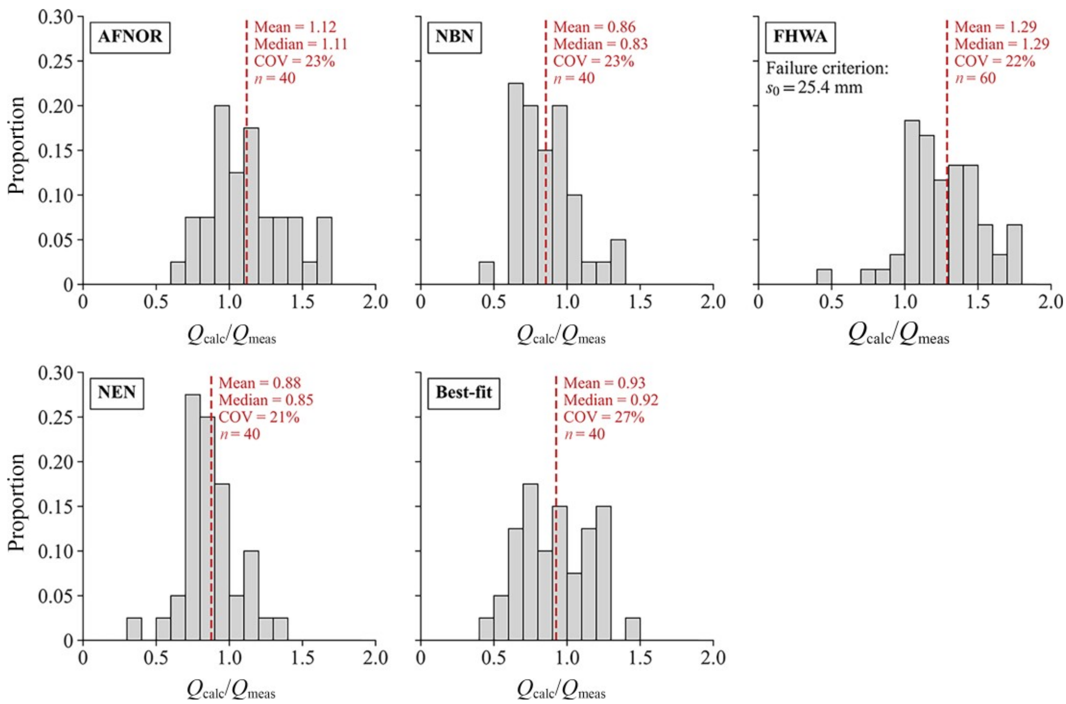

Figure 8. Comparison of the calculated capacity by each design method to the measured capacity, where n is the number of tests

Table 3. Calculation results of the total pile capacity

	Q_{meas}: MN		Q_{calc}/q_{meas}: MN				
Site & pile	$Q_{0,1inch}$	$Q_{0,0.1D}$	AFNOR	FHWA	NBN	NEN	Best-fit
Type 1: Smooth shaft							
Almere SBP4	1.7	—	—	0.96	—	—	—
Almere SBP5	1.0	—	—	1.10	—	—	—
Almere SBP6	1.0	—	—	0.75	—	—	—
Limelette A1	2.9	3.0	0.79	0.89	0.66	0.78	0.55
Limelette C1	1.6	—	—	1.42	—	—	—
Seildeich P1	2.6	—	—	1.32	—	—	—
Seildeich P2	3.0	—	—	1.03	—	—	—
Terneuzen P02	5.8	6.0	1.02	1.28	0.88	0.77	1.08
Terneuzen P04	4.8	5.3	1.09	1.46	0.95	0.83	1.11
Terneuzen P06	3.7	4.3	1.26	1.79	1.03	0.95	1.18
Zimmerplatz P1	2.8	—	—	1.02	—	—	—
Type 2: Helical shaft							
Berlin P2	3.3	—	—	1.29	—	—	—
Gent P1	2.6	—	—	1.25	—	—	—
Gent P2	2.6	2.9	0.94	1.08	0.68	0.84	0.60
Limelette A2	2.9	—	—	1.16	—	—	—
Limelette B3	2.9	3.4	0.85	1.11	0.67	0.82	0.71
Limelette B4	2.9	3.3	1.05	1.09	0.76	0.82	0.72
Limelette C2	2.6	—	—	1.19	—	—	—
Oldenburg P1	2.3	—	—	1.60	—	—	—
Oldenburg P2	1.6	—	—	1.08	—	—	—
Oostwoud P27	1.2	1.2	1.46	1.53	1.12	1.15	1.14
Oostwoud P28	1.3	1.4	1.32	1.37	0.85	1.06	0.94
Zeeland O1	1.2	1.3	1.65	1.78	1.21	0.94	1.29
Zeeland O2	1.5	1.5	1.64	1.74	1.32	1.32	1.29
Zeeland O3	1.5	1.8	1.43	1.67	1.09	0.97	1.11
Werkendam P5	1.6	—	—	1.50	—	—	—
Werkendam P6	1.8	—	—	1.39	—	—	—
Werkendam P8	1.7	—	—	1.35	—	—	—
Type 3: Displacement body							
BEST C2	1.8	2.0	0.62	0.42	0.41	0.37	0.42
Elblag P9	1.1	—	—	1.18	—	—	—
Gdansk P809	1.5	1.6	1.18	1.38	0.92	0.98	0.87
Grottgera P600	1.1	1.2	1.15	1.41	0.72	0.68	0.81
Grottgera P602	1.4	1.5	0.91	1.15	0.62	0.59	0.68
Hamburg PA	2.8	—	—	1.04	—	—	—
Hamburg PB	1.9	2.1	0.86	1.05	0.62	0.68	0.60
Limelette A3	2.5	2.8	0.79	0.95	0.62	0.75	0.62
Limelette A4	2.1	2.4	0.92	1.16	0.73	0.86	0.72
Limelette C3	2.4	2.8	0.79	1.03	0.68	0.74	0.61
Limelette C4	2.0	2.3	1.00	1.11	0.77	0.83	0.73
Loenhout S2	1.0	1.2	1.32	1.44	1.06	1.07	0.88
Lomme P1	1.4	1.5	1.33	1.43	1.06	1.17	0.99
Oostende P1	2.6	3.2	1.20	1.60	0.99	1.00	1.05
Oostwoud P1	1.4	1.5	1.41	1.55	0.98	1.16	1.04
Oostwoud P22	1.7	1.7	1.13	1.19	0.86	0.87	0.92
Oostwoud P6	1.1	1.1	1.66	1.80	1.40	1.25	1.49
Vilvoorde P5	1.4	1.8	1.02	1.27	0.72	0.76	0.80
Werkendam P1	2.1	2.4	0.94	1.25	0.75	0.71	0.88
Werkendam P2	2.3	—	—	1.12	—	—	—
Werkendam P3	2.1	—	—	1.30	—	—	—
Werkendam P4	1.9	2.1	1.16	1.46	0.99	0.89	1.19
Zulawy P6293	0.8	0.9	1.54	2.16	0.80	1.12	0.96
Type 4: Screw injection							
Amaliahaven SI1	15.0	20.4	1.13	1.39	0.93	0.93	1.22
Amaliahaven SI2	16.1	23.4	1.28	1.39	0.92	0.86	1.25
Amaliahaven SI3	13.2	18.6	1.12	1.63	0.91	0.92	1.29

(continued on next page)

Table 3. Continued

	Q_{meas}: MN		Q_{calc}/q_{meas}: MN				
Site & pile	$Q_{0,1inch}$	$Q_{0,0.1D}$	AFNOR	FHWA	NBN	NEN	Best-fit
Amaliahaven SI4	14.6	19.0	1.17	1.30	0.76	0.78	1.21
Delft F1	2.5	2.7	0.85	1.03	0.69	0.78	0.72
Delft F2	2.3	2.6	0.92	1.05	0.67	0.73	0.69
Delft T1	2.3	2.5	0.92	1.14	0.76	0.86	0.79
Terneuzen P03	5.0	6.1	0.97	1.43	0.84	0.74	0.95
Terneuzen P05	5.3	6.4	0.94	1.35	0.81	0.72	0.93

Each method also gave different results in terms of the relative shaft-to-base contributions. The FHWA, NBN and best-fit methods tended to result in primarily shaft-bearing piles, with the shaft resistance in sand at least 60% to the total capacity on average. By comparison, the AFNOR and NEN methods tended to give comparable shaft-to-base contributions in sand. The contribution of fine-grained soils to each method (as determined by the Bustamente and Gianeselli method) was less than 15% for most of the database records. The exceptions were the Gent and Limelette sites, where clay contributed 10–50% of the calculated capacities. Correspondingly, the measured capacity at these sites was generally underestimated by four of the design methods – the only exception being the FHWA method.

The coefficient of variation of Q_{calc}/Q_{meas} across all design methods ranged from 21 to 27%, with the lowest variation given by the NEN method and the highest variation by the best-fit method. For context, the unified design method for driven piles (Lehane *et al*., 2020), developed on a high-quality dataset of 71 piles (Lehane *et al*., 2017), yielded a COV of 24%. Similarly, a study of bored and continuous flight auger piles (Doan and Lehane, 2021) found a COV of 30% in sand.

6.2 Performance by pile type

To explore the results further, the predictions by pile type (Table 4) show that the best fit to the instrumented data performed well for Type 1, Type 2 and Type 4 piles, with Q_{calc}/Q_{meas} close to 1.0. The NBN and NEN methods also performed well for Type 2 piles, but underestimated the capacity of the other pile types by 10–20% on average. AFNOR yielded good predictions for Type 1 and Type 4 piles, but the method overestimated the capacity of Type 2 piles by 29% and Type 3 piles by 10%. The FHWA method tended to overestimate the capacity of all pile types by at least 18%.

Most of the prediction error occurred with Type 3 piles. NBN, NEN and the best-fit method tended to underestimate their measured capacities, whereas AFNOR and FHWA tended to overestimate the capacities. Predictions for Type 3 piles also exhibited the highest variability of all four pile types, with a COV ranging from 25 to 28% across all design methods.

Correspondingly, all five design methods show a clear bias with respect to pile slenderness, exemplified by the best-fit method in Figure 9. This bias may indicate that a change in the respective shaft-to-base contributions influences the prediction accuracy, corroborated by the instrumented records (Figure 6), which suggest that Type 3 piles mobilise higher shaft resistances compared to the other pile types. Furthermore, several laboratory and numerical studies (Larisch, 2014; Pucker and Grabe, 2012; Shi *et al*., 2019; Slatter, 2000) suggest that the radial stress conditions around Type 3 piles vary with depth, depending on both the downward and rotational installation velocities as well as the soil conditions prior to installation. However, further research is needed to incorporate such findings into CPT-based design, particularly full-scale tests focusing on the influence of drilling tool extraction and concreting on the radial stresses generated from installation.

7. Conclusion

This paper presents the largest public database of static load tests on screw displacement piles to date, consisting of screw displacement piles that derive most of their capacity from granular soil. Using the database, CPT-based axial design methods from Belgium, France, the Netherlands and the USA were compared to the measured base, shaft and total capacities. The results suggest that current design methods tend to overestimate the pile base contribution while underestimating the shaft contribution, particularly at high CPT cone tip resistances where design methods assume a

Table 4. Summary of the capacity prediction by pile type

	Mean					Coefficient of variation (COV): %				
	AFNOR	FHWA	NBN	NEN	Best-fit	AFNOR	FHWA	NBN	NEN	Best-fit
Type 1	1.04	1.18	0.88	0.83	0.98	19	25	18	10	30
Type 2	1.29	1.36	0.96	0.99	0.98	24	17	26	18	28
Type 3	1.10	1.28	0.83	0.87	0.86	25	26	27	26	28
Type 4	1.03	1.30	0.81	0.81	1.01	14	15	12	10	24
All piles	1.12	1.29	0.86	0.88	0.93	23	22	24	21	27

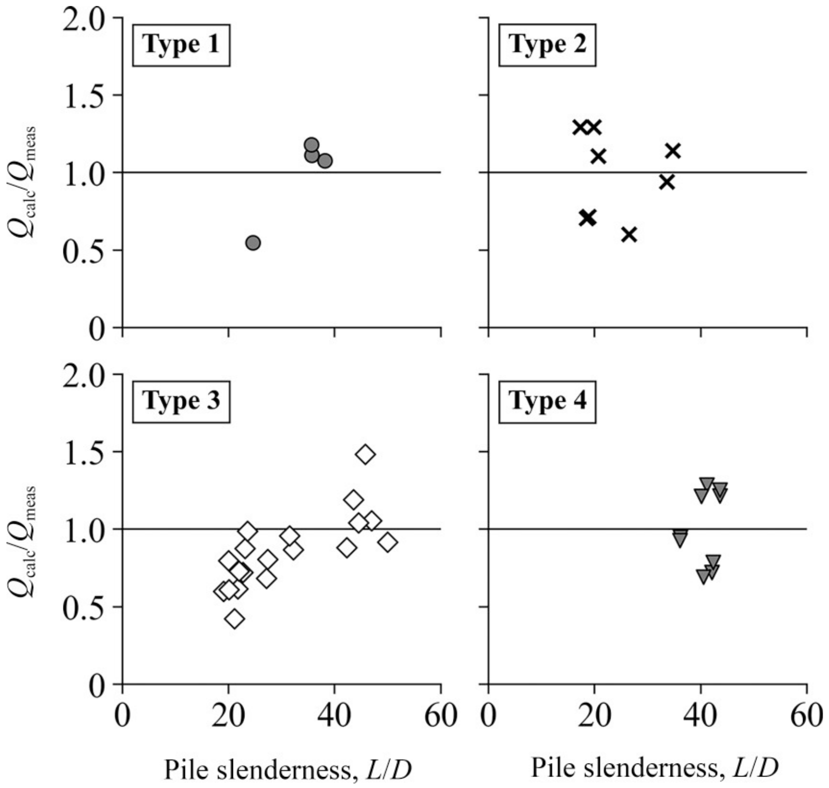

Figure 9. Change in predictive performance with pile slenderness for the best-fit method

non-linear relationship between the cone tip resistance and the pile base and shaft capacities.

A design method was derived from a linear best fit to the measured shaft and base capacities (α_s = 0.012 and α_p = 0.25), yielding better predictions on average compared to the existing design methods. However, the high variability in the predictions, particularly for piles with a displacement body (e.g. De Waal and Omega piles), may suggest a possible dependency of the piles' capacities on the installation procedure and drilling tool geometry. Further investigation into these effects may help resolve this variability and provide specific design recommendations for each type of screw displacement pile. Additional instrumented load tests with installation data along with small-scale physical and numerical models would help in this regard, particularly with understanding the exact mechanisms affecting stress development around the drilling tool during installation and casing withdrawal.

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