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Pile shoe geometry influencing installation and bear capacity of ductile iron piles – field tests and validation

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ABSTRACT: Piles are used in a variety of applications to transfer high loads, bridge insufficiently load-bearing soil strata or to reduce settlements. In addition to (large diameter) bored piles, displacement piles such as driven ductile cast iron piles, are often used, which are characterised by their installation and the possibilities of a straightforward adaptation to the subsoil. In particular, the use of different pile shoes enables an adjustment in terms of both installation and load-bearing capacity. Their shape and geometry have a significant influence, which is usually validated by pile load tests and only allows limited considerations during the design. Therefore, the influence of different pile shoe geometries was investigated in a series of field tests in which cone-penetration equipment was used to determine their resistance. The test was performed in an artificially constructed sandy layer with a depth of 1.2 m underlaying a dense gravel section. The focus was on a relative comparison between different pile shoe geometries under controlled conditions in order to derive the load-bearing capacity.

1 DRIVEN DUCTILE PILES

In times of ever-increasing settlement requirements, construction sites with generally poorer and sometimes contaminated subsoil conditions, and increasingly rapid implementation requests, a flexible pile foundation system is becoming increasingly important. Displacement piles made of ductile cast iron combine flexibility with increased cost effectiveness. Offering a range of pile geometries design loads of up to 2,300 kN can be safely transferred. A significant advantage is the use of a pile driving criterion, i.e. the resistance recorded during pile production can be correlated with pile bearing capacities, thus enabling the in-situ bearing capacity to be derived for each pile.

Considering their cost-effectiveness and simplicity in installation, ductile piles have found wide application in geo-engineering problems with difficult access and poor ground conditions. Use cases include, but are not restricted to rockfall protection systems, pile foundations and retrofitting of existing buildings.

Generally, the design of related structures requires a reasonable understanding of the pile performance under operating conditions. The verification of piles is routinely carried out using static pile load tests (SPLT) that provide information about the load-displacement behaviour and design parameters, such as the pile resistance at the ultimate limit state. The interpretation of results, however, is non-trival, mainly due to the complex behaviour of soils and limited control of site boundary conditions.

Static pile load tests (SPLT) constitute an essential part of piling contracts, especially in projects with limited knowledge on the subsoil ground conditions. The interested reader may refer to the German Geotechnical Society (DGGT, 2014) for a detailed discussion on relevant application limits.

According to EN ISO 22477-1;2, the primary aim of SPLT is to evaluate the pile load-displacement behaviour. Principal components of the test equipement are a load application device situated on a reaction system – depending on tensile or compression testing – and the necessary measurment equipment to gather loads and displacements. Furthermore, testing may use advanced instrumentation, to montior the load sharing between pile shaft and base or skin friction distribution (e.g. Monsberger et al., 2016) or inlcude numerical simulations on the reaction system (Granitzer et al., 2024). Furthermore, with a static load test system - the Pile HAY-Proof-System® (Figure 1 and Hayden, 2025), a difference between base resistance and shaft friction and a determination of the overall load-bearing capacity could be given.

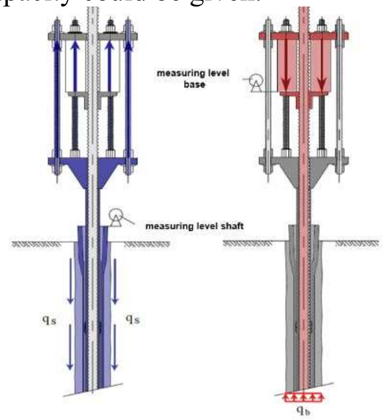

Figure 1: Operation principle Pile HAY-Proof-System® (Hayden M. , 2025).

The test system has been successfully applied to more than 200 tests piles of different pile types. It was mainly used to measure load-bearing capacity of grouted ductile iron piles. In addition, the resistance components of base resistance and shaft friction were also measured on large bored piles, micropiles (GEWI® piles), non-grouted steel piles and Zeissl piles (Hayden M. , 2025).

The results of such pile load tests reflect the properties of the subsoil, but can also be influenced by installation effects and other boundaries. It can be assumed that with a dynamic installation method such as used for ductile iron piles, among other things, the geometry of the pile shoes, can play a major role. For this reason, different pile shoes with varrying geometries were investigated in a field test in order to draw conclusions about their influence on the installation and load-bearing behavior.

2 TEST FIELD, SET-UP AND PILE SHOE GEOMETRY

A test field was set up to investigate the influence of pile shoe geometry on ductile iron piles. The aim was to determine the effects of different pile shoe types and diameters (see Chapter 2.2) on load-bearing capacity. An overview of the investigation area can be seen in Figure 2.

2.1 Artificial test pit

In order to ensure as homogeneous a substrate as possible for validating the influences of the pile shoe geometry, an artificial pit was created in which indentation tests could be carried out. For this purpose, an excavator trench with a width of 90 cm and a depth of 90 cm, as shown in Figure 2 (a) and (b) was created. This was then backfilled with sand with a D_{10} of 0.259 mm and a coefficient of uniformity of 2. The backfill was carried out layerwise while compacting.

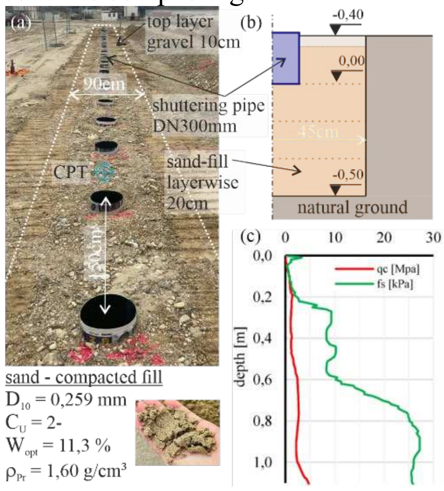

Figure 2. Field testing pile shoe geometries, artificial pit filled with sand and installed shuttering pipes (a), cross-section of the test pit (b), representative CPT-data of the test-pit (c).

The pile shoes were then pressed into this artificial soil body. To ensure adequate guidance, shuttering pipes (see Figure 2 (b)) with an inner diameter of 300 mm were installed in the top layer. These also served to prevent surface disturbance of the subsoil during testing. A layer of gravel with a depth of 400 mm was also applied as the top layer to enable vehicle access and to provide erosion protection.

2.2 Pile shoe geometries

For the investigations, two key parameters were varied for the different geometries. The first was their design ranging from a flat bottom on one level (Types 1, 5, and 7), a flat bottom on different levels (Types 3 and 8) conical shapes (Type 4, 6), and designs with a rock point (Type 2) were examined. The second parameter was that three different diameters (170, 220, and 270 mm) were used. An overview of the types used is provided in Figure 3.

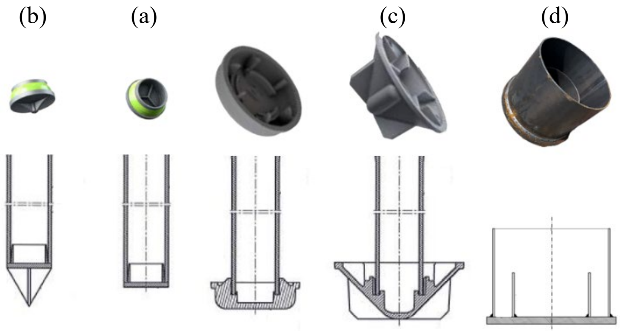

Figure 3. Pile shoe geometries and cross sections used.

Table 1. Types of pile shoes and geometries.

type	design	bottom	diameter D [mm]
1	a	flat TRM	170
2	b	rock point	170
3	a	flat TRM	220
4	c	conical	220
5	d	flat Keller	220
6	c	conical	270
7	d	flat Keller	270
8	a	flat TRM	270

2.3 Load application and testing

In order to detect a difference in the load-bearing capacity of different pile shoe geometries, they needed to be subjected to an appropriate load. For this purpose, the artificial testing pit was constructed and shuttering pipes with a length of 500 mm (400 mm beneath the surface) were installed to rule out any influence from the surrounding soil and to neglect low stressed near-surface zones of the soil body. As a preliminary step, a test was carried out in which the general procedure and the test concept were validated using a simple force-controlled cylinder without displacement measurement as shown in Figure 4 (b).

For further tests, equipment otherwise used in cone penetration testing (CPT) was utilzied. This ensured that the test elements were subjected to the appropriate load and also enabled the subsoil in the test area to be investigated.

For the load test, the CPT-truck was positioned directly above the test area with the installed shuttering pipes, as shown in Figure 4 (a). A coupling which was shaped in analogous a pile pipe was attached to the pile shoes already placed on the soil body. This was necessary in order to incorporate a piece of ductile iron pile into the test, which ensured an even load application on the pile shoe and enabled a connection between the sounding rod and the pile pipe. In addition, as shown in Figure 4 (c), a rope was attached to the coupling to ensure that recovery after the test was performed was possible.

In addition, cone penetration tests were carried out between the tested pile shoes in order to draw conclusions about the structure and homogeneity of the artificially created test pit. A representative test result is given in Figure 2 (a), showing that the soil is approximately homogeneous, loose to medium dense, down to a depth of 90 cm. The results are similar for all investigation areas, from which it can be concluded that the initial conditions for the investigations were the same.

Figure 4. Performing field tests, CPT ruck arranged over the artificial test pit (a), preliminary load tests using a hydraulic jack and a excavator as counterweight (b), CPT-piston used for load application during the test (c).

3 TEST RESULTS

During the field tests, the pile shoes (type 1 to 8 see Figure 3) were continuously pressed into the artificial test pit. For this purpose, a penetration rate of 2 mm/sec. for the CPT device was chosen and the test elements were pressed to a depth of at least 60 cm below ground level or, for scientific reasons, until the maximum applicable load of 150 kN was reached. Since, as can be seen in Figure 2, after a depth of 90 cm the measurements reached natural ground at which the stratification, and compaction were unknown. Therefore, the following interpretations are limited to a depth of 60 cm.

A total of 5 pile shoes per type (see Figure 1) was installed in the artificial test pit. The following evaluation and result discussion is based on the mean values of these investigations. This approach was chosen due to the low fluctuations between the individual tests.

An overview of the load tests performed and its reults is provided in Figure 5. Here the normalized apllied load ($F_{b,ult,nom}$) is plotted over the depth of the test pit. For further reference, the maximum applied load ($F_{b,ult}$) and the loads at 2 cm (green dotted line, $F_{b,2}$) and 20 cm (yellow dotted line, $F_{b,20}$) depth are given in Table 2.

The representation in Figure 5 shows that almost all tests have been carried out up to the maximum load ($F_{b,ult}$ in Table 2), but also reveals that different load increases occurred over depth. In addition, the results in Table 2 show, that although the maximum load could be applied during the test, there are significant differences in the settlement behavior within the considered investigation depth of 60 cm. For example, between type 1 and type 8 a variation of more than 70% can be seen.

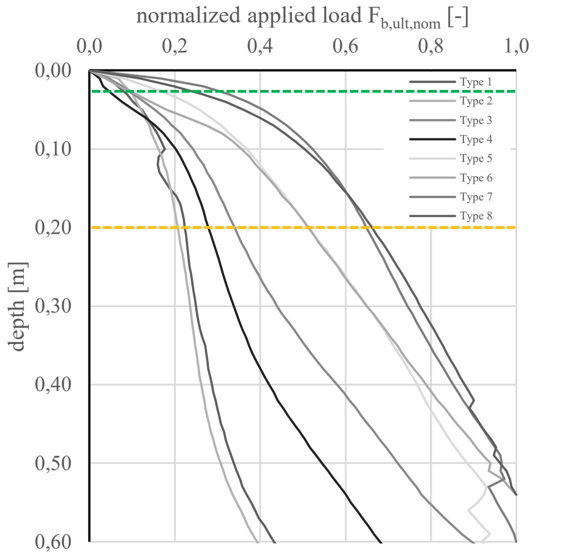

Figure 5. Normalized load-deformation-curve limited to a depth of 60 cm below GL of the test pit.

Table 2. Bearing capacity at representative depth of 2 cm and 20 cm – green and yellow lines Figure 5.

type	max load	$F_{b,i}$ at depth [m]	
	$F_{b,ult}$ [kN]	0.02	0.20
1	121.80	8.05	27.18
2	140.10	11.68	28.97
3	148.09	11.37	50.36
4	146.80	4.67	40.89
5	148.76	19.98	76.35
6	142.91	9.81	73.17
7	143.02	38.14	92.68
8	146.58	28.92	96.92

As described in Chapter 2 and shown in Figure 3, the pile shoes examined are characterized by different head geometries and diameters. To enable a sufficient comparison, the loads ($F_{b,i}$ in Table 2) were therefore related to the pile base area A_i according to Formula 1, as shown in Figure 6. These loads at 2 cm ($q_{b,2}$ – green) and 20 cm ($q_{b,20}$ – yellow) are further used in the interpretation.

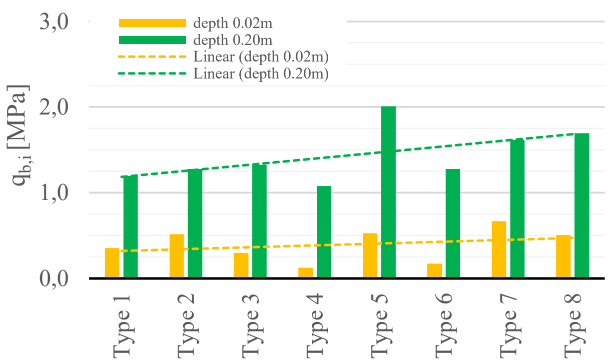

Figure 6. Bearing capacity at the pile base $q_{b,i}$ for a depth of 2 cm (orange) and 20 cm (green).

$$q_{b,ult,i} = F_{b,i}/A_i \qquad (1)$$

The diagram illustrates, that there is a clear increase in bearing capacity with depth, which is to be expected with a simultaneous increase in the effective stresses. This explains the resulting parallel shift between the depth of 2 cm (green) and 20 cm (yellow). However, the difference in inclination of the resulting linear regressions lines is not relates to this soil mechanical boundary.

4 CONCLUSIONS

As can be deduced from this article, the shape of the driving shoe has a significant influence on the bearing capacity of the base. Even the fact that the flat shape of the driving shoe lies on one level (types 1, 5 and 8) or whether the total diameter is divided into 2 levels (types 3 and 8) already has a significant influence on the load-bearing capacity. If one now compares the conical (type 4 and 6) with the flat (driving shoe surface lying on one level – types 5 and 7), a reduction in the load-bearing capacity of approx. 74 – 77 % can be recorded, depending on the diameter in a serviceability stage realistic depth of 2 cm.

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