Design of piles – German practice

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ABSTRACT

Germany has a long tradition of standardization with regard to the execution and design of pile foundations and other pile systems. With the introduction of Eurocode 7 the principle of partial safety factor approach have replaced the global safety factor approach so far used for pile design as well as for other geotechnical design since many decades. In consequence the existing standards and recommendations were revised and adapted to the new European regulations. After a transition period with German codes adapted to the partial safety factor approach, Eurocode 7-1 in combination with the National Annex and with DIN 1054:2010-12 including national supplementary rules to EC7-1 – all three together called ′German Handbook EC 7 - Part 1′, are the basis for geotechnical design and as well for execution and design of pile foundations in Germany; they are implemented as binding building regulations since 2012. Additional guidance for pile design and execution is provided by the ′Recommendations on Piling (EA-Pfähle)′ which were elaborated by the German Piling Committee. These recommendations firstly published in 2007 are now well-established as best practice regulations and provide comprehensive support for all aspects of pile design covering also specific issues like negative skin friction, group effects, cyclic and dynamic loading etc. This combination of standards and recommendations reflects also the German basic understanding that standards should focus on the principles of design and safety concepts whereas recommendations might provide more detailed support for engineering practice e.g. with different calculation methods, background information, continuative literature etc. In this context the ′EA-Pfähle′ offers a specific approach to design axially loaded piles whereby this approach used as standard method bases on the empirical evaluation of comprehensive databases with pile load test results. Due to the geological diversity in the subsurface of Germany the soil and rock conditions vary in a wide field and therefore a wide spectrum of pile types is used in Germany comprising nearly all kinds of bored piles, displacement piles and micro piles.

1. REGIONAL GEOLOGY

1.1. Geological overview

The geology of Germany is heavily influenced by several phases of orogeny in the Paleozoic and the Cenozoic, by sedimentation in shelf seas and epicontinental seas and on plains in the Permian and Mesozoic as well as by the Quaternary glaciations.
The Geological sketch map of Germany (Figure 1) reflects the amazing diversity in the subsurface of Germany. It is the result of many, for hundreds of millions years ongoing processes that have shaped the geological underground in this part of Central Europe: sedimentation, mountain building, intrusive and volcanic eruptions of magma, metamorphism, erosion and glaciation.
The oldest rocks in the Precambrian of Germany arised more than 540 million years ago. They are found in *Bohemian*, *Bavarian* and *Upper Palatinate Forest*, the *Erzgebirge, Lusatian Mountains*, in the *Saxon Granulitgebirge*, the *Münchberg Gneissmasse*, the *Black Forest* and parts of the *Odenwald* and *Spessart*. These crystalline regions which consist of both metamorphic converted sedimentary rocks and granitic intrusive rocks have changed strongly since their creation by pressure, temperature and changing chemistry.
In the Cambrian up to Silurian (540-410 million years ago) shallow seas flooded the Germany space indicated today by shale and sandstone in *Saxony* and *North-East Bavaria*.
In the Devonian period (410-355 million years ago) these seas deepened to large basins in which sediments accumulated to powerful beds. This is proved by the shales, sandstones and limestones in the *Rhenish Slate Mountains*, *Hunsrück* and *Taunus* and in the *Harz* as well as in the *Thuringian-Saxon-Frankish-slate mountains.*
During the subsequent Carboniferous (355-295 million years ago) the ocean basins filled with sandy-clay and calcareous sediments. At the same time the Cambrian sedimentary rocks were folded gradually. Generally, this Variscan mountain belt strike from southwest to northeast such as in the *Rhenish Slate*

Mountains. At the end of this geological period large parts of Germany were covered by jungle and swamps. Dead trees and other organic material collected in sinks and turned over millions of years by the pressure of overlying sediments to coal, wellknown from *Ruhr area*.
The following time of the Permian period was marked by a warm, dry desert climate. The reddish desert sand deposits of the *Cisuralian* and *Guadalupian (Rotliegend)* (295-260 million years ago) are often associated with volcanic rocks such as in the *Saar-Nahe* region. In the *Lopingian (Zechstein)* (260-250 million years ago) shallow seas pushed forward from the north. They gradually evaporated, leaving behind limestone, dolomite and salt, which today are mined as rock salt and potash in *Northern Germany* and in the area of *Hessen - Thuringia*.
Also in the subsequent Triassic Germany consisted mainly of land. Especially during the periods of *Early (Buntsandstein)* (250 to 240 million years ago) and *Late Triassic (Keuper)* (230-203 million years ago), in rivers and lakes sandstones and claystones were constituted. In the *Middle Triassic (Muschelkalk)* the area was flooded, leaving limestone and shell limestone in the *German mountain range.*
In the Jurassic (203-135 million years ago) Germany was again maritime area. During this period massive layers of limestone, sandstone and claystone were deposited which built together with those of the Triassic stage and hogbacks of the *Swabian and Franconian Alb* in southern Germany and in the *Weser- and Leine-Bergland.*
In the north the floods stayed until *Cretaceous* (135 to 65 million years ago). Besides the well known chalk cliffs on *Rügen* limestone and shales were built. Near the coast sandstones arised, eg at *Teutoburg Forest* and *Egge Range, Deister* and at the edge of the *Harz* as well as in the *Saxon Switzerland* and nearby *Zittau*, which today are often washed out bizarrely shaped rock formations. In the Cretaceous the formation of the *Alps* began to arise in southern Europe. This geologically young mountain range is comparatively high and not so far eroded than the older mountain ranges. The *Alps* are typical fold mountains characterized i.e. by the formation of extended rock bodies, torn from their formation, moved and stacked like blankets.
In the central and southern Germany in the *Tertiary* (about 65 to 1.75 million years) many active volcanoes exists. The volcanic rocks at *the Vogelsberg, Knüll, Rhön, Habichtswald* and *Meissner* in *Hessen*, in *Lusatia* and in *Northern Bavaria*, in the *Westerwald* and the *Siebengebirge* beside the Rhine, the *Kaiserstuhl* at Breisgau and the *Hohentwiel* in Swabia testify, as are the crater lakes of the *Eifelmaare*, which origin take up to Quaternary.
In the Tertiary brown coal was formed of the *Lower Rhine*, *East German*, and *Lusatian* and *Helmstedter* grounds. At the same time the *Rhine Valley* lowered and filled with sediments that were deposited in the foothills of debris from the rising Alps as molasses. In the Late Tertiary (about 14.7 million years ago) in *Nördlingen* a meteorite alighted and altered rocks and landscape of the area proposed sustainable.
The recent and still ongoing geological period, the Quaternary period began 1.75 million years ago. In the Pleistocene, until 10,000 years ago, Germany was characterized by deposits and landforms of the ice, such as Moraines, ground moraines and glacial valleys. In *Northern Germany*, the glaciers from Scandinavia outreached across the *Baltic Sea* up to south of the mountain ranges. The main glaciations in the north German lowlands are named after rivers, indicating the scope of its ice sheets: *Elster glaciation*, *Saale glaciation* and *Weichsel glaciation*.
At the same time glaciers reached out from the Alps into the Alpine foothills. The main glaciations in alpine areas are named *Günz, Mindel, Riss* and *Würm* glaciation.
The deposits of the Quaternary can be differentiated also on genetic factors: Particulary beside the North Sea coast areas are found which have been formed by processes occurring in the sea. The *North German Plain* is dominated by large peat bogs. High-and low-moors are closely associated. In wide floodplains fluvial layers which are caused by the influence of flowing waters are differentiated according to their temporal development during the various hot and cold periods. Last but not least in Northern Germany, large areas are found caused by the influence of the wind.

1.2. Consequences for application of pile foundations in Germany

Due to diversity of the geological conditions in Germany the soil and rock conditions vary in a wide field and therefore a wide spectrum of pile types is used in Germany comprising nearly all kinds of bored piles, displacement piles and micro piles. Also it is hardly possible to describe ´typical´ conditions piles in Germany are quite often installed as follows:

- bored piles in all soil conditions varying from quarternarry and terrtiary granular and cohesive deposits to soft and hard rock conditions like sandstones, lime- and claystones etc.
- all kind of displacement piles in granular and cohesive soil conditions especially when an underlying stiffer soil layer can be reached by the piles,
- micro piles used in a wide field of application (tension piles, improvement of foundations etc.) in all soil conditions.

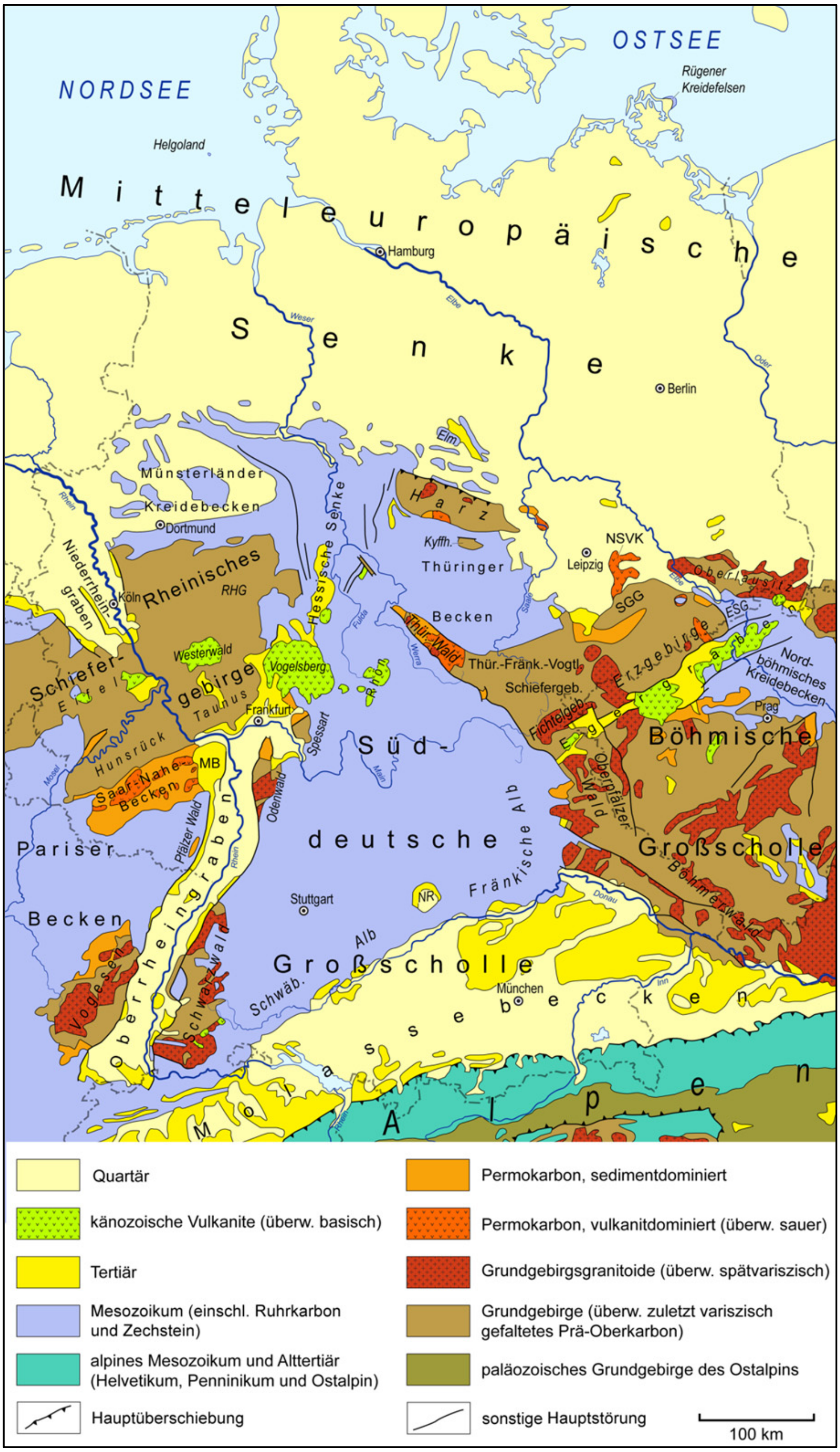


Figure 1: Geological sketch map of Germany and adjacent areas, based on Henningsen & Katzung (2006), Pawlewicz et al. (2003), BGR (2008) and Freudenberger & Schwerd (1996): Simplified map of the surface geology of Germany. The Central European Depression (Mitteleuropäische Senke) (light yellow) is almost completely covered by Quaternary deposits (Quartär). The Central European Blocks area appears mainly in violet (Mesozoic + Zechstein + Ruhr Carboniferous (Silesian)) and brown (before late Carboniferous). In the far south are the Alps.

2. SOIL INVESTIGATION

The structure and properties of the soil and rock and the groundwater conditions must be known in sufficient detail for any piling project. This is necessary to reliably assess the stability and serviceability of the pile foundations and of the overall structure as required by EC 7 and DIN 1054 and to assess the effects of pile foundations on their surroundings. This information must also be sufficient to allow technically the competent pile installation or construction, e.g. based on DIN EN 1536, DIN EN 12699 and DIN EN 14199, taking the German classification standard DIN 18301 (VOB/C) into consideration.

To this end, project-specific geotechnical investigations shall be carried out in accordance with the EC 7-2 Handbook (DIN 2011b). The results shall be summarised in the Geotechnical Investigation Report and be evaluated in the Geotechnical Design Report regarding the technical consequences for the construction.

The German EC 7-2 Handbook (DIN 2011b) stipulates that the type and scope of geotechnical investigations depend on the geotechnical categories (see section 5.1) and shall be specified in detail by the geotechnical expert.

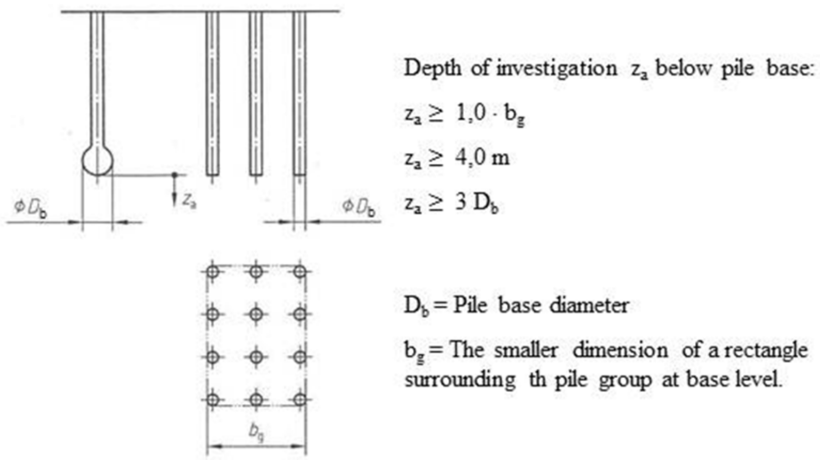


Figure 2: Minimum ground investigation depths for pile foundations, from EC 7-2 Handbook (DIN 2011b); Note: if the pile resistances of compression pile foundations are determined based on data from proven experience acc. to ´EA-Pfähle´, the ground investigations should extend to a depth below the pile base of at least $z_a \geq 4D_b$

The geotechnical investigations must extend to sufficient depth to record all ground formations and strata influencing the structure and its execution, and to identify the load-bearing and deformation properties of the ground as mentioned in EC 7-2 Handbook [45] and Figure 2. In addition to the stipulations in Figure 2, the ground investigations should extend to a depth of at least $z_a \geq 4\ D_b$ below the pile base, if the pile resistances are determined based on empirical data according to ´EA-Pfähle´ (see section 5).

In German design practice the undrained shear strength c_u for piles in cohesive soils and the CPT cone resistance q_c in non-cohesive (granular) soils are the relevant parameter mostly used as relevant parameters to consider in calculation the skin friction and base resistance.

Soil investigations for pile foundations usually combine explaratory boreholes requesting a full recovery of soil and rock cores with soundings and laboratory tests on soil and rock samples. As soundings heavy dynamic probing (DPH) and cone penetration tests (CPT) are most frequently used, whereas the use of CPTs is increasing. Pressuremeter tests and other borehole tests are still relatively seldom used additionally. Laboratory tests often focus on classification tests, on tests to determine the c_u-value for cohesive soils and on uniaxial compression tests (q_u) for rock conditions.

It is permissible to correlate empirical data, if similarity can be demonstrated by means of suitable investigations, e.g. penetration tests, vane tests, pressiometer and similar tests.

The German ´Recommendation on Piling (EA-Pfähle)´ defines requirements on the extent of soil investigation for pile foundations as well as on the content of a Geotechnical Investigation Report and a Geotechnical Design Report.

EA-Pfähle does also provide some correlation data e.g. for correlations between different types of investigation for pile foundations (see Table 1 for non-cohesive soils and Table 2 for cohesive soils). The applicability of the tabled data for the respective, specific application must be confirmed by the geotechnical expert.

Table 1: Orientation values for relationships between relative densities and penetration resistances in non-cohesive soils ($U \leq 3$) above the groundwater for use with pile foundations (´EA-Pfähle´)

Relative Density D	Density Index I_D	Description	Penetration resistances		
			q_c [MN/m²] CPT	N_{30} BDP	N_{10} DPH
< 0.15	< 0.15	Very loose	< 5.0	< 7	< 4
0.15 ... 0.30	0.15 ... 0.35	Loose	5.0 ... 7.5	7 ... 15	4 ... 9
0.30 ... 0.50	0.35 ... 0.65	Medium-dense	7.5 ... 15.0	14 ... 30	8 ... 18
0.50 ... 0.70	0.65 ... 0.85	Dense	15.0 ... 25.0	23 ... 50	14 ... 30
> 0.70	> 0.85	Very dense	> 25	> 50	> 25

Table 2: Orientation values for conversion from CPT cone resistances q_c in MN/m² and blow count N_{30} of borehole dynamic probing (BDP) (´EA-Pfähle´)

Soil type	q_c/N_{30} [MN/m²]
Fine to medium sands or slightly silty sand	0.3 to 0.4
Sand, or sand with some gravel	0.5 to 0.6
Widely-graded sand	0.5 to 1.0
Sandy gravel or gravel	0.8 to 1.0

3. PILING TECHNOLOGY & CLASSIFICATION

Due to diversity of the geological conditions in Germany a wide spectrum of pile types is used comprising nearly all known kinds of bored piles, displacement piles and micro piles.

The available pile systems, highly variable in their structure and their application options, differentiate between three groups in accordance with the respective execution standards:

a) Bored piles according to DIN EN 1536 and DIN SPEC 18140,
b) Displacement piles according to DIN EN 12 699 and DIN SPEC 18538,
c) Micropiles according to DIN EN 14199 and DIN SPEC 18539.

Figure 3 taken from ´EA-Pfähle´ classifies the pile types used in Germany into these three main groups and provides a more detailed definition and description of the execution of the different pile types.

There are no reliable data available on the piling market in Germany; therefore it is not possible to quantify the use of the different pile types. It is anticipated that bored piles might be the most often used pile type (for foundations as well as bored pile walls for excavations) followed by displacement piles (especially driven prefabricated reinforced concrete piles and cast-in-place displacement piles) and micro piles (especially cast-in-place piles). Due to offshore-activities the use of tubular steel piles has been increased during the last years.

4. NATIONAL DOCUMENTS

Since the implementation of DIN EN 1997-1:2009-09: ´Eurocode 7: Geotechnical Design - Part 1: General Rules´, pile analysis and design in Germany is governed by

- Section 7 of Eurocode EC 7-1 (DIN EN 1997-1:2009-09), in conjunction with
- DIN 1054:2010-12: Subsoil - Verification of the Safety of Earthworks and Foundations - Supplementary Rules to the German version DIN EN 1997-1, and the
- National Annex to EC 7-1, namely DIN EN 1997-1/NA:2010-12: National Annex - Nationally Determined Parameters - Eurocode 7: Geotechnical Design - Part 1: General Rules.

These three coordinated documents are summarised in the so called German ´Eurocode 7 Handbook, Volume 1´ (DIN 2011b). Only this Handbook makes these documents applicable as the German standard DIN 1054:2010-12 is quite comprehensive and contains many rules specifying the application of EC7-1

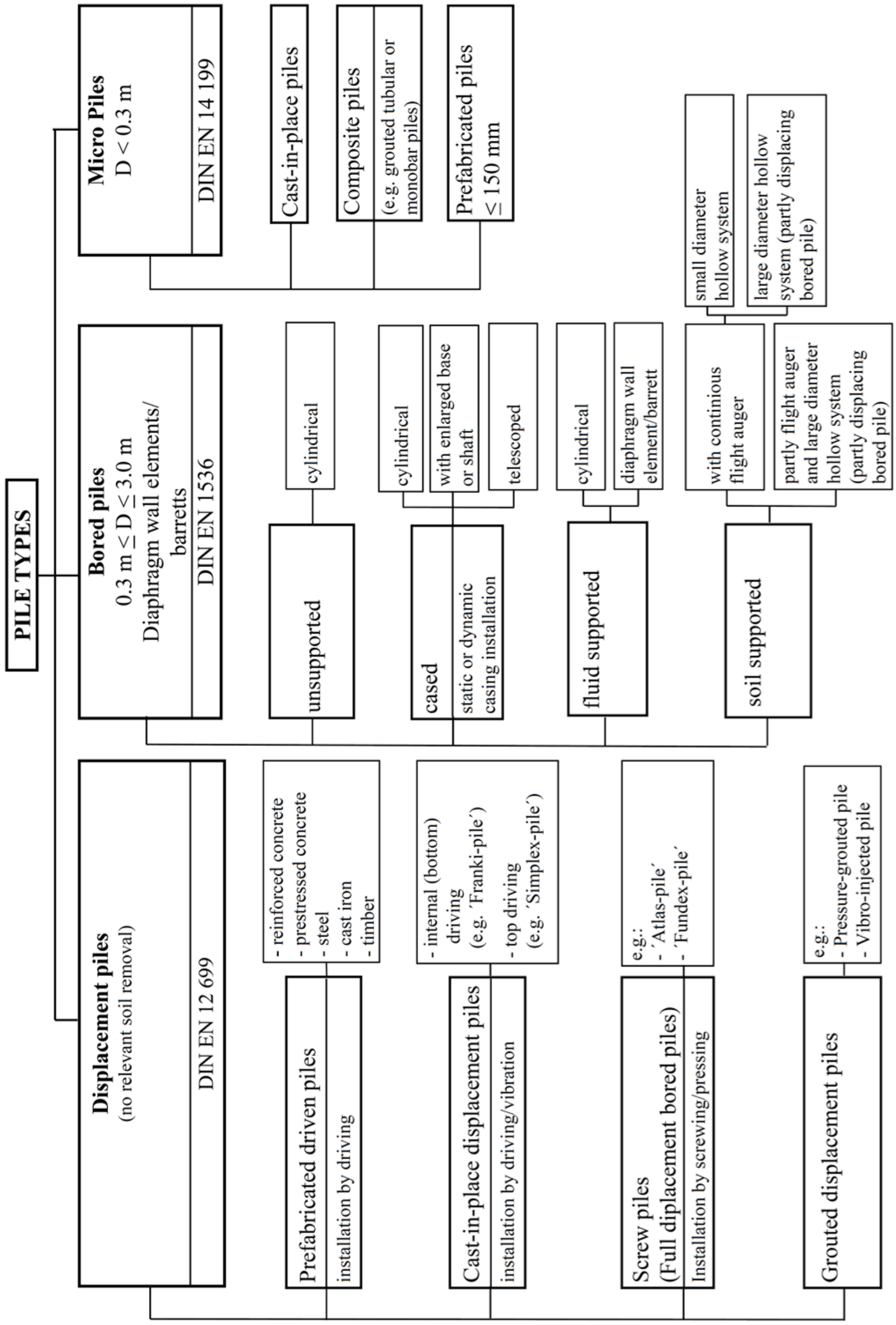


Figure 3: Pile types – Classification and pile types used in Germany

in Germany. The standards EC 7-1 (DIN EN 1997-1:2009-09), National Annex to EC 7-1 (´DIN EN 1997-1/NA:2010-12´) and DIN 1054:2010-12 were implemented as binding building regulations in Germany since July 2012.

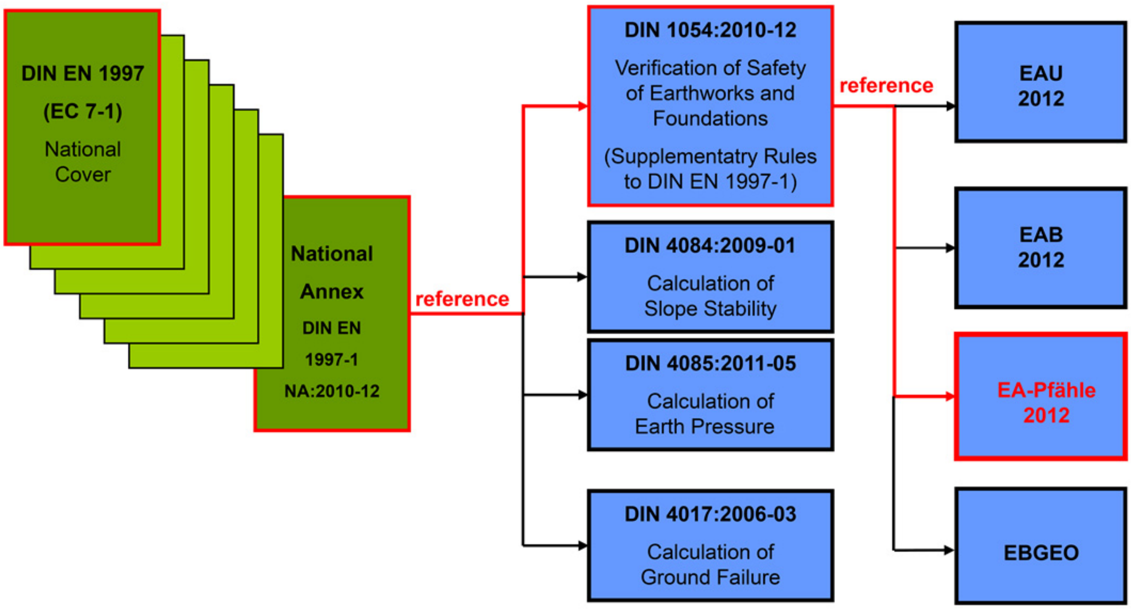


Figure 4: Overview of European and national standards and recommendations for pile design in Germany

Germany has a long tradition of standardization with regard to the execution and design of pile foundations and other pile systems. The German standardisation committee in ´Piles´ (DIN NA 005-05-07 AA) and the Working Group 2.1 of the German Geotechnical Society (DGGT), both hereafter called as the German Piling Committee, have cooperated on these topics for many years, with members sitting in both bodies. To compile the specific experiences and rules for pile design and to supplement the application of the new European standardisation the German Piling Committee has elaborated a summarizing recommendation for pile design and analysis of which the first edition was published in 2007 called ´EA-Pfähle´ (in German: "Empfehlungen des Arbeitskreises Pfähle") (DGGT 2007). The second edition of ´EA-Pfähle´ (DGGT 2012a) finished in 2012 was also published in English (´Recommendation on Piling (EA-Pfähle)´) (DGGT 2012b) (Figure 5). On 498 pages the recommendation provides a quite comprehensive support for all aspects of pile design and analysis covering also specific issues like negative skin friction, group effects, cyclic and dynamic loading etc. as well as recommendations on static and dynamic pile load testing, quality assurance guidelines and methods etc. Table 3 gives an indication on the content of ´EA-Pfähle´ (2nd edition).

These recommendations are now well-established as best practice regulations. As the German standard DIN 1054 refers at various points dealing with pile design to the recommendation ´EA-Pfähle´ this recommendation have also an ´official´ meaning in the German design regulations for piles (Figure 4).

This combination of standards and recommendations reflects also the German basic understanding that standards should focus on the principles of design and safety concepts whereas recommendations might provide more detailed support for engineering practice e.g. with different calculation methods, background information, continuative literature etc.

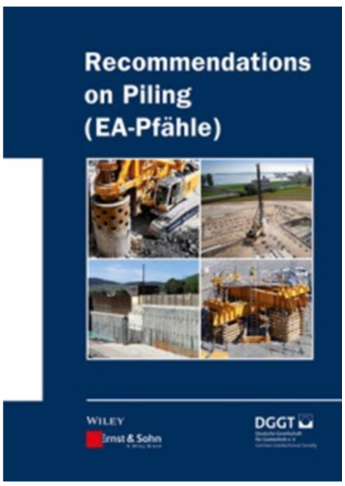


Figure 5: Recommendation on Piling (EA-Pfähle) – Recommendations by the German Piling Committee on design, analysis and excecution of piles (DGGT 2012a,b)

Table 3a: Content of ´Recommendations on Piling (EA-Pfähle)´

Table 3b: Content of ´Recommendations on Piling (EA-Pfähle)´ (continued)

	10.5	Evaluation and Interpretation of Dynamic Load Tests
	10.6	Calibrating Dynamic Pile Load Tests
	10.7	Qualifications of Testing Institutes and Personnel
	10.8	Documentations and Reporting
	10.9	Testing Driving Rig Suitability
11	**Quality Assurance during Piling Execution**	
	11.1	Introduction
	11.2	Bored Piles
	11.3	Displacement Piles
	11.4	Grouted Micro Piles (Composite Piles)
12	**Pile Integrity Testing**	
	12.1	Purposes and Procedures
	12.2	Low Strain Integrity Tests
	12.3	Ultrasonic Integrity Testing
	12.4	Testing Piles by Core Drilling
	12.5	Other Specific Testing Methods
13	**Bearing Capacity and Analyses of Piles under Cyclic, Dynamic and Impact Actions**	
	13.1	Introduction
	13.2	Cyclic, Dynamic and Impact Actions
	13.3	Supplementary Geotechnical Investigations
	13.4	Bearing Behaviour and Resistances under Cyclic Loads
	13.5	Bearing Behaviour and Resistances under Dynamic Loads
	13.6	Bearing Behaviour and Resistances under Impact Loads
	13.7	Stability Analyses of Cyclic Axially Loaded Piles
	13.8	Stability Analyses of Cyclic Laterally Loaded Piles
	13.9	Stability Analyses of Dynamic- or Impact-loaded Piles
Annex A	**Terms, Partial Safety Factors and Principles for Analysis**	
	A.1	Definitions and Notations
	A.2	Partial Safety Factors γ_F and γ_E for actions and effects from EC 7-1 Handbook
	A.3	Partial Safety Factors for Geotechnical Parameters and Resistances from EC 7-1 Handbook
	A.4	Correlation Factors ξ_i for Determining the Characteristic Pile Resistances
	A.5	Procedure for Determining the Resistance of Piles against Buckling Failure in Soil Strata with Low Lateral Support
	A.6	Bonding Stress in Grouted Displacement Piles
Annex B	**Example Calculations for Pile Resistance Analysis and Verifications**	
	B.1	Determining the Axial Pile Resistances from Static Pile Load Tests, Ultimate and Serviceability Limit State Analysis
	B.2	Characteristic Axial Pile Resistances from Dynamic Pile Load Tests
	B.3	Determining the Characteristic Axial Pile Resistances from Empirical Data for a Bored Pile
	B.4	Determining the Characteristic Axial Pile Resistances from Empirical Data for a Prefabricated Driven Pile
	B.5	Determining the Characteristic Axial Pile Resistances from Empirical Data for a Fundex Pile
	B.6	Principle of the Evaluation of Static Pile Load Test Using a Prefabricated Driven Pile and Comparison with Empirical Data
	B.7	Preliminary Design and Analysis of the Ultimate Limit State of Franki Piles Based on Empirical Data and Comparison to a Pile Load Test Result
	B.8	Negative Skin Friction for a Displacement Pile as a Result of Fill
	B.9	Determining the Effect on a Laterally Loaded Pile (Perpendicular to the Pile Axis) and Analysis of Structural Failure
	B.10	Laterally Loaded Piles
	B.11	Pillar Foundation on 9 Piles – Ultimate and Serviceability Limit State Analysis Taking the Group Effect into Consideration
	B.12	Tension Pile Group Analyses on the Ultimate Limit State
	B.13	Laterally Loaded Pile Group: Determining the Distribution of Horizontal Subgrade Moduli
Annex C	**Examples of Dynamic Pile Load Testing and Integrity Testing**	
	C.1	Dynamic Pile Load Test Evaluation: Example using the Direct Method
	C.2	Dynamic Pile Load Test Evaluation: Example using the Extended Method with Complete Modelling
	C.3	Rapid Load Tests Evaluation Example Using the Unloading Point Method
	C.4	Low Strain Integrity Test Case Studies
	C.5	Integrity Tests during Driving and/or High Strain Integrity Tests
	C.6	Example: Ultrasonic Integrity Testing

Table 3c: Content of ´Recommendations on Piling (EA-Pfähle)´ (continued)

In addition, the individual pile systems are governed by the following execution standards:

- DIN EN 1536: Execution of special geotechnical works – Bored piles.
  DIN SPEC 18140: German national supplementary provisions to DIN EN 1536.
- DIN EN 12699: Execution of special geotechnical works – Displacement piles.
  DIN SPEC 18538: German national supplementary provisions to DIN EN 12699.
- DIN EN 14199: Execution of special geotechnical works – Micropiles.
  DIN SPEC 18539: German national supplementary provisions to DIN EN 14199.
- DIN EN 12794: Precast concrete products – Foundation piles.
- DIN EN 1993-5: Design of steel structures - Part 5: Piling.

Because diaphragm wall elements are often employed in the same way as pile foundations, the respective execution standard must also be considered:

- DIN EN 1538: Execution of special geotechnical works - Diaphragm walls

in conjunction with:

- DIN 4126: Stability analysis of diaphragm walls.

5. DESIGN METHOD ACCORDING TO THE PRINCIPLES OF EUROCODE 7

5.1. General principles

In Germany pile foundations are classified as either Geotechnical Category GC 2 or Geotechnical Category GC 3. The German Handbook EC 7-1 (DIN 2011a) classifies pile foundations into the following geotechnical categories:

- Geotechnical Category GC 1:
  in Germany pile foundations shall not normally be assigned to the Geotechnical Category GC 1.

- Geotechnical Category GC 2:
  a) pile foundations for which the pile resistances are determined on the basis of empirical data, e.g. as described in section 5.4 of ´EA-Pfähle´, in cases where ´simple´ ground conditions exist;
  b) common cyclic, dynamic and impact actions;
  c) piles subjected actively to lateral actions with respect to the pile axis, e.g. resulting from structural loads;
  d) piles with negative skin friction.

- Geotechnical Category GC 3:
  a) substantial cyclic, dynamic and impact actions and seismic actions;
  b) raked tension piles with inclinations less than 45° to the horizontal;
  c) tension pile groups;
  d) grouted pile systems (micropiles to DIN EN 14199 and grouted displacement piles to DIN EN 12699) as anchorage elements;
  e) determination of tensile pile resistances;
  f) loading lateral to the pile axis or bending resulting from lateral ground pressure or settlements;
  g) highly utilised piles in conjunction with special serviceability requirements;
  h) piles with shaft and/or base grouting;
  i) piled raft foundations.

For ultimate limit state analysis (ULS), Eurocode EC 7-1 provides three options. With one exception (slope stability considerations), the supplementary rules of DIN 1054 for use in Germany are based on

Design Approach DA 2 of EC 7-1. The partial safety factors are applied to both, effects and resistances. To differentiate this from the other permitted scenario, in which the partial safety factors are not applied to the effects but to the actions, this procedure is designated as DA 2*.

Only for failure of the ground in conjunction with the analysis of the overall stability, i.e. when utilising the shear strength for analysis of the safety against slope failure and global failure including consideration of structural elements, e.g. piles Design Approach DA 3 is applied in Germany.

Therefore the following subordinate limit states of the ultimate limit state (ULS) are relevant for pile design in Germany:

a) EQU: Loss of equilibrium of the structure, regarded as a rigid body, or the ground.

b) UPL: Loss of equilibrium of the structure or the ground due to buoyancy or water pressure.

c) STR: Internal failure, where the strength of the materials governs the resistance.

d) GEO-2: Failure or very large deformation of the ground in conjunction with the calculation of the action effects and the dimensions, i.e. when utilising shear strength for passive earth pressure, for sliding resistance and bearing resistance, and when analysing deep slide surface stability for anchored retaining walls, and for base resistance and skin friction of pile foundations. The GEO-2 limit state calculation follows Analysis Method 2* as outlined in the German Handbook EC 7-1.

e) GEO-3: Failure or very large deformation of the ground in conjunction with the analysis of the overall stability, i.e. when utilising the shear strength for analysis of the safety against slope failure and global failure and, normally, when analysing slope stabilisation measures, including consideration of structural elements, e.g. anchors or piles. The GEO-3 limit state calculation follows Analysis Method 3 as outlined in the German Handbook EC 7-1.

In addition to actions, design situations are also taken into consideration for pile analyses, similar to other structural elements. The previous German loading cases LC 1, LC 2 and LC 3, adopted for use in analysis according to DIN 1054:2005-01, have been converted to design situations for use in analyses after DIN EN 1997-1 (EC 7-1) and DIN 1054:2010-12, and DIN EN 1990 as follows:

- DS-P: persistent (design) situation;
- DS-T: transient (design) situation and
- DS-A: accidental (design) situation.

In addition, there is the seismic design situation BS-E.

The following procedure applies for analysing the 'external' capacity (load transfer to the ground) of an axially loaded, single pile of a pile foundation for the governing design situation in the ultimate limit state GEO-2 applying design approach DA 2*:

a) The characteristic, axial actions F_k, t the pile head are determined as foundation loads of the chosen system. The foundation loads comprise the loads imposed by the structure and, as applicable, negative skin friction, and are separately determined as persistent and transient situations.

b) The design values F_d are determined from the characteristic axial actions F_k on the pile:

$$E_d = E_{G,k} \cdot \gamma_G + E_{Q,rep} \cdot \gamma_Q \tag{1}$$

where γ_G and γ_Q are adopted from the German Handbook EC 7-1, Table A2.1, here documented as Table 4.

c) Adopting the characteristic axial pile resistances the design values of the pile resistances in the ultimate limit state result from:

$$R_{c,d} = R_{c,k}/\gamma_t \quad \text{for compression pile resistance} \tag{2}$$

$$R_{t,d} = R_{t,k}/\gamma_{s,t} \quad \text{for tension pile resistance} \tag{3}$$

where γ_t or $\gamma_{s,t}$, are adopted from the German Handbook EC 7-1, Table A2.3, here documented as Table 6. The partial factors apply equally to both the base and the shaft resistance.

Using the determined axial design actions and resistances, it must be demonstrated that the piles fulfil the limit state conditions for all governing load cases and load combinations, as follows

$$F_{c,d} \le R_{c,d} \text{ or } \sum F_{c,d} \le \sum R_{c,d} \quad \text{for compression pile resistances} \tag{4}$$

$$F_{t,d} \le R_{t,d} \text{ or } \sum F_{t,d} \le \sum R_{t,d} \quad \text{for tension pile resistances} \tag{5}$$

For grouted tension pile systems (grouted micropiles to DIN EN 14199 and grouted displacement piles to DIN EN 12699) in accordance with the EC 7-1 Handbook [44], a model factor η_M shall be taken into consideration for the calculation of the design values and Eq. (3) becomes:

$$R_{t,d} = R_{t,k} / (\gamma_{s,t} \cdot \eta_M) \qquad (6).$$

The model factor is $\eta_M = 1.25$, regardless of the pile inclination. Eq. (6) also applies if, in well-founded, exceptional cases, no pile load test data are available and the pile resistances of grouted pile systems are derived from empirical data.

Table4 documents the partial safety factors γ_G and γ_Q for actions and effects from German Handbook EC 7-1.

Table4: Partial safety factors γ_G and γ_Q for actions and effects from German Handbook EC 7-1, Table A 2.1

Action or effect	Notation	Design situation		
		DS-P	DS-T	DS-A
HYD and UPL: Limit state of failure by hydraulic heave and buoyancy				
Destabilising permanent actions[a)]	$\gamma_{G,dst}$	1.,05	1.05	1,00
Stabilising permanent actions	$\gamma_{G,stb}$	0.95	0.95	0,95
Destabilising variable actions	$\gamma_{Q,dst}$	1.50	1.30	1,00
Stabilising variable actions	$\gamma_{Q,stb}$	0	0	0
Seepage force in favourable subsoil	γ_H	1.35	1.30	1,20
Seepage force in unfavourable subsoil	γ_H	1.80	1.60	1,35
EQU: Limit state of loss of static equilibrium				
Unfavourable permanent actions	$\gamma_{G,dst}$	1.10	1.05	1.00
Favourable permanent actions	$\gamma_{G,stb}$	0.90	0.90	0.95
Unfavourable variable actions	γ_Q	1.50	1.25	1.00
STR and GEO-2: Limit state of failure of the structure, structural elements and the ground				
Effects of permanent actions in general[a)]	γ_G	1.35	1.20	1.10
Effects of favourable permanent actions[b)]	$\gamma_{G,inf}$	1.00	1,00	1.00
Effects of permanent actions from at-rest earth pressure	$\gamma_{G,E0}$	1.20	1,10	1.00
Effects of unfavourable variable actions	γ_Q	1.50	1,30	1.10
Effects of favourable variable actions	γ_Q	0	0	0
GEO-3: Limit state of failure by loss of overall stability				
Permanent actions[a)]	γ_G	1.00	1.00	1.00
Unfavourable variable actions	γ_Q	1.30	1.20	1.00
SLS: Serviceability limit state				
γ_G = 1,00 for permanent actions or effects				
γ_Q = 1,.00 for variable actions or effects				

a) Including permanent and variable water pressure
b) Only in the special case dealt with in Handbook EC 7-1, 7.6.3.1 A (2).

Note 1: In contrast to DIN EN 1990 the partial safety factors γ_G and γ_Q for the effects of permanent and unfavourable, variable actions for the DS-A design situation have been increased from $\gamma_G = \gamma_Q$ = 1,00 to $\gamma_G = \gamma_Q$ = 1,10 in order to retain the proven previous level of safety.

Note 2: The partial safety factors $\gamma_{G,E0}$ are reduced compared to the factors γ_G, because the at-rest earth pressure already decreases to a lower value for minor relaxing movements, and to the considerably smaller active earth pressure in the limit case.

Note 3: DIN EN 1990 prescribes no partial safety factors for the DS-E design situation.

Tables 5 and 6 document the partial safety factors γ_M for geotechnical parameters and γ_Q for geotechnical resistances from German Handbook EC 7-1.

Table 5: Partial safety factors γ_M for geotechnical parameters acccording to German Handbook EC 7-1, Table A 2.2

Soil parameter	Notation	Design situation		
		DS-P	DS-T	DS-A
HYD and UPL: Limit state of failure by hydraulic heave and buoyancy				
Friction coefficient tan φ′ of the drained soil and friction coefficient tan φ_u of the undrained soil	γ_{φ}', $\gamma_{\varphi u}$	1.00	1.00	1.00
Cohesion c′ of the drained soil and shear strength c_u of the undrained soil	γ_c', γ_{cu}	1.00	1.00	1.00
GEO-2: Limit state of failure of the structure, structural elements and the ground				
Friction coefficient tan φ′ of the drained soil and friction coefficient tan φ_u of the undrained soil	γ_{φ}', $\gamma_{\varphi u}$	1.00	1.00	1.00
Cohesion c′ of the drained soil and shear strength c_u of the undrained soil	γ_c', γ_{cu}	1.00	1.00	1.00
GEO-3: Limit state of failure by loss of overall stability				
Friction coefficient tan φ′ of the drained soil and friction coefficient tan φ_u of the undrained soil	γ_{φ}', $\gamma_{\varphi u}$	1.25	1.15	1.10
Cohesion c′ of the drained soil and shear strength c_u of the undrained soil	γ_c', γ_{cu}	1.25	1.15	1.10

[1)] The coefficient γ_M is a generic for the partial safety factors relative to the respective, individual cases.

Table 6: Partial safety factors γ_R for geotechnical resistances acccording to German Handbook EC 7-1, Table A 2.3

Resistance	Notation	Design situation		
		DS-P	DS-T	DS-A
STR and GEO-2: Limit state of failure of the structure, structural elements and the ground				
Pile resistances from static and dynamic pile testing				
– Base resistance	γ_b	1.10	1.10	1.10
– Skin resistance (compression)	γ_s	1.10	1.10	1.10
– Overall resistance (compression)	γ_t	1.10	1.10	1.10
– Skin resistance (tension)	$\gamma_{s,t}$	1.15	1.15	1.15
Pile resistances based on empirical data				
– Compression piles	γ_b, γ_s, γ_t	1.40	1.40	1.40
– Tension piles (in exceptional cases only)	$\gamma_{s,t}$	1.50	1.50	1.50
GEO-3: Limit state of failure by loss of overall stability				
Shear Strength				
– See Table 5				
Pull-out resistances				
– See STR and GEO-2				

[1)] The coefficient γ_R is a generic for the partial safety factors relative to the respective, individual resistance cases.

5.2. Definitions and symbols

Definitions and symbols are used in accordance to EN 1997-1.

5.3. ULS Design based on soil investigation test results

5.3.1. Introduction

The German Handbook EC 7-1 allows axial pile resistances to be derived from empirical data, in addition to determining pile resistances from both static and dynamic pile load tests.

Of the methods described in EC 7-1, 7.6.2.3, under the heading 'Ultimate compressive resistance determined from ground test results', only the method using Eq. (7) below should be adopted in Germany, see German Handbook EC 7-1, Nationally Determined Parameters (NDP) to 7.6.2.3 (5)P and NDP to 7.6.3.3 (4). The method is known in Germany as 'Determining axial pile resistances based on empirical data' and is explained in the following. The 'ground tests' (geotechnical investigations) shall be performed such that it is possible to reliably assign the characteristic empirical data, which have been derived from load tests, for the pile end bearing capacity $q_{b,k}$ and pile skin friction $q_{s,k}$ results. The number of test results or extent of soil investigation does not influence the design resistance hence a correlation factor will not be applied (see below).
To this end the following fundamental equations were used to calculate the characteristic pile resistance by this approach:

$$R_{b,k} = A_b \cdot q_{b,k} \tag{7a}$$

$$R_{s,k} = \sum_i A_{s,i} \cdot q_{s,i,k} \tag{7b}$$

$$R_{c,k} = R_{b,k} + R_{s,k} \tag{7c}$$

However, in principle, the German Handbook EC 7-1 allows all data derived from experience to be used, in addition to the empirical data dealt with below, assuming their utility for the proposed case can be appropriately demonstrated.

5.3.2. Axial compression of a single pile

Evaluation of empirical data for skin friction and base resistance

Empirical pile end bearing capacity and pile skin friction data for the various pile systems are summarised in the German recommendation ´EA-Pfähle´, Section 5.4, as a function of the respective ground conditions. The numerical values listed in the tables refer to the CPT cone resistances q_c in non-cohesive soils and the undrained shear strength c_u of cohesive soils.

The German Handbooks EC 7-1 and EC 7-2 state that soil parameters scatter considerably as a result of the geological formation conditions and history. This applies especially to pile capacity and pile resistances in the ultimate and serviceability limit states, because in addition to the ground-related spatial variability, considerable influences can result from the piling execution. When specifying characteristic soil properties, normally ‘conservative mean values’ are adopted.

The empirical pile base resistance q_b and pile skin friction q_s data ranges summarised in the tables of ´EA-Pfähle´, Section 5.4, are based on numerous and predominantly static pile load test results, which were analysed for this purpose. The evaluation strategies and principles are described in Elborg (1993), Kempfert & Becker (2007), Lüking (2010) and Witzel (2004). As outlined by Kempfert & Becker (2007), empirical pile load test evaluations were made and related to statistical values, differentiated into 10%, 20% and 50% quantiles as input for the tables. This allows the user to also assess the probability and the risk of pile resistance deviations below of the tabled values for a specific project application.

The tables as documented in ´EA-Pfähle´, Section 5.4, contain a range of empirical values for quantiles from 10% to 50% as shown in Figure 6. This means that, as a result of scatter, around 10% of the in-situ pile resistances can be below the lower value in the tables, and around 50% below the upper value (mean value).

Concerning the magnitude of the range of table values it is expressly pointed out that the quantile range in Figure 6 represents an orientation only. The stated boundaries of around 10% to 50% can vary depending on the adopted load test results and the resulting distribution and scatter, because they are based on the adopted load test result population and the boundary conditions of the ground.

Normally and under condition that the site investigation has been carried out in line with the Handbook EC 7-2 the lower table values (minimal values) should be adopted.

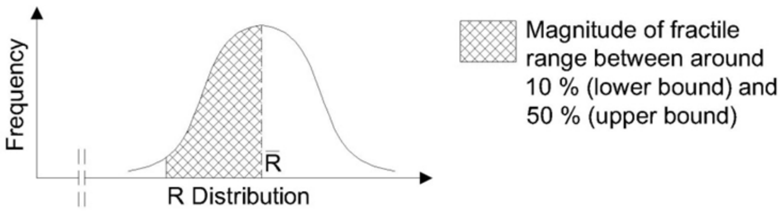


Figure 6: Fractiles for the ranges of pile resistance table data from empirical data, compared to load test results

Pile resistances above the lower (minimal) values, interpolated between the lower and the upper table values, may only be selected for the specific application by the designer if they are expressly confirmed by a geotechnical expert. Local conditions and experience, and the actual situation on the ground must be taken into consideration. The local conditions and experience, and the proposed application, must also be taken into consideration.

If, in terms of pile type and ground conditions, comparable load test results are available, they can be adopted to determine pile resistances as outlined in the Handbook EC 7-1. Comparability must be confirmed by a geotechnical expert or geotechnical designer.

The soil strength range given in the table data includes mean CPT cone resistances q_c = 7.5 to 25 MN/m² for non-cohesive soils and undrained shear strengths $c_{u,k}$ = 100 to 250 kN/m² for cohesive soils related to the end bearing capacity, and $c_{u,k}$ = 60 to 250 kN/m² related to skin friction.

Guidance on comparing the empirical table data to measured data from static and dynamic pile load tests: the tabled characteristic values of pile base resistance and skin friction were derived from pile load test results by statistical application of the quantiles mentioned above to the measured data. Correlation factors ξ were not applied. They are instead already incorporated in the partial factors for empirical values as stipulated in the Handbook EC 7-1, Table A2.3 and Annex A3.2. This is done by a model factor η_E in accordance with the Handbook EC 7-1, NDP to 7.6.2.3 (8) and NDP to 7.6.3.3 (6), which is a fact to be realised when comparing the characteristic table data to characteristic values derived from data measured during pile load tests.

Based on this approach ´EA-Pfähle´, Section 5.4, provides such empirical data for skin friction and base resistance for the following pile types:

- Prefabricated driven piles, i.e.:
  - prefabricated, reinforced concrete and prestressed concrete driven piles where D_{eq} = 0,25 to 0,50 m;
  - closed-ended steel tube piles with diameters up to 800 mm;
  - open-ended steel tube and hollow box piles with diameters between 300 mm and 1 600 mm;
  - steel sections with flange widths between 300 mm and 500 mm, and section heights between 290 mm and 500 mm, and
  - steel box piles.
- Driven Cast-in-place concrete piles
  - Simplex piles
  - Franki piles
- Bored piles (also values for soft rock and rock conditions)
- Partial displacement piles
- Screw piles
- Grouted displacement piles and micropiles
  - pressure-grouted piles
  - vibro-injection piles
  - grouted micropiles
  - tubular grouted piles
  - grouted displacement piles

In the following as an example the approach is documented for bored piles in cohesive and cohesive soils.

Example 1: Prefabricated driven piles

The elements of the characteristic resistance-settlement curve for bored piles are shown in Figure 7 for settlement up to $s_{ult} = s_g$, whereby s_{ult} = settlement in the ultimate limit state and s_g = limit settlement or failure settlement (Normally, s_{ult} and s_g are regarded as equal; s_{ult} formally designates the ultimate limit state analysis method in accordance with Handbook EC 7-1; s_g designates the settlement on pile failure).

The settlement-dependent pile base resistance $R_b(s)$ and the pile shaft resistance $R_s(s)$ are differentiated.

The limit settlement applies for $R_{b,k}$ ($s_{ult} = s_g$):

$$s_g = 0{,}10 \cdot D_b \tag{8}$$

where:
D_b diameter of the pile base in m.

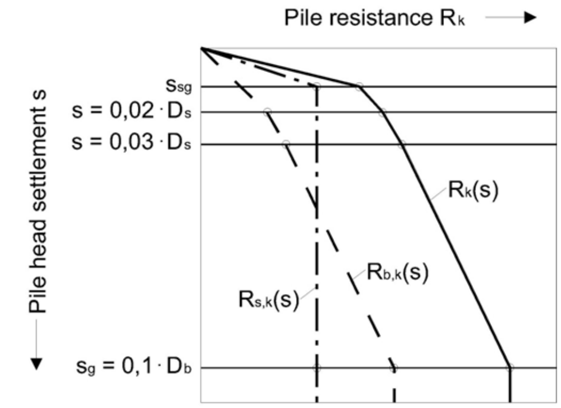


Figure 7: Elements of the characteristic resistance-settlement curve for bored piles

The governing reference variable for settlement in the serviceability limit state is the pile shaft diameter D_s. On piles without an enlarged base the pile diameter D (here: $D = D_s = D_b$) is adopted as the reference variable for settlement at the ultimate limit state. The pile diameter D_b is the governing reference variable for piles with an enlarged base.

The limit settlement applies for the characteristic pile shaft resistance $R_{s,k}(s_{sg})$ in MN in at ultimate limit state:

$$s_{sg}\,[\mathrm{cm}] = 0{,}5 \cdot R_{s,k}\left(s_{sg}\right)[\mathrm{MN}] + 0{,}5\,[\mathrm{cm}] \leq 3\,[\mathrm{cm}] \tag{9}$$

The characteristic axial pile resistance is determined from

$$R_{c,k}(s) = R_{b,k}(s) + R_{s,k}(s) = q_{b,k} \cdot A_b + \sum_i q_{s,k,i} \cdot A_{s,i} \tag{10}$$

where:

A_b	nominal value of the pile base area;
$A_{s,i}$	nominal value of the pile shaft area in stratum i;
$q_{b,k}$	characteristic value of the base resistance, derived from Tables 7 and 9;
$q_{s,k,i}$	characteristic value of the skin friction in stratum i, derived from Tables 8 and 10;
$R_{c,k}(s)$	settlement-dependent, characteristic compressive pile resistance;
$R_{b,k}(s)$	settlement-dependent, characteristic base resistance;
$R_{s,k}(s)$	settlement-dependent, characteristic shaft resistance;
s_{sg}	limit settlement for the settlement-dependent characteristic shaft resistance.

The empirical data for pile base resistance and skin friction given in Tables 7 to 10 apply to bored piles from D_s or D_b = 0,30 to 3.0 m, which embed at least 2.50 m into a load-bearing stratum and depend on:

- the mean cone resistance q_c of the CPT with depth in non-cohesive soil and
- the shear strength of the undrained soil $c_{u,k}$ for cohesive soils.

When specifying the governing mean cone resistance q_c of the CPT or the characteristic undrained shear strength $c_{u,k}$ differentiation shall be made between:

- the zone governing the base resistance from $1 \cdot D_b$ above and $4 \cdot D_b$ below the pile base for pile diameters up to D_b = 0.6 m, and $1 \cdot D_b$ above and $3 \cdot D_b$ below the pile base for diameters greater than D_b = 0.6 m and
- the zone governing the skin friction (mean value for the affected stratum);

If ground stratification has a great influence on the CPT cone resistance or the undrained shear strength, two or more mean pile skin friction zones must be specified separately.

Condition for the application of the values of Tables 7 and 9 are:

– the thickness of the load-bearing layer below the pile base is not less than 3 times the pile base diameter, but at least 1.50 m and
– $q_c \geq 7{,}5$ MN/m² or $c_{u,k} \geq 100$ kN/m² is confirmed in this zone.

Regardless of this, founding the pile bases in zones where $q_c \geq 10$ MN/m² is recommended.

If the above geometrical values are not met, analysis of safety against a punching failure is required. In addition, it must then be verified that the underlying ground does not substantially impair settlement behaviour.

An example for determining the characteristic resistances of bored piles is included in section 8 of this paper.

Table 7: Empirical data ranges for the characteristic base resistance $q_{b,k}$ for bored piles in non-cohesive soils

Relative settlement of the pile head s/D_s or s/D_b	Pile base resistance $q_{b,k}$ [kN/m²]		
	mean CPT cone resistance q_c [MN/m²]		
	7,5	15	25
0,02	550 – 800	1 050 – 1 400	1 750 – 2 300
0,03	700 – 1 050	1 350 – 1 800	2 250 – 2 950
0,10 ($\hat{=} s_g$)	1 600 – 2 300	3 000 – 4 000	4 000 – 5 300
Intermediate values may be linearly interpolated. For bored piles with enlarged base the values shall be reduced to 75 %.			

Table 8: Empirical data ranges for the characteristic skin friction $q_{s,k}$ for bored piles in non-cohesive soils

Mean CPT cone resistance q_c [MN/m²]	Ultimate limit state value $q_{s,k}$ of pile skin friction [kN/m²]
7,5	55 – 80
15	105 – 140
≥ 25	130 – 170
Intermediate values may be linearly interpolated.	

Table 9: Empirical data ranges for the characteristic base resistance $q_{b,k}$ for bored piles in cohesive soils

Relative settlement of the pile head s/D_s or s/D_b	Pile base resistance $q_{b,k}$ [kN/m²]		
	Shear strength $c_{u,k}$ of the undrained soil [kN/m²]		
	100	150	250
0,02	350 – 450	600 – 750	950 – 1 200
0,03	450 – 550	700 – 900	1 200 – 1 450
0,10 ($\hat{=} s_g$)	800 – 1 000	1 200 – 1,500	1 600 – 2 000
Intermediate values may be linearly interpolated. For bored piles with a flared base the values are reduced to 75 %.			

Table 10: Empirical data ranges for the characteristic skin friction $q_{s,k}$ for bored piles in cohesive soils

Shear strength $c_{u,k}$ of the undrained soil [kN/m²]	Ultimate limit state value $q_{s,k}$ of pile skin friction [kN/m²]
60	30 – 40
150	50 – 65
≥ 250	65 – 85
Intermediate values may be linearly interpolated.	

5.3.3. *Axial tension of a single pile*

According to Handbook EC 7-1 pile load tests should always be performed when dealing with tension piles. Handbook EC 7-1 allows the estimation of tension pile resistances from empirical data in exceptional cases only.

If, in well-founded, exceptional cases, tension piles resistances based on empirical data are adopted, the derived characteristic skin friction values must be confirmed by a geotechnical expert or geotechnical designer for the specific situation. In this case it shall also be checked if the empirical data given in the tables of ´EA-Pfähle´, Abs. 5.4, should be further and considerably reduced for deriving tension pile resistances, e.g. by applying appropriate calibration factors.

When determining a characteristic resistance-heave curve based on empirical data, the limit heave $s_{sg,t}$ may be approximately determined using:

$$s_{gt} = 1.,30 \cdot s_{sg} \tag{11}$$

where s_{sg} is adopted after Eq. (9), or accordingly for other pile types.

5.3.4. *Lateral loading of a single pile*

For laterally loaded piles the German recommendations like the ´EA-Pfähle´ provides little additional information.

In engineering practice often simplified models were used where the soil resistances lateral to the pile axis is simulated as a subgrade reaction moduli, in particular for long, flexible piles. The subgrade reaction modulus is often assumed to be a constante values determined from the relationship: $k_{s,k} = E_{s,k}/D_s$ as outlined in the German Handbook EC 7-1 for simple cases where only the effects within the pile, e.g. bending moments should be caculated. Not regulated in the Handbook EC 7-1 is, however, the effect of the shape of progression with depth of the modulus of subgrade reaction which can be decisive, even when using the simplified approach.

More advanced calculation models like the p-y curves are applied in national practice especially for more demanding structures like pile foundations for bridges, wind turbines etc., but are not coverey by national standards or ´EA-Pfähle´ so far.

It is mentioned by ´EA-Pfähle´ that for more accurate analysis, the modulus of subgrade reaction should be determined from horizontal pile load tests as a function of deflections and be adopted corresponding to the deflections resulting from the effects on the complete structure.

5.3.5. *Specific issues*

General

The German recommendation ´EA-Pfähle´ covers many specific issues like

- negative skin friction,
- lateral thrust on piles due to horizontal soil movements,
- pile group behaviour for axial and lateral loading
- behaviour and design concept for piles due to cyclic, dynamic and impact actions
- resistance of piles against buckling failure in soft soil
- and others

providing additional guidance to engineering practice in terms of calculation methods, design concepts etc..

No specific rules are provided for seismic design of piles.

In the following exemplarily the recommendations for negative skin friction (downdrag) are summarized.

Negative skin friction

According to ´EA-Pfähle´, Section 4.4, Negative skin friction in piles has to be regarded as a permanent action F_n, originating from relative axial movement between the ground and the pile, when the ground settles more than the pile. This relative movement is generally initiated by settlement of a soft stratum, which can be the result e.g. of surcharges, consolidation processes or groundwater table fluctuations. The self-weight of the settling stratum and of the overlying strata act on the pile via skin friction. This skin friction acts counter to the skin friction from the pile settlement and is therefore known as negative skin friction.

The pile continues to settle until the actions from negative skin friction τ_n, together with the actions imposed on the pile by the superstructure, and the pile resistances resulting from the pile end bearing capacity and supporting skin friction q_s, are in equilibrium. Figure 8 shows these relationships for two cases:

- For small effects F_a resulting from structural loads and thus small pile settlement s_a, and large negative skin friction effects F_n, the depth influenced by τ_n is great.
- Reciprocally, a large effect F_b leads to large pile settlements and thus to activation of positive skin friction q_s as a result of relative movement between the ground and the pile.

The boundary between numerically positive and negative skin friction is known as the neutral point, see Figures 8.

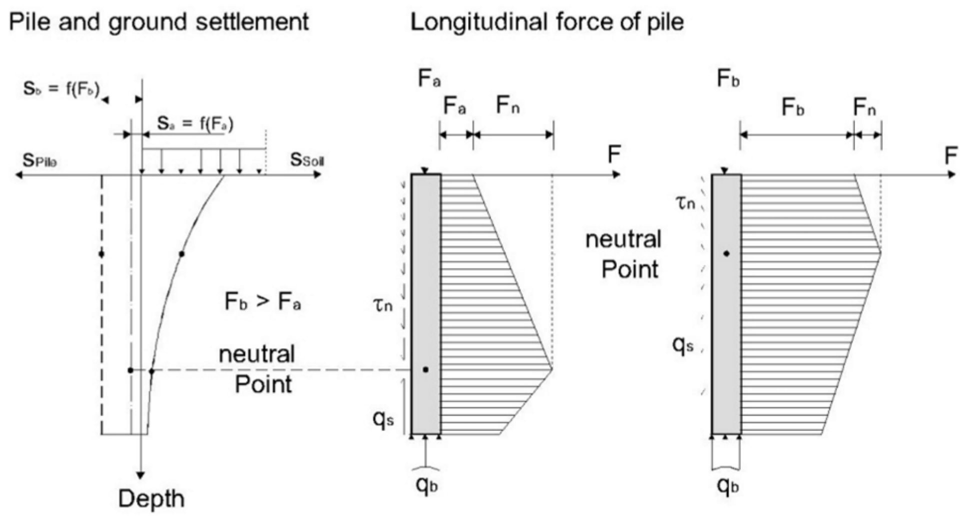


Figure 8 Qualitative relationships between pile resistances and effects from structural loads, and negative skin friction in homogeneous ground, and the definition of the neutral point.

In the partial safety factor approach the negative skin friction is generally defined as a permanent action on pile foundations, leading to an additional effect on the piles.

An appropriate estimate of the pile's negative skin friction $\tau_{n,k}$ requires information on:

- pile settlements with depth;
- soil strata settlements with depth;
- the resulting relative movements and;
- any mobilisation functions of $\tau_{n,k}$ and $q_{s,k}$.

Two principle approaches for deriving the characteristic negative skin friction $\tau_{n,k}$ are given in the literature dealing with negative skin friction:

- Using total stresses for cohesive soils

$$\tau_{n,k} = \alpha \cdot c_{u,k} \tag{12}$$

where:

α factor for specifying the value of the characteristic negative skin friction for cohesive soils;
$c_{u,k}$ characteristic value of the shear strength of the undrained soil.

Depending on the soil type and pile type the factor α generally ranges between 0.15 and 1.60, whereby $\alpha = 1$ is often adopted in approximation, which is generally recommended for cohesive soils.

- Using effective stresses for non-cohesive and cohesive soils:

$$\tau_{n,k} = K_0 \cdot \tan\varphi'_k \cdot \sigma'_v = \beta \cdot \sigma'_v \quad (13)$$

where:
σ'_v effective vertical stress;
K_0 coefficient of at-rest earth pressure;
φ'_k characteristic value of the friction angle;
β factor for specifying the value of the characteristic negative skin friction for non-cohesive and cohesive soils.

According to the literature the factor β generally ranges between 0.1 and 1.0, depending on soil type. For non-cohesive soils $\beta = 0.25$ to 0.30 is often used.

To determine the depth of the neutral point, and thus the value of the characteristic action $F_{n,k}$(SLS) in the serviceability limit state it is recommended to normally determine the deformations of the ground surrounding the pile for the final situation and using characteristic values, i.e. taking consolidation and creep deformations s_n into consideration,.

Comparing the deformations resulting from pile settlement s and the settlement of the surrounding soft stratum s_n gives the location of the neutral point.

In order to determine the neutral point, and thus the characteristic action $F_{n,k}$(ULS) in the ultimate limit state (“external” pile capacity), it is recommended to specify the pile settlement $s_g = s_{ult}$ in the ultimate limit state in accordance with Section 5, depending on the method selected to determine pile capacity.

Comparing the deformations for $s_g = s_{ult}$ with the settlement of the surrounding soft strata s_n gives the location of the neutral point for the ultimate limit state, which can be located at a different depth than in the serviceability limit state.

When allocating the action of negative skin friction to a load case it is recommended to allocate it to the persistent design situation DS-P, if the negative skin friction continues to exist throughout the pile's functional lifetime and the deformed soft stratum remains as a permanent action around the pile, even after settlements of the soft stratum have ceased.

To proof the “External” pile capacity the following to situation have to be checked:

a) Serviceability limit state (SLS): the characteristic action $F_{n,k}$(SLS) and the location of the neutral point have to be calculated by the deformation behaviour associated with the pile settlement s and the settlements in the soft stratum s_n. The design value of the effects is:

$$F_d = F_k = F_{G,k} + F_{n,k}(SLS) + F_{Q,rep} \quad (14)$$

b) Ultimate limit state (ULS): the characteristic action $F_{n,k}$(ULS) and the location of the neutral point have to be calculated by comparing the deformations associated with the pile settlement $s_g = s_{ult}$ and the settlements in the soft stratum s_n. The location of the neutral point is normally higher than in the serviceability limit state, because the imaginary pile settlement s_{ult} is greater than s(SLS) (except for piles on rock, for example). The design value of the effects is:

$$F_d = (F_{G,k} + F_{n,k}(ULS)) \cdot \gamma_G + F_{Q,rep} \cdot \gamma_Q \quad (15)$$

To proof the “Internal” pile capacity (structural analysis) an analysis is usually done for the ultimate limit state (ULS) adopting the actions resulting from negative skin friction in the serviceability limit state $F_{n,k}$(SLS) for pile settlement s(SLS).

5.3.6. Problems not covered by National Annexes and future developments

The German rules for pile design are subject of continuously revision and extension. Subjects of further development are presently e.g.

- improved recommendation for laterally loaded piles,
- extension of pile design with empirical data for CPT-values $q_c > 25$ MN/m²,
- seismic design of pile foundations,
- improved calculation models for pile groups,
- simplified calculation models for piled rafts.

5.4. SLS design

5.4.1. Axially loaded piles

If an appropriate examination reveals that the deformations of the pile foundation are relevant to the structure, an analysis of the serviceability limit state (SLS) must be performed for the governing design situations. Serviceability is demonstrated if the following condition is met:

$$F_d(SLS) = F_k \leq R_d(SLS) = R_k(SLS) \tag{16}$$

Partial factors of γ = 1.0 are normally adopted for actions and resistances.
Analysis may also be performed using a value for the allowable settlements (´allow. s_k´) as provided by the structural designer, assuming characteristic effects on the pile foundation in the serviceability limit state, as follows:

$$s_k \leq \text{allow. } s_k \tag{17}$$

Where pile systems only show minor settlements under service loads, the serviceability limit state analysis can be covered in the analysis of the ultimate limit state according to the Note for clause 7.6.4.1(2) of EC 7-1.

According to ´EA-Pfähle´ (Section 6.4.1) it is assumed in a first step that single pile performance is prevalent for the pile foundation structure as a whole. Regardless of this, differential settlements Δs_k between the piles of a piled foundation structure can occur, even when single pile behaviour is anticipated. Such effects can result e.g. of heterogeneous ground conditions and/or influences from pile execution. After Figure 9 differentiation is to be made between anticipated

- minor differential settlements and
- substantial differential settlements within the pile group.

If only minor differential settlements are anticipated between individual piles in a structure, then the characteristic pile resistance shall be derived from an assessment of pile load test results or from a calculation with the empirical data of ´EA-Pfähle´ using a specified, allowable characteristic settlement s_k as shown in Figure 9 a). For the consideration of the ground-foundation-structure interaction, spring constants may be adopted for the piles, which are derived from the secant on the characteristic resistance-settlement or -heave curve, or from empirical data.

If substantial differential settlements are anticipated between the individual piles of a structure additionally possible upper $s_{k,max}$ and lower bounds $s_{k,min}$ of the settlements s_k after Figure 9 b) shall be determined in the range of the resulting pile resistance R(SLS), adopting the following equation:

$$\Delta s_k = \kappa \cdot s_k \tag{18}$$

The factor κ depends on pile installation, ground stratification and the locations of the piles within the foundation structure, and should be specified in agreement with the geotechnical expert and recomendations by Kempfert 2009. It has to be checked whether as a result of these possible differential settlements between the piles or within a pile group for the characteristic pile resistance R(SLS), an ultimate limit state (ULS) or serviceability limit state (SLS) might result as a consequence of imposed effects in the pile head slab or the superstructure.

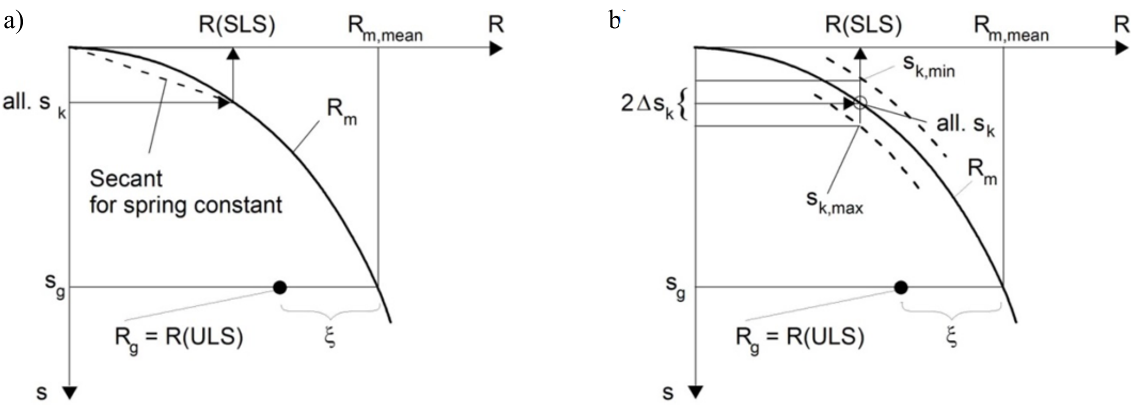


Figure 9: Possible method for derivation of characteristic resistances of isolated piles R(SLS) in the serviceability limit state from test data and data from resistance-settlement curves a) for anticipated minor differential settlements between isolated piles; b) for anticipated substantial differential settlements between isolated piles

For the evaluation of the settlments of axially loaded compression pile groups an approach that bases on nomogramms derived from numerical simulation of bored pile groups is provided by ´EA-Pfähle´ (section 8.2) which enables to calculate the settleemnt the average settlement of the pile group by:

$$s_G = s_E \cdot G_s \qquad (19)$$

where:
s_G mean settlement of a pile group;
s_E settlement of a comparable single pile;
G_s settlement-related group factor for the mean settlement of a pile group.

The settlement-related group factor G_s for determining the mean settlement of a pile group subject to a centrallyacting, vertical action is given by:

$$G_s = S_1 \cdot S_2 \cdot S_3 \qquad (20)$$

where:

S_1 factor concerning the influence of the soil type and the group geometry (pile length L, pile embedment depth in load-bearing ground d, pile centre distances a,
S_2 group size influence factor,
S_3 pile type influence factor.

For the factors nomogrmms can be found in ´EA-Pfähle´ (section 8.2).

The obeservational method might be applied additionally to measure e.g. the settlement behaviour in the easiest case or even the pile loads distribution within a pile group or a piled raft in a more complex case. In any case the observational method would be used in Germany to validate the analytical or numerical predicition but not for SLS design of axially loaded piles.

5.4.2. Laterally loaded piles

For laterally loaded piles no specific rules or recommendation are provided by the German standards. ´EA-Pfähle´ states that analytical approaches like using a lateral subgrade modulus / p-y-curves might be not accurate enough to predict the lateral displacements reliably. Therefore it is recommended that for more accurate analysis, the modulus of subgrade reaction should be determined from horizontal pile load tests as a function of deflections and be adopted corresponding to the deflections resulting from the effects on the complete structure.

5.5. Design based on load tests

The German ´Handbook Eurocode 7, Part 1´ requests that generally the load-settlement behaviour of axially loaded piles respectively the lateral load-displacement behaviour of laterally loaded piles should be evaluated on the results of static pile load tests.

For tension piles the execution of static axial pile load tests is considered to be mandatory, whereas a calculation should be the exception.

For micropiles under compression the execution of static pile load tests on at least 3 % of all executed piles, at the least at n = 2 piles is requested according to DIN EN 14199.

In consequence the pile design based on load tests still plays an important role for pile design in Germany. Beside static pile load test dynamic tests are executed quite frequently for onshore and offshore conditions. Generally the reliability of dynamic load tests is considered to be lower than for static pile load tests and therefore higher correlation factors have to be applied for dynamic pile load tests. Results of a round robin test on bored piles in sandy soil near Berlin (Baeßler et al. 2012; Herten et al. 2013) also indicated that for cast-in-place bored and driven piles a quite large scattering of the analysis of the results of dynamic pile load tests may occur, which makes dynamic pile load tests predominately applicable for prefabricated driven piles (made of concrete or steel) only.

5.5.1. Static pile load tests

Recommendations for the planning and execution of static pile tests

The recommendation ´EA-Pfähle´ (Section 9) contains detailed recommendations on the planning and execution of static pile load tests covering all details like:

- Installation of test piles,

- Test planning including number of piles, test load, instrumentation etc.,
- Loading system including reaction system, hydraulic jacks etc.,
- Instrumentation and monitoring,
- Testing procedure,
- Evaluation of test data,
- Documentation and reports.

Such recommendations are provided by ´EA-Pfähle´

- for static axial pile load tests,
- for static lateral pile load tests,
- for static pile load tests on micropiles (composite piles).

If the ultimate bearing resistance is not obvious from the form of the measured resistance-settlement curve for compression piles, then

$$s_g = s_{ult} = 0.10 \cdot D_b \tag{21}$$

can, in approximation and for all pile systems, be adopted for the limit settlement s_g or s_{ult}, where s_{ult} = settlement in the ultimate limit state and s_g = limit settlement or failure settlement.

Correlation factors from static pile tests

The characteristic pile resistances $R_{c,k}$ (compression) and $R_{t,k}$ (tension) are determined from the data measured in static pile tests by dividing by the correlation factors ξ_i given in Handbook EC 7-1 , Table A7.1. The German nationally determined parameters ξ_1 and ξ_2 are very close to the values proposed by EC 7-1, Table A.9, but are not identical.

For the ultimate compressive resistance the following equation must be fulfilled for structures incapable of redistributing loads from "flexible" to "stiff" compression piles.

$$R_{c;k} = MIN\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\} \tag{22}$$

In the German approach the correlation factors ξ_1 and ξ_2 depend according to Table 11 (only) on the number of pile tests performed, and are applied to the mean $(R_{c,m})_{mean}$ or the smallest value $(R_{c,m})_{min}$ of $R_{c,m}$.

If structures possess sufficient stiffness to redistribute loads from "weaker" to "stiffer" compression piles, the numerical values of ξ_1 and ξ_2 may be divided by 1.1, assuming that ξ_1 never becomes smaller than 1.0. There is no clear criterion for a "sufficient stiff" structure. There is the same approach but no such differentiation between piles of "weak" and "stiff" structures for tension piles.

Table 11: Correlation factors ξ_i for deriving characteristic pile resistances from static pile testing on compression and tension piles

n	1	2	3	4	≥ 5
ξ_1	1.35	1.25	1.15	1.05	1.00
ξ_2	1.35	1.15	1.00	1.00	1.00
n is the number of tested piles.					

5.5.2. Dynamic pile load tests

General comments on dynamic pile testing

In accordance with the Handbook EC 7-1 under certain circumstances the compressive pile resistances may also be derived from dynamic pile load tests. Calibration against static pile tests is required to ensure that the damping factors for determining the dynamic component of the total resistance using direct methods are correctly selected. Use of the extended method with complete modelling is considered to represent current best practice for determining pile resistances. This method shall preferentially be adopted.

If predominantly non-cohesive soils are prevalent in the load-bearing strata in which the piles are embedded, and the testing institute can prove that extensive experience from, dynamic pile tests is available for the region, the characteristic pile resistances Rk, may be derived even if no static pile load tests have been carried out on the site. In this case R_k may be deducted from the tested or measured data from dynamic pile tests Rm with the correlation factors $\xi_{5,6}$ and $\Delta\xi$ for the case 'calibration of dynamic pile load tests with static pile load tests carried out on similar construction projects'. The reason for this is the number of comparative dynamic and static pile load tests available for non-cohesive ground conditions in the whole of northern Germany and, resultantly, the availability of reliable calibrations and experience. These provisions only apply to prefabricated driven piles

For dynamic load tests on piles in cohesive soils, in accordance with the Handbook EC 7-1, ´EA-Pfähle`asks for the following procedure to be followed:

a) For piles in soils sensitive to creep and in unsaturated cohesive soils, the results of dynamic pile load tests shall always be calibrated against static pile load tests from the same site.
b) In saturated cohesive soils excessive porewater pressures can increase the capacities measured in dynamic pile load tests. Dynamic pile load tests may therefore not be adopted to determine the characteristic pile capacity if the pile base is situated in or the governing skin friction is mobilised in such soils.
c) In deviation to the provisions in a) and b), dynamic pile load tests may be carried out in cohesive soils only if reliable, regional, empirical data are available and their applicability is expressly confirmed by a geotechnical expert for the respective case.

When deriving characteristic pile resistances from dynamic pile load tests the following steps should normally be followed:

a) The testing institute employed for the dynamic pile load testing provides the test results in a test report containing the test or measured data $R_{c,m,i} = R_{c,stat}$ differentiated into $(R_{c,m})_{mean}$ and $(R_{c,m})_{min}$.
b) A plausibility check of the measured values is necessary in order to minimise uncertainties connected with the dynamic load testing procedure. To this end, the geotechnical expert or geotechnical designer shall compare the mean dynamic pile load test values $(R_{c,m})_{mean}$ to the upper bounds of the calculated empirical data for the pile system or with local experience. After carrying out the plausibility check the geotechnical expert or the geotechnical designer shall confirm or modify the mean value $(R_{c,m})_{mean}$.
c) The geotechnical expert or geotechnical designer then converts the test data or the modified values in accordance with a) and b) into characteristic pile resistances $R_{c,k}$ on a project-specific basis for use in stability analyses. In this process, he shall take the stipulations in the Handbook EC 7-1 and Annex 4.2 into consideration.
d) If the plausibility check after c) reveals significant differences and considerable assessment uncertainties, it is recommended that the test data (raw data) be independently examined and evaluated again, e.g. by means of a separate evaluation employing complete modelling. The independent examination and evaluation is of particular importance for conditions falling into Geotechnical Category GC 3.

The procedure for deriving characteristic pile resistances from dynamic pile load tests is in principle shown in Figure 10.

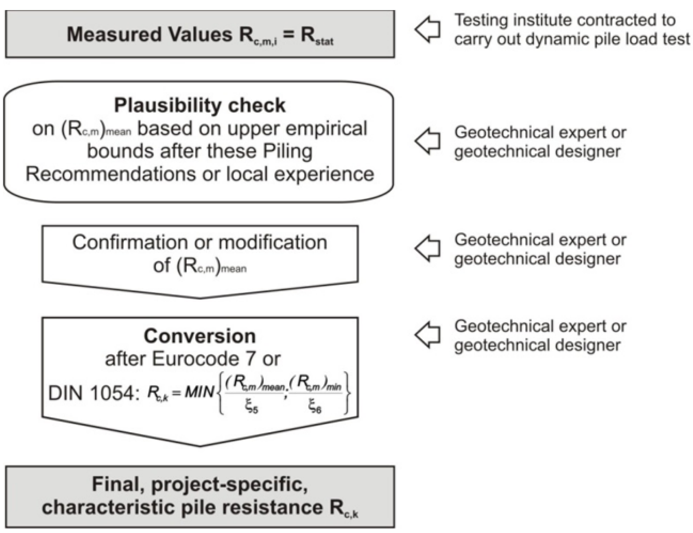


Figure 10: Steps to determine characteristic pile resistances from measured or test data of dynamic pile load tests

Table 12 Base values $\xi_{0,i}$ with corresponding increase factors and model factors for correlation factors ξ_5 and ξ_6 used to derive characteristic values from impact or dynamic pile tests

$\xi_{0,i}$ for n =	2	5	10	15	≥ 20
$\xi_{0,5}$	1,60	1,50	1,45	1,42	1,40
$\xi_{0,6}$	1,50	1,35	1,30	1,25	1,25

– n is the number of tested piles;
– Intermediate values of $\xi_{0,5}$ and $\xi_{0,6}$ for n = 2 to 20 may be linearly interpolated;

a) To calculate the correlations factors ξ_i : $\xi_i = \left(\xi_{0,i} + \Delta\xi\right) \cdot \eta_D$, also see Figure 11.

b) For the surcharge value $\Delta\xi$:

→ $\Delta\xi = 0$: for calibrating dynamic evaluation methods with static pile test results on the same site;
→ $\Delta\xi = 0.10$: for calibrating dynamic evaluation methods with static pile test results on a comparable construction project;
→ $\Delta\xi = 0.40$: for calibrating dynamic evaluation methods based on documented or common empirical data for pile resistances. Adoption of a direct method such as the Case or TNO method is not permitted.

c) The following apply to the model factor η_D for consideration of the evaluation method

→ $\eta_D = 1.00$: for direct evaluation methods;
→ $\eta_D = 0.85$: For extended methods with complete modelling.

d) If structures possess sufficient stiffness and strength to redistribute loads from “weaker” to “stiffer” piles, the numerical values of ξ_5 and ξ_6 may be divided by 1.1.

c) The following apply to the model factor η_D for consideration of pile driving formulae:

→ $\eta_D = 1.05$: if the wave equation method is adopted;
→ $\eta_D = 1.10$: if a pile driving formula with measurement of the quasi-elastic pile head movement under driving impact is adopted;
→ $\eta_D = 1.20$: if a pile driving formula without measurement of the quasi-elastic pile head movement under driving impact is adopted.

f) If different piles are used in the foundation, groups of similar piles should be considered separately when selecting the number n of test piles. This also applies to areas of similar ground conditions within the same site.

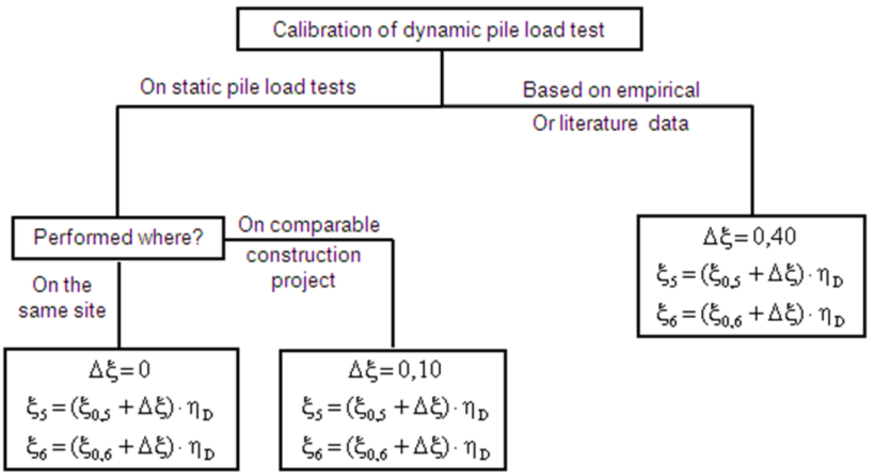


Figure 11: Procedure for deriving the correlation factors ξ_5 and ξ_6 for dynamic pile load tests as a function of the calibration based on Table 12

Recommendations for the planning and execution of dynamic pile tests

The recommendation ´EA-Pfähle´ (Section 10) contains detailed recommendations on the planning and execution of dynamic pile load tests covering all details like:

- Range of Application and General Conditions
- Theoretical Principles
- Description of Testing Methods, Test Planning and Execution
- Evaluation and Interpretation of Dynamic Load Tests
- Calibrating Dynamic Pile Load Tests
- Qualifications of Testing Institutes and Personnel
- Documentations and Reporting
- Testing Driving Rig Suitability

Beside ´conventional´ dynamic tests also Rapid Load Tests are considered.

Correlation factors from dynamic pile tests

The characteristic pile resistance $R_{c,k}$ is determined from the data tested or measured in impact or dynamic pile tests by dividing by the ξ_I correlation factors. The base values of the $\xi_{0,i}$ correlation factors are used to calculate the ξ_i correlation factors, together with the associated ´surcharge factors´ $\Delta\xi$ and model factors η_D from Table 12 (Handbook EC 7-1, Table A7.2) and Figure 11 (Handbook EC 7-1, Figure A7.1).

For the ultimate compressive resistance the following equation must be fulfilled for structures incapable of redistributing loads from “weaker” to “stiffer” compression piles.

$$R_{c;k} = MIN\left\{\frac{(R_{c;m})_{mean}}{\xi_5};\frac{(R_{c;m})_{min}}{\xi_6}\right\} \qquad (23)$$

ξ_5 and ξ_6 are correlation factors which as for the static pile load tests depend (only) on the number of pile tests performed, and are applied to the mean $(R_{c,m})_{mean}$ or the smallest value $(R_{c,m})_{min}$ of $R_{c,m}$.

If structures possess sufficient stiffness to redistribute loads from “weaker” to “stiffer” compression piles, the nume1rical values of ξ_5 and ξ_6 may be divided by 1.1.

5.6. Design based on experience

There are no specific rules for “design based on experience” in Germany although the findings from pile foundations realized and tested under similar geotechnical and geometrical conditions is often considered as additional information for a current piling project.

5.7. Structural safety

The structural design of piles is carried out according to the structural codes for concrete (EC 2) and steel (EC 3) and their national annexes.

The German ´Recommendations on Piling (EA-Pfähle)´ provides additional recommendations on the structural design (section 5.9: ´Internal pile Capacity´) as well as on the execution of piles e.g. concerning the reinforcement and the required concrete cover for bored piles etc.

Additionally the ´EA-Pfähle´ contain recommendations to evaluate the resistance of piles against buckling failure in soil strata with low lateral support providing a simplified approach for buckling analysis and to determine the characteristic resistance against pile buckling (Vogt & Vogt 2013).

6. QUALITY CONTROL, MONITORING AND TESTING PRACTICE

With regard to quality assurance during execution, the requirements of the European execution standards issued for the individual pile types and the German prestandards (DIN SPEC) published to facilitate their use in Germany has to be considered in Germany. Therefore it is common practice to document the piling execution with a record for each individual pile.

´EA-Pfähle´ (Section 11) provides additional recommendations for the quality assurance during pile execution, especially for bored piles, displacement piles and grouted micro piles, and give hints for avoiding common mistakes.

As mentioned in sections 5.5.1 and 5.5.2 of this paper the recommendation ´EA-Pfähle´ contains also detailed information and recommendations for the planning and execution of both static and dynamic pile load tests.

Section 12 of the ´EA-Pfähle´ provides an overview of pile integrity testing methods which serve to control pile quality and geometry after pile installation. In this context

- non-destructive “low strain” tests,
- non-destructive ultrasonic method for testing the concrete in the pile shaft (“cross-hole” method or “single hole” ultrasonic logging) and
- core drilling in the pile with core recovery and core testing and/or video borehole surveys or “single hole” tests

were discussed and recommendation for the execution of such tests are given.

7. PARTICULAR NATIONAL EXPERIENCES AND DATABASES

The particular national experience with a wide spectrum of different pile types and with databases considering a huge number of results from static and dynamic axial pile load tests and their stochastic evaluation mentioned in section 5.3.2.

8. DESIGN EXAMPLES

The most commonly used design methods for single standing, axially loaded piles are illustrated in the following by three examples:

- Example 1: Determining the axial pile resistances from static pile load tests with ultimate and serviceability limit state analyses (section 8.1),
- Example 2: Determining the characteristic axial pile resistances from empirical data for a bored pile (section 8.2),
- Example 3: Determining the Characteristic Axial Pile Resistances from Empirical Data for a Prefabricated Driven Pile (section 8.3).

All three examples are taken from a collection of application and design examples which are added as an annex to the ´Recommendations on Piling (EA-Pfähle)´.

8.1. Example 1: Determining the axial pile resistances from static pile load tests with ultimate and serviceability limit state analyses

8.1.1. Objectives

Figure 12 shows a foundation situation with a pile of diameter D = 1.2 m and a permanent load $F_{G,k}$ = 1,5 MN, in addition to a variable load $F_{Q,rep,k}$ = 1,0 MN. Two static pile load tests were executed, the results of which are included in Figure B1.1b and Table B1.1 as R_{m1} and R_{m2}. The ultimate settlement is defined as $s_g = s_{ult} = 0{,}1 \cdot 120$ cm = 12 cm using Eq. (8). As the static pile load tests were only executed up to a settlement s = 10 cm, the ultimate settlement was extrapolated.

a)

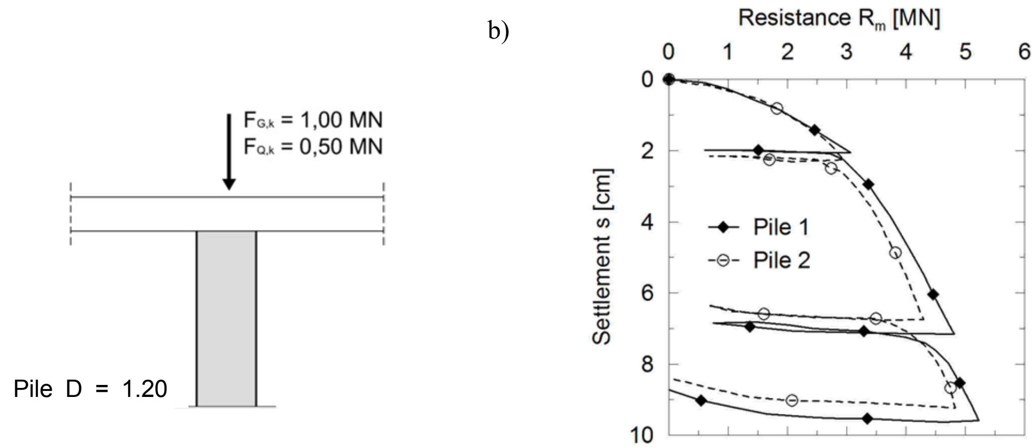


Figure 12 a) System and effect; b) Logged R_m values for both static pile load tests

The analysis comprises the evaluation of the characteristic pile resistances in the ultimate limit state (ULS) and of the characteristic boundary lines in the serviceability limit state (SLS) as well as the external capacity and serviceability for the specified pile load. The evaluation is done for both situations according to section 5.5.1 of this paper, i.e. structures incapable to redistribute loads from ´weaker´ to ´stiffer´ piles als well as structure possesing sufficient stiffness to restribute loads from ´weaker´ to ´stiffer´ piles. An allowable pile settlement allow. s_k = 2,0 cm is specified by the structural design for the serviceability limit state (SLS).
The static pile load tests can be taken from Weiß & Hanack (1983).

8.1.2. Deriving the characteristic pile resistances in the ultimate and serviceability limit states

The characteristic, ultimate pile resistance $R_{c,k}$ is given by the lesser of either the mean value $(R_{c,m})_{mean}$ or the minimum value $(R_{c,m})_{min}$ of the pile load test results using Equation (A4.1) as follows:

$$R_{c;k} = MIN\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\}$$

The correlation factors ξ_1 and ξ_2 depend on the number of static pile load tests performed and are selected after section 5.5. The correlation factors given there apply to ´less stiff´ structures (called "flexible"). If "stiff" structures are used, the correlation factors may be divided by 1.1, assuming that ξ_1 does not become smaller than 1.0.

For the range of small pile settlements, after section 5.4.1, Eq. (17), characteristic boundary lines were derived for the serviceability limit state analysis using the κ values (based on Kempfert 2009). For the present case κ = 0.15 was adopted to relate to the average of the measured values.

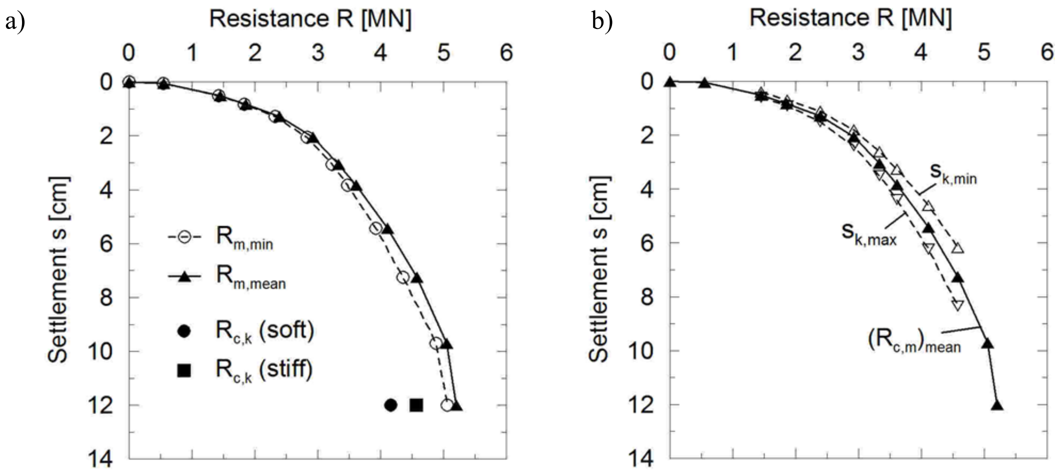


Figure 13 a) Characteristic pile resistances in the ultimate limit state (ULS) for piles of "flexible" and "stiff" structures derived from the $R_{m,i}$ values recorded in static pile load tests; b) Section of the resistance-settlement curves in the serviceability limit state (SLS) and derivation of characteristic boundary curves

Note: The method adopted here for the characteristic boundary lines in the service load range represents only one possible option. Other reasoned procedures are also possible.

Figure 13 and Table 13 show the results for the determination of $R_{c,k}$ (SLS) and $R_{c,k} = R_{c,k}$ (ULS).

Table 13: Results of two static pile load tests and derivation of the characteristic resistance-settlement curves for piles of "flexible" and "stiff" structures

Settlement s [cm]	R_{m1} [MN]	R_{m2} [MN]	$(R_{c,m})_{mean}$ [MN]	$(R_{c,m})_{min}$ [MN]	$\Delta s_k = \kappa \cdot s_k$			
					$s_{k,min}$ [cm]		$s_{k,max}$ [cm]	
0	0	0	0	0	0		0	
0.51	1.483	1.424	1.454	1.424	0.4		0.6	
0.83	1.891	1.831	1.861	1.831	0.7		1.0	
1.28	2.458	2.321	2.390	2.321	1.1		1.5	
2.06	3.015	2.830	2.922	2.830	1.8		2.4	
3.05	3.427	3.230	3.329	3.230	2.6		3.5	
3.83	3.750	3.469	3.610	3.469				
5.42	4.301	3.924	4.112	3.924				
7.26	4.803	4.354	4.579	4.354	Flexible		Stiff	
9.71	5.222	4.881	5.051	4.881	ξ_1	ξ_2	ξ_1	ξ_2
12.0	5.347	5.060	5.204	5.060	1.25	1.15	1.14	1.05

Using the results from Table 13, the characteristic pile resistance $R_{c,k}$ in the serviceability limit state (ULS) can be determined for piles of "flexible" and "stiff" structures in Table 14.

Table 14: Deriving the characteristic pile resistance $R_{c,k}$ in the ultimate limit state for piles of "flexible" and "stiff" structures

Piles of	$\frac{(R_{c;m})_{mean}}{\xi_1}$ [MN]	$\frac{(R_{c;m})_{min}}{\xi_2}$ [MN]	$R_{c,k}$ [MN]
"flexible" structures	4.163	4.400	4.163
"stiff" structures	4.565	4.819	4.565

8.1.3. Bearing capacity analysis

The limit state condition (Section 5.1):

$F_{c,d} \leq R_{c,d}$

must be adhered to for ultimate limit state (ULS) analysis.

a) For single piles acting independently (piles of a "flexible" structure):

$F_{c,d} = F_{G,k} \cdot \gamma_G + F_{Q,rep,k} \cdot \gamma_Q = 1.500 \cdot 1.35 + 1.,000 \cdot 1.50 = 3.525$ MN

$R_{c,d} = R_{c,k}/\gamma_t = 4.163/1,10 = 3.785$ MN

$F_{c,d} = 3.525$ MN $< R_{c,d} = 3.785$ MN

b) For load distribution by means of a rigid capping slab (piles of a "stiff" structure):

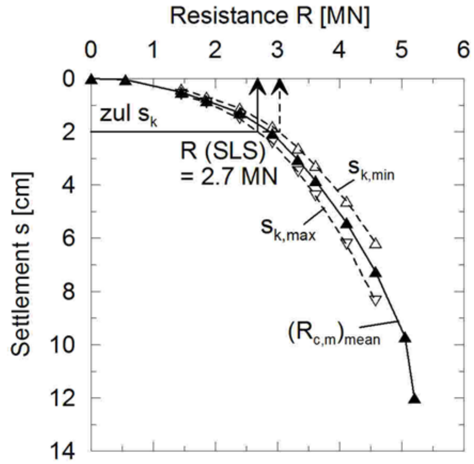
$F_{c,d} = 3.525$ MN

$R_{c,d} = R_{c,k}/\gamma_t = 4.565/1.10 = 4.150$ MN

$F_{c,d} = 3,525$ MN $< R_{c,d} = 4.150$ MN

8.1.4. Serviceability analysis

When determining the pile resistances R(SLS) in the serviceability limit state, differentiation after 5.4 is required whether minor or major (adopted here) differential pile settlements are to be expected. To this end, the characteristic boundary curves in the service load range were derived from the recorded pile load test data in Table 13 and Figure 13 b.

a) 

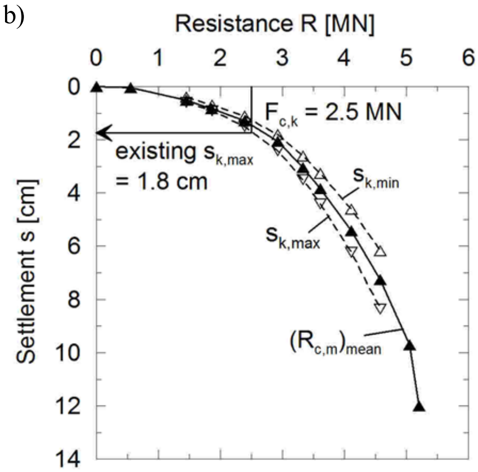
b) 

Figure 14 a) Analysing serviceability via the characteristic pile forces; b) Analysing serviceability via existing and allowable settlements

The specified allowable settlement (e.g. specified in the structural design) in the example is allow. s_k = 2 cm.
Using Figure 14 a, serviceability is demonstrated via pile forces after 5.4.1, Eq. (16) as:

F_d (SLS) = F_k = 2.5 MN < R_d (SLS) = R_k (SLS) = 2.7 MN.

Analysis by comparing settlements in accordance with Figure 14 b after 6.3, Eq. (17) results to:

exist. $s_{k,max}$ = 1.8 cm < allow. s_k = 2.0 cm

8.2. Example 2: Determining the characteristic axial pile resistances from empirical data for a bored pile

8.2.1. Objectives

Figure 15 (example taken from DIN 4014:1990-03) summarises the information on soil type, ground strength and pile geometry necessary for the determination of the axial pile resistance $R_{c,k}(s)$ based on empirical data.

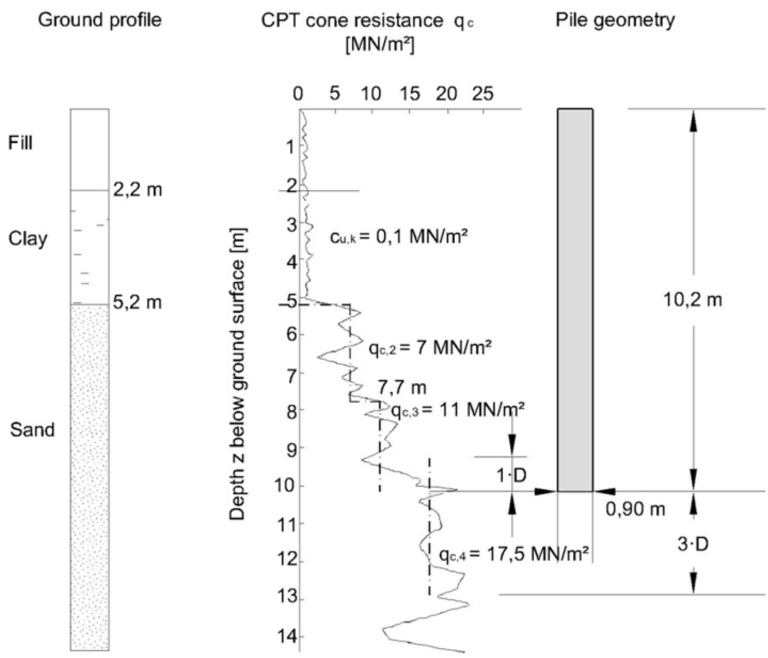


Figure 15: Ground profile, penetration test diagram and dimensions for an example calculation of the resistance-settlement curve; D = 0.9 m, U = 2.83 m, A = 0.64 m²

The characteristic resistance-settlement curve shall be determined using the table data after 5.3.2 (Tables 9 to 10).

8.2.2. Analysis for lower and upper table values

Note: Reference is made to the application principles and limitations in 5.4.3, in particular with regard to the upper table values. In the example presented here both the lower and the upper table values are used as examples (not as a rule).

Determining the pile shaft resistance $R_{s,k}$

The ultimate limit state skin friction values for the sand and the clay are given in Tables 8 and 10 in 5.3.2. By adopting the associated pile skin areas, the ultimate limit state pile shaft resistances $R_{s,k,i}$ are provided in Table 15.
The settlement s_{sg} , in [cm], is calculated as follows, adopting the ultimate limit state pile shaft resistance $R_{s,k}$ in [MN]:

$s_{sg} = 0.50 \cdot R_{s,k} + 0.50.$

Using the figures from the example the pile head settlement is:

$s_{sg} = 0.50 \cdot 1.243 + 0,50 = 1.1$ cm for the lower table values and

$s_{sg} = 0.50 \cdot 1.726 + 0.50 = 1.4$ cm for the upper table values.

Table 15: Ultimate pile shaft resistance for the lower and upper table values

Stratum i [m]	$A_{s,i}$ [m²]	$c_{u,k,i}$ or $q_{c,i}$ [MN/m²]	$q_{s,k,i}$ [MN/m²]	$R_{s,k,i}$ [MN]
2.20 to 5.20	8.48	0.10	0.039 – 0.051	0.331 – 0.432
5.20 to 7.70	7.07	7.00	0.051 – 0.075 a)	0.361 – 0.530
7.70 to 10.20	7.07	11.00	0.078 – 0.108	0.551 – 0.764
a) Extrapolated data				$R_{s,k}$ = 1.243 – 1.726 MN

Determining the pile base resistance $R_{b,k}$

A mean soil strength is adopted in a region from 1 · D (0,9 m) above and 3 · D (3 · D = 2.70 m) below the pile base to determine $R_{b,k}$. For this zone a mean cone resistance $q_{c,m}$ = 17.5 MN/m² is shown in the penetration test diagram in Figure 15.

The pile base capacity can be calculated by adopting the figures from Table 8 in 5.3.2 and taking the previously determined value of $q_{c,m}$ into consideration. Table 16 reproduces the calculated figures.

Table 16: Pile base resistance for the lower and upper table values

Relative settlement s/D	$q_{b,k}$ [MN/m²]	$R_{b,k}(s)$ [MN]
0.02	1.225 – 1.625	0.784 – 1.040
0.03	1.575 – 2.088	1.008 – 1.336
0.10	3.250 – 4.325	2.080 – 2.768

Characteristic resistance-settlement curve

The pile resistances calculated from the pile base and pile shaft resistances are listed in Tables 17 and 18 as a function of the pile head settlement and are given for the lower and upper values. The settlement of the pile head for each value of the pile resistance $R_{c,k}$ is given by the characteristic resistance-settlement curve in Figure 16.

Table 17: Pile resistance as a function of pile head settlement (lower values)

Relative settlement s/D	Pile head settlement [cm]	$R_{s,k}(s)$ [MN]	$R_{b,k}(s)$ [MN]	$R_{c,k}(s)$ [MN]
s_{sg}	1.1	1.243	0.479	1.722
0.02	1.8	1.243	0.784	2.027
0.03	2.7	1.243	1.008	2.251
0.10	9.0	1.243	2.080	3.323

Table 18: Pile resistance as a function of pile head settlement (upper values)

Relative settlement s/D	Pile head settlement [cm]	$R_{s,k}(s)$ [MN]	$R_{b,k}(s)$ [MN]	$R_{c,k}(s)$ [MN]
s_{sg}	1.4	1.726	0.809	2.535
0.02	1.8	1.726	1.040	2.766
0.03	2.7	1.726	1.336	3.062
0.10	9.0	1.726	2.768	4.494

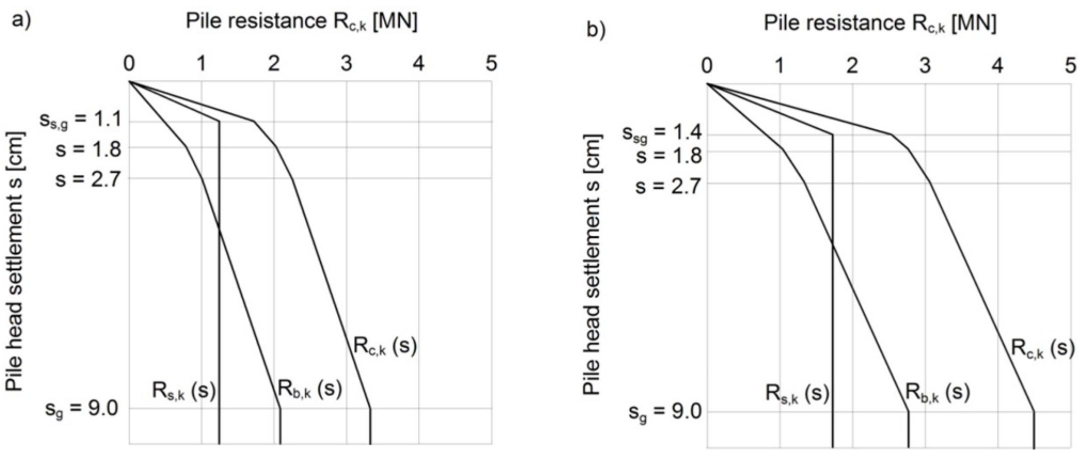


Figure 16: Resistance-settlement curve; a) Lower values, b) Upper values

8.3. Example 3: Determining the characteristic axial pile resistances from empirical data for a prefabricated driven pile

8.3.1. Objective

Figure 17 summarises the information on soil type, ground strength and pile geometry required to determine the axial pile resistance $R_{c,k}(s)$ based on empirical data.

The characteristic resistance-settlement curve shall be determined using the table data after ´EA-Pfähle´, Section 5.3.2 (Tables 5.1 to 5.4).

8.3.2. Characteristic axial pile resistance from empirical data for lower and upper table values

Note: Reference is made to the application principles and limitations in 5.3.2, in particular with regard to the upper table values. In the example presented here both the lower and the upper table values are used as examples (not as a rule).

Determining the pile shaft resistance $R_{s,k}$

The empirical skin friction values in the zones of the load-bearing non-cohesive soil and the weak cohesive soil are given by ´EA-Pfähle´, 5.2 and 5.4 in Section 5.4.4. Together with the corresponding pile skin areas, taking the correlation factor for the skin area from ´EA-Pfähle´, Table 5.5 in Section 5.4.4 into consideration, the pile shaft resistance upon mobilisation of the ultimate limit state state $R_{s,k}$ (s_{sg*}) is given in Table 19 and the pile shaft resistance $R_{s,k}$ (s_g) at failure in Table 20.

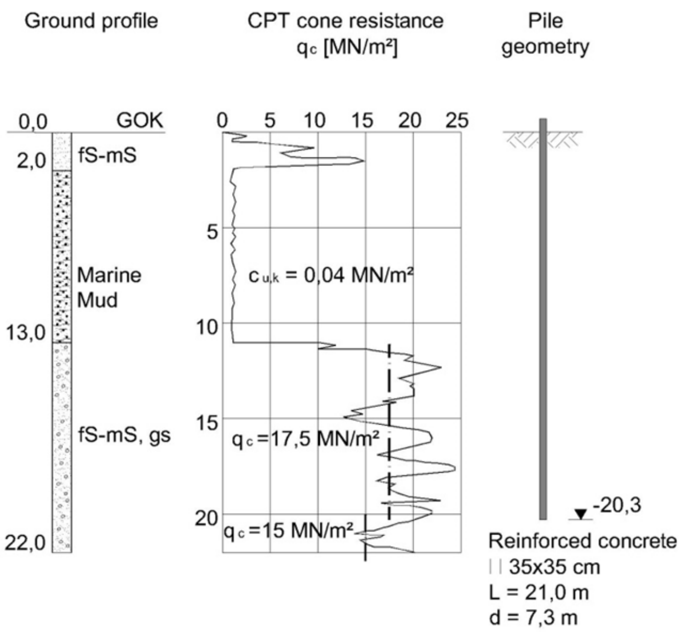


Figure 17 Ground profile, penetration test diagram and dimensions for an example calculation for the resistance-settlement curve

Table 19: Pile shaft resistance upon mobilisation of the ultimate limit state for the lower and upper table values

Stratum i [m]	$A_{s,i}$ [m²]	$q_{c,i}$ [MN/m²]	$q_{s,k,i}$ (s_{sg*}) [MN/m²]	η_s [–]	$R_{s,k,i}$ (s_{sg*}) [MN]
1.0 to 20.3	10.22	17.50	0.070 – 0.098	1.0	0.715 – 1.002
					$R_{s,k}$ (s_{sg*}) = 0.715 – 1.002 MN

Table 20: Shaft resistance at failure for the lower and upper table values

Stratum i [m]	$A_{s,i}$ [m²]	$q_{c,i}$ [MN/m²]	$q_{s,k,i}$ (s_{sg*}) [MN/m²]	η_s [–]	$R_{s,k,i}$ (s_{sg*}) [MN]
13.0 to 20.3	10.22	17.50	0.103 – 0.134	1.0	1.053 – 1.370
					$R_{s,k}$ (s_g) = 1.053 – 1.370 MN

Upon mobilisation of the failure state the settlement in [cm] for the skin friction s_{sg*}, adopting $R_{s,k}(s_{sg*})$ in [MN], is determined using the following equation for the pile shaft resistance $R_{s,k}(s_{sg*})$:

$s_{sg*} = 0.50 \cdot R_{s,k}(s_{sg*})$.

Using the figures from the example the pile head settlement is:

$s_{sg*} = 0.50 \cdot 0.715 = 0.4$ cm for the lower table values and

$s_{sg*} = 0.50 \cdot 1.002 = 0.5$ cm for the upper table values.

Determining the pile base resistance $R_{b,k}$

For determination of $R_{b,k}$ a mean soil strength is adopted from $4 \cdot D_{eq}$ below to $1 \cdot D_{eq}$ above the pile base.
The equivalent pile diameter of a square prefabricated driven pile is determined using:

$D_{eq} = 1.13 \cdot a_s$.

Using the dimensions of the example the equivalent pile diameter is:

$D_{eq} = 1.13 \cdot 0,35 = 0.40$ m.

The nominal value of the square pile base area in this case is:

$A_b = a_s^2 = 0.35^2 = 0.123$ m².

The penetration test diagram in Figure 17 displays a mean characteristic cone resistance along the respective length of:

$$q_{c,m} = \frac{1 \cdot 17{,}5 + 4 \cdot 15{,}0}{5} = 15.5\ MN/m^2$$

Using the figures in ´EA-Pfähle´, Table 5.1 ,and referring to the previously determined value of $q_{c,m}$ and the correlation factor for the pile base area in ´EA-Pfähle´, Table 5.5 (5.4.4), the pile base resistance can be calculated. Table 21 contains the determined numerical values.

Table 21: Pile base resistance for the lower and upper table values

Relative settlement s/D	$q_{b,k}$ [MN/m²]	η_b [–]	$R_{b,k}(s)$ [MN]
0.035	4.025 – 6.550	1.0	0.495 – 0.806
0.100	7.658 – 10.265	1.0	0.942 – 1.263

Characteristic resistance-settlement curve

Tables 22 and 23 contain the pile resistances calculated for the lower and upper values from the pile base and shaft resistances as a function of the pile head settlement. The settlement of the pile head for each value of the pile resistance $R_{c,k}$ is given by the characteristic resistance-settlement curve in Figure 18.

Table 22: Pile resistance as a function of pile head settlement (lower values)

Relative settlement s/D	Pile head settlement [cm]	$R_{s,k}(s)$ [MN]	$R_{b,k}(s)$ [MN]	$R_{c,k}(s)$ [MN]
s_{sg*}	0.4	0.715	0.141	0.856
0.035	1.4	0.809	0.495	1.304
0.100	4.0	1.053	0.942	1.995

Table 23: Pile resistance as a function of pile head settlement (upper values)

Relative settlement s/D	Pile head settlement [cm]	$R_{s,k}(s)$ [MN]	$R_{b,k}(s)$ [MN]	$R_{c,k}(s)$ [MN]
s_{sg*}	0.5	1.002	0.288	1.290
0.035	1.4	1.097	0.806	1.903
0.100	4.0	1.370	1.263	2.633

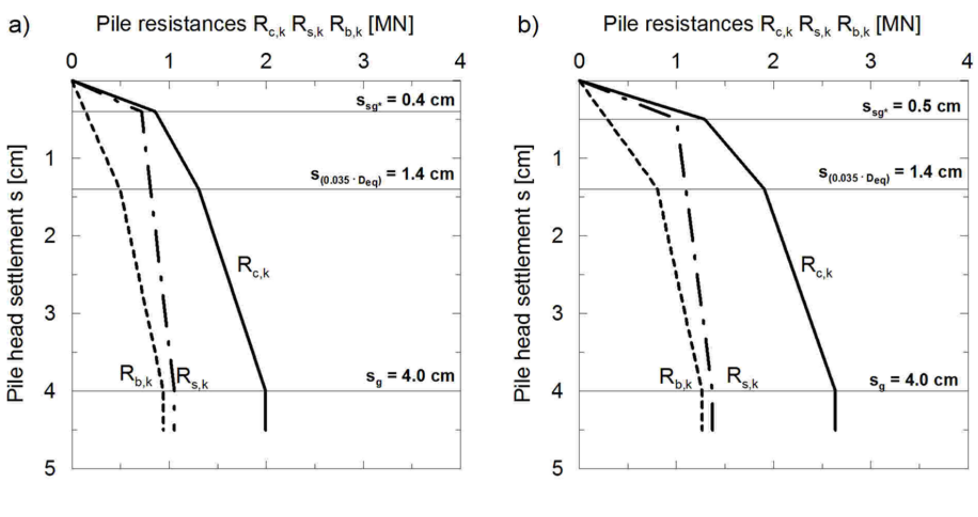


Figure 18: Resistance-settlement curve; a) Lower values, b) Upper values

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