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, well-known 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 Riigen limestone and shales were built. Near the coast sandstones arose, e.g. 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: Particular 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 quarternary and tertiary 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 Freudengerber & 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.

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 \(z_a \geq 4D_b\) 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 exploratory 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
geofrchnical 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')

<table>
<thead>
<tr>
<th>Relative Density D</th>
<th>Density Index $I_D$</th>
<th>Description</th>
<th>Penetration resistances</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$q_c$ [MN/m²]</td>
</tr>
<tr>
<td>$&lt; 0.15$</td>
<td>$&lt; 0.15$</td>
<td>Very loose</td>
<td>$&lt; 5.0$</td>
</tr>
<tr>
<td>$0.15 \ldots 0.30$</td>
<td>$0.15 \ldots 0.35$</td>
<td>Loose</td>
<td>$5.0 \ldots 7.5$</td>
</tr>
<tr>
<td>$0.30 \ldots 0.50$</td>
<td>$0.35 \ldots 0.65$</td>
<td>Medium-dense</td>
<td>$7.5 \ldots 15.0$</td>
</tr>
<tr>
<td>$0.50 \ldots 0.70$</td>
<td>$0.65 \ldots 0.85$</td>
<td>Dense</td>
<td>$15.0 \ldots 25.0$</td>
</tr>
<tr>
<td>$&gt; 0.70$</td>
<td>$&gt; 0.85$</td>
<td>Very dense</td>
<td>$&gt; 25$</td>
</tr>
</tbody>
</table>

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')

<table>
<thead>
<tr>
<th>Soil type</th>
<th>$q_c/N_{30}$ [MN/m²]</th>
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<tbody>
<tr>
<td>Fine to medium sands or slightly silty sand</td>
<td>0.3 to 0.4</td>
</tr>
<tr>
<td>Sand, or sand with some gravel</td>
<td>0.5 to 0.6</td>
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<tr>
<td>Widely-graded sand</td>
<td>0.5 to 1.0</td>
</tr>
<tr>
<td>Sandy gravel or gravel</td>
<td>0.8 to 1.0</td>
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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

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
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.
Germany has a long tradition of standardization with regard to the execution and design of pile foundations and other pile systems. The German standardization 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 4: Overview of European and national standards and recommendations for pile design in Germany

Figure 5: Recommendation on Piling (EA-Pfähle) – Recommendations by the German Piling Committee on design, analysis and execution of piles (DGGT 2012a,b)
Table 3a: Content of ‘Recommendations on Piling (EA-Pfähle)’

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<td>Piles Subjected to Cyclic Lateral Loads</td>
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<td>D.4</td>
<td>Procedure to Determine an Equivalent Single-Stage Load Spectrum</td>
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**Literature**

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.

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 \( F_k \), at 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:

\[
E_d = E_{G,k} \cdot \gamma_G + E_{Q,mp} \cdot \gamma_Q
\]

where \( \gamma_G \) and \( \gamma_Q \) 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:

\[
R_{c,d} = \frac{R_{c,k}}{\gamma_t} \quad \text{for compression pile resistance}
\]

\[
R_{t,d} = \frac{R_{t,k}}{\gamma_s} \quad \text{for tension pile resistance}
\]

where \( \gamma_t \) 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

\[
F_{c,d} \leq R_{c,d} \quad \text{or} \quad \sum F_{c,d} \leq \sum R_{c,d} \quad \text{for compression pile resistances}
\]

\[
F_{t,d} \leq R_{t,d} \quad \text{or} \quad \sum F_{t,d} \leq \sum R_{t,d} \quad \text{for tension pile resistances}
\]
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 \( \eta_M \) shall be taken into consideration for the calculation of the design values and Eq. (3) becomes:

\[
R_{ca} = R_{ct}/(\gamma_{ct} \cdot \eta_M)
\]

(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.

Table 4 documents the partial safety factors \( \gamma_G \) and \( \gamma_Q \) for actions and effects from German Handbook EC 7-1.

<table>
<thead>
<tr>
<th>Table 4: Partial safety factors ( \gamma_G ) and ( \gamma_Q ) for actions and effects from German Handbook EC 7-1, Table A 2.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Action or effect</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>HYD and UPL: Limit state of failure by hydraulic heave and buoyancy</td>
</tr>
<tr>
<td>Destabilising permanent actions (^a)</td>
</tr>
<tr>
<td>Stabilising permanent actions</td>
</tr>
<tr>
<td>Destabilising variable actions</td>
</tr>
<tr>
<td>Stabilising variable actions</td>
</tr>
<tr>
<td>Seepage force in favourable subsoil</td>
</tr>
<tr>
<td>Seepage force in unfavourable subsoil</td>
</tr>
<tr>
<td>EQU: Limit state of loss of static equilibrium</td>
</tr>
<tr>
<td>Unfavourable permanent actions</td>
</tr>
<tr>
<td>Unfavourable variable actions</td>
</tr>
<tr>
<td>STR and GEO-2: Limit state of failure of the structure, structural elements and the ground</td>
</tr>
<tr>
<td>Effects of permanent actions in general (^b)</td>
</tr>
<tr>
<td>Effects of favourable permanent actions (^b)</td>
</tr>
<tr>
<td>Effects of permanent actions from at-rest earth pressure</td>
</tr>
<tr>
<td>Effects of unfavourable variable actions</td>
</tr>
<tr>
<td>Effects of favourable variable actions</td>
</tr>
<tr>
<td>GEO-3: Limit state of failure by loss of overall stability</td>
</tr>
<tr>
<td>Permanent actions (^c)</td>
</tr>
<tr>
<td>Unfavourable variable actions</td>
</tr>
</tbody>
</table>

Note 1: In contrast to DIN EN 1990 the partial safety factors \( \gamma_G \) and \( \gamma_Q \) 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 \( \gamma_{G,EO} \) are reduced compared to the factors \( \gamma_G \), 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 $\gamma_M$ for geotechnical parameters and $\gamma_Q$ for geotechnical resistances from German Handbook EC 7-1.

**Table 5: Partial safety factors $\gamma_M$ for geotechnical parameters according to German Handbook EC 7-1, Table A 2.2**

<table>
<thead>
<tr>
<th>Soil parameter</th>
<th>Notation</th>
<th>Design situation</th>
</tr>
</thead>
<tbody>
<tr>
<td>HYD and UPL: Limit state of failure by hydraulic heave and buoyancy</td>
<td>$\gamma_{\phi}'$, $\gamma_{\phi u}$</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>$\gamma_{c}'$, $\gamma_{cu}$</td>
<td>1.00</td>
</tr>
<tr>
<td>GEO-2: Limit state of failure of the structure, structural elements and the ground</td>
<td>$\gamma_{\phi}'$, $\gamma_{\phi u}$</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>$\gamma_{c}'$, $\gamma_{cu}$</td>
<td>1.00</td>
</tr>
<tr>
<td>GEO-3: Limit state of failure by loss of overall stability</td>
<td>$\gamma_{\phi}'$, $\gamma_{\phi u}$</td>
<td>1.25</td>
</tr>
<tr>
<td></td>
<td>$\gamma_{c}'$, $\gamma_{cu}$</td>
<td>1.25</td>
</tr>
</tbody>
</table>

1) The coefficient $\gamma_M$ is a generic for the partial safety factors relative to the respective, individual cases.

**Table 6: Partial safety factors $\gamma_R$ for geotechnical resistances according to German Handbook EC 7-1, Table A 2.3**

<table>
<thead>
<tr>
<th>Resistance</th>
<th>Notation</th>
<th>Design situation</th>
</tr>
</thead>
<tbody>
<tr>
<td>STR and GEO-2: Limit state of failure of the structure, structural elements and the ground</td>
<td>$\gamma_b$</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>$\gamma_s$</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>$\gamma_t$</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>$\gamma_{st}$</td>
<td>1.15</td>
</tr>
<tr>
<td>Pile resistances based on empirical data</td>
<td>$\gamma_b$, $\gamma_s$, $\gamma_t$</td>
<td>1.40</td>
</tr>
<tr>
<td></td>
<td>$\gamma_{st}$</td>
<td>1.50</td>
</tr>
<tr>
<td>GEO-3: Limit state of failure by loss of overall stability</td>
<td>Shear Strength</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pull-out resistances</td>
<td></td>
</tr>
<tr>
<td></td>
<td>– See STR and GEO-2</td>
<td></td>
</tr>
</tbody>
</table>

1) The coefficient $\gamma_R$ 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:

\[
R_{b,k} = A_b \cdot q_{b,k} \quad (7a)
\]

\[
R_{s,k} = \sum_i A_{s,i} \cdot q_{s,i,k} \quad (7b)
\]

\[
R_{c,k} = R_{h,k} + R_{s,k} \quad (7c)
\]

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.
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 $c_{uk} = 100$ to $250$ kN/m² for cohesive soils related to the end bearing capacity, and $c_{uk} = 60$ 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 $\xi$ 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 $\eta_E$ 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 $1600$ 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 \( R_{b,k} \) and the pile shaft resistance \( R_{s,k} \) are differentiated.

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

\[
s_g = 0.10 \cdot D_b
\]

where:

\( D_b \) = diameter of the pile base in m.

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 \( R_{s,k} \) in MN in at ultimate limit state:

\[
s_{ig} [cm] = 0.5 \cdot R_{s,k} \left( s_{ig} \right) [MN] + 0.5 [cm] \leq 3 [cm]
\]

The characteristic axial pile resistance is determined from

\[
R_{c,k}(s) = R_{h,k}(s) + R_{s,k}(s) = q_{b,k} \cdot A_b + \sum q_{s,k,i} \cdot A_{s,i}
\]

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;
\( R_{h,k}(s) \) = settlement-dependent, characteristic base resistance;
\( R_{s,k}(s) \) = settlement-dependent, characteristic shaft resistance;
\( s_{ig} \) = 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 · \( D_b \) above and 4 · \( D_b \) below the pile base for pile diameters up to \( D_b = 0.6 \) m, and 1 · \( D_b \) above and 3 · \( D_b \) 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/m\(^2\) or \( c_{u,k} \geq 100 \) kN/m\(^2\) is confirmed in this zone.

Regardless of this, founding the pile bases in zones where \( q_c \geq 10 \) MN/m\(^2\) 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**

<table>
<thead>
<tr>
<th>Relative settlement of the pile head ( s/D_b )</th>
<th>Pile base resistance ( q_{b,k} ) [kN/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>mean CPT cone resistance ( q_c ) [MN/m(^2)]</td>
</tr>
<tr>
<td></td>
<td>7.5</td>
</tr>
<tr>
<td>0.02</td>
<td>550 – 800</td>
</tr>
<tr>
<td>0.03</td>
<td>700 – 1 050</td>
</tr>
<tr>
<td>0.10 (( \hat{s}_h))</td>
<td>1 600 – 2 300</td>
</tr>
</tbody>
</table>

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**

<table>
<thead>
<tr>
<th>Mean CPT cone resistance ( q_c ) [MN/m(^2)]</th>
<th>Ultimate limit state value ( q_{s,k} ) of pile skin friction [kN/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.5</td>
<td>55 – 80</td>
</tr>
<tr>
<td>15</td>
<td>105 – 140</td>
</tr>
<tr>
<td>( \geq 25 )</td>
<td>130 – 170</td>
</tr>
</tbody>
</table>

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**

<table>
<thead>
<tr>
<th>Relative settlement of the pile head ( s/D_b )</th>
<th>Pile base resistance ( q_{b,k} ) [kN/m(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear strength ( c_{u,k} ) of the undrained soil [kN/m(^2)]</td>
</tr>
<tr>
<td></td>
<td>100</td>
</tr>
<tr>
<td>0.02</td>
<td>350 – 450</td>
</tr>
<tr>
<td>0.03</td>
<td>450 – 550</td>
</tr>
<tr>
<td>0.10 (( \hat{s}_h))</td>
<td>800 – 1 000</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Shear strength $c_{u,k}$ of the undrained soil [kN/m²]</th>
<th>Ultimate limit state value $q_{s,k}$ of pile skin friction [kN/m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>30 – 40</td>
</tr>
<tr>
<td>150</td>
<td>50 – 65</td>
</tr>
<tr>
<td>$\geq 250$</td>
<td>65 – 85</td>
</tr>
</tbody>
</table>

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 $s_{g,t}$ may be approximately determined using:

$$s_{g,t} = 1.30 \cdot s_{g}$$  \hspace{1cm} (11)

where $s_{g}$ 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 calculated. 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 \( \tau_n \), 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 \( \tau_n \) 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

\[ \tau_{n,k} = \alpha \cdot c_{u,k} \quad (12) \]

where:

- \( \alpha \) 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 $\alpha$ 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:
  \[ \tau_{uk} = K_0 \cdot \tan \varphi'_k \cdot \sigma'_v = \beta \cdot \sigma'_v \]  
  where:
  - $\sigma'_v$ effective vertical stress;
  - $K_0$ coefficient of at-rest earth pressure;
  - $\varphi'_k$ characteristic value of the friction angle;
  - $\beta$ factor for specifying the value of the characteristic negative skin friction for non-cohesive and cohesive soils.

According to the literature the factor $\beta$ 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 $F_{\text{n,k}}(\text{SLS})$ 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 $F_{\text{n,k}}(\text{ULS})$ in the ultimate limit state ("external" pile capacity), it is recommended to specify the pile settlement $s_g = s_{\text{ult}}$ in the ultimate limit state in accordance with Section 5, depending on the method selected to determine pile capacity.

Comparing the deformations for $s_g = s_{\text{ult}}$ 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 $\text{DS-P}$, 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 $F_{\text{n,k}}(\text{SLS})$ 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:
  \[ F_d = F_{\text{n,k}} + F_{\text{n,k}}(\text{SLS}) + F_{\text{Q,rep}} \]  

b) Ultimate limit state (ULS): the characteristic action $F_{\text{n,k}}(\text{ULS})$ and the location of the neutral point have to be calculated by comparing the deformations associated with the pile settlement $s_g = s_{\text{ult}}$ 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 $s_{\text{ult}}$ is greater than $s(SLS)$ (except for piles on rock, for example). The design value of the effects is:
  \[ F_d = (F_{\text{G,k}} + F_{\text{n,k}}(\text{ULS})) \cdot \gamma_G + F_{\text{Q,rep}} \cdot \gamma_Q \]  

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 $F_{\text{n,k}}(\text{SLS})$ 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 $q_c > 25 \text{ MN/m}^2$,
- 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:

\[ F_{y}(\text{SLS}) = F_{y} \leq R_{d}(\text{SLS}) = R_{k}(\text{SLS}) \]  \hspace{1cm} (16)

Partial factors of \( \gamma = 1.0 \) 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:

\[ s_{k} \leq \text{allow. } s_{k} \]  \hspace{1cm} (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 \( \Delta s_{k} \) 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 \( s_{k,\text{max}} \) and lower bounds \( s_{k,\text{min}} \) of the settlements \( s_{k} \) after Figure 9 b) shall be determined in the range of the resulting pile resistance \( R(\text{SLS}) \), adopting the following equation:

\[ \Delta s_{k} = \kappa \cdot s_{k} \]  \hspace{1cm} (18)

The factor \( \kappa \) 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 recommendations 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 \( R(\text{SLS}) \), 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 \( R(\text{SLS}) \) 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 settlements 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 settlement the average settlement of the pile group by:

$$s_G = s_E \cdot G_s$$  \hspace{1cm} (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 centrally acting, vertical action is given by:

$$G_s = S_1 \cdot S_2 \cdot S_3$$  \hspace{1cm} (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 nomogramms can be found in 'EA-Pfähle' (section 8.2).

The observational 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 prediction 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 predominantly 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

\[
s_g = s_{ult} = 0.10 \cdot D_b (21)
\]

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 \( \xi \) given in Handbook EC 7-1, Table A7.1. The German nationally determined parameters \( \xi_1 \) and \( \xi_2 \) 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.

\[
R_{c,k} = \text{MIN} \left\{ \left( \frac{R_{c,m}}{\xi_1} \right) \frac{R_{c,m}}{\xi_2} \right\} (22)
\]

In the German approach the correlation factors \( \xi_1 \) and \( \xi_2 \) depend according to Table 11 (only) on the number of pile tests performed, and are applied to the mean \( \left( R_{c,m} \right)_{\text{mean}} \) or the smallest value \( \left( R_{c,m} \right)_{\text{min}} \) of \( R_{c,m} \).

If structures possess sufficient stiffness to redistribute loads from “weaker” to “stiffer” compression piles, the numerical values of \( \xi_1 \) and \( \xi_2 \) may be divided by 1.1, assuming that \( \xi_1 \) 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 \( \xi \) for deriving characteristic pile resistances from static pile testing on compression and tension piles

<table>
<thead>
<tr>
<th>( n )</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>( \geq 5 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \xi_1 )</td>
<td>1.35</td>
<td>1.25</td>
<td>1.15</td>
<td>1.05</td>
<td>1.00</td>
</tr>
<tr>
<td>( \xi_2 )</td>
<td>1.35</td>
<td>1.15</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
</tr>
</tbody>
</table>

\( n \) 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 $R_k$, 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 $R_m$ 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 $(R_{c,m})_{\text{mean}}$ and $(R_{c,m})_{\text{min}}$.

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 $(R_{c,m})_{\text{mean}}$ 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 $(R_{c,m})_{\text{mean}}$.

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](image-url)
Table 12  Base values $\xi_{0,i}$ with corresponding increase factors and model factors for correlation factors $\xi_5$ and $\xi_6$ used to derive characteristic values from impact or dynamic pile tests

<table>
<thead>
<tr>
<th>$\xi_{0,i}$ for n =</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>15</th>
<th>$\geq$ 20</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\xi_{0,5}$</td>
<td>1.60</td>
<td>1.50</td>
<td>1.45</td>
<td>1.42</td>
<td>1.40</td>
</tr>
<tr>
<td>$\xi_{0,6}$</td>
<td>1.50</td>
<td>1.35</td>
<td>1.30</td>
<td>1.25</td>
<td>1.25</td>
</tr>
</tbody>
</table>

- 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 $\xi_i$:

\[ \xi_i = (\xi_{0,i} + \Delta \xi) \cdot \eta_D, \]
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 $\eta_D$ 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 $\xi_5$ and $\xi_6$ may be divided by 1.1.

c) The following apply to the model factor $\eta_D$ 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 $\xi_5$ and $\xi_6$ 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 the $\xi_i$ correlation factors. The base values of the $\xi_{0,i}$ correlation factors are used to calculate the $\xi_i$ correlation factors, together with the associated ‘surcharge factors’ $\Delta\xi$ and model factors $\eta_D$ 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.

$$R_{c,k} = \min \left\{ \frac{(R_{c,m})_{\text{mean}}}{\xi_5}, \frac{(R_{c,m})_{\text{min}}}{\xi_6} \right\}$$

$\xi_5$ and $\xi_6$ 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 $(R_{c,m})_{\text{mean}}$ or the smallest value $(R_{c,m})_{\text{min}}$ of $R_{c,m}$.

If structures possess sufficient stiffness to redistribute loads from “weaker” to “stiffer” compression piles, the numerical values of $\xi_5$ and $\xi_6$ 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.

![Figure 12](image)

**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 as well as structure possessing sufficient stiffness to redistribute loads from ‘weaker’ to ‘stiffer’ piles. An allowable pile settlement allows $s_k = 2.0 \text{ cm}$ 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 $(R_{c,m})_{\text{mean}}$ or the minimum value $(R_{c,m})_{\text{min}}$ of the pile load test results using Equation (A4.1) as follows:

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

The correlation factors $\xi_1$ and $\xi_2$ 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 $\xi_1$ 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 $\kappa$ values (based on Kempfert 2009). For the present case $\kappa = 0.15$ 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).
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

<table>
<thead>
<tr>
<th>Piles of</th>
<th>$(R_{c,m})_{\text{mean}}$</th>
<th>$(R_{c,m})_{\text{min}}$</th>
<th>$R_{c,k}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;flexible&quot; structures</td>
<td>4.163</td>
<td>4.400</td>
<td>4.163</td>
</tr>
<tr>
<td>&quot;stiff&quot; structures</td>
<td>4.565</td>
<td>4.819</td>
<td>4.565</td>
</tr>
</tbody>
</table>

### 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):

$$F_{c,d} = F_{G,k} \cdot \gamma_G + F_{Q,\text{rep},k} \cdot \gamma_Q = 1.500 \cdot 1.35 + 1.000 \cdot 1.50 = 3.525 \text{ MN}$$

$$R_{c,d} = R_{c,k}/\gamma_t = 4.163/1.10 = 3.785 \text{ MN}$$

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

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

$$F_{c,d} = 3.525 \text{ MN}$$

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

$$F_{c,d} = 3.525 \text{ MN} < R_{c,d} = 4.150 \text{ MN}$$

### 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 13b.
The specified allowable settlement (e.g. specified in the structural design) in the example is \( s_k = 2 \text{ cm} \).

Using Figure 14 a, serviceability is demonstrated via pile forces after 5.4.1, Eq. (16) as:

\[
F_d (\text{SLS}) = F_k = 2.5 \text{ MN} < R_d (\text{SLS}) = R_k (\text{SLS}) = 2.7 \text{ MN}.
\]

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

\[
\text{exist. } s_{k,\text{max}} = 1.8 \text{ cm} < \text{allow. } s_k = 2.0 \text{ cm}
\]

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](image)

*Figure 15: Ground profile, penetration test diagram and dimensions for an example calculation of the resistance-settlement curve; \( D = 0.9 \text{ m}, U = 2.83 \text{ m}, A = 0.64 \text{ m}^2 \)*
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 Rₘₘₜ**

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**

<table>
<thead>
<tr>
<th>Stratum i</th>
<th>$A_{s,i}$ [m²]</th>
<th>$c_{q,i}$ or $q_{c,i}$ [MN/m²]</th>
<th>$q_{b,i}$ [MN/m²]</th>
<th>$R_{s,k,i}$ [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.20 to 5.20</td>
<td>8.48</td>
<td>0.10</td>
<td>0.039 – 0.051</td>
<td>0.331 – 0.432</td>
</tr>
<tr>
<td>5.20 to 7.70</td>
<td>7.07</td>
<td>7.00</td>
<td>0.051 – 0.075</td>
<td>0.361 – 0.530</td>
</tr>
<tr>
<td>7.70 to 10.20</td>
<td>7.07</td>
<td>11.00</td>
<td>0.078 – 0.108</td>
<td>0.551 – 0.764</td>
</tr>
</tbody>
</table>

$^{a}$ 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 $1 \cdot D$ (0.9 m) above and $3 \cdot D$ ($3 \cdot D = 2.70$ 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**

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

**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 $R_{s,k}$ is given by the characteristic resistance-settlement curve in Figure 16.
Table 17: Pile resistance as a function of pile head settlement (lower values)

<table>
<thead>
<tr>
<th>Relative settlement s/D</th>
<th>Pile head settlement [cm]</th>
<th>Rc,k(s) [MN]</th>
<th>Rb,k(s) [MN]</th>
<th>Rc,k(s) [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>s_0 = 0.02</td>
<td>1.1</td>
<td>1.243</td>
<td>0.784</td>
<td>1.722</td>
</tr>
<tr>
<td>s_0 = 0.03</td>
<td>2.7</td>
<td>1.243</td>
<td>1.008</td>
<td>2.027</td>
</tr>
<tr>
<td>s_0 = 0.10</td>
<td>9.0</td>
<td>1.243</td>
<td>2.080</td>
<td>3.323</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Relative settlement s/D</th>
<th>Pile head settlement [cm]</th>
<th>Rc,k(s) [MN]</th>
<th>Rb,k(s) [MN]</th>
<th>Rc,k(s) [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>s_0 = 0.02</td>
<td>1.4</td>
<td>1.726</td>
<td>0.809</td>
<td>2.535</td>
</tr>
<tr>
<td>s_0 = 0.03</td>
<td>2.7</td>
<td>1.726</td>
<td>1.336</td>
<td>3.062</td>
</tr>
<tr>
<td>s_0 = 0.10</td>
<td>9.0</td>
<td>1.726</td>
<td>2.768</td>
<td>4.494</td>
</tr>
</tbody>
</table>

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 $R_{s,k}(s_{ug})$ 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

<table>
<thead>
<tr>
<th>Stratum i [m]</th>
<th>$A_{s,i}$ [m²]</th>
<th>$q_{c,i}$ [MN/m²]</th>
<th>$q_{s,k,i}(s_{sg^*})$ [MN/m²]</th>
<th>$\eta_s$ [-]</th>
<th>$R_{s,k,i}(s_{sg^*})$ [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.0 to 20.3</td>
<td>10.22</td>
<td>17.50</td>
<td>0.070 – 0.098</td>
<td>1.0</td>
<td>0.715 – 1.002</td>
</tr>
</tbody>
</table>

$R_{s,k}(s_{sg^*}) = 0.715 – 1.002$ MN

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

<table>
<thead>
<tr>
<th>Stratum i [m]</th>
<th>$A_{s,i}$ [m²]</th>
<th>$q_{c,i}$ [MN/m²]</th>
<th>$q_{s,k,i}(s_{sg^*})$ [MN/m²]</th>
<th>$\eta_s$ [-]</th>
<th>$R_{s,k,i}(s_{sg^*})$ [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.0 to 20.3</td>
<td>10.22</td>
<td>17.50</td>
<td>0.103 – 0.134</td>
<td>1.0</td>
<td>1.053 – 1.370</td>
</tr>
</tbody>
</table>

$R_{s,k}(s_{sg^*}) = 1.053 – 1.370$ 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 $R_{s,k}(s_{sg^*})$:

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

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

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

$s_{sg^*} = 0.50 \times 1.002 = 0.5$ 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 $4 \times D_{eq}$ below to $1 \times D_{eq}$ above the pile base.

The equivalent pile diameter of a square prefabricated driven pile is determined using:

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

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

$D_{eq} = 1.13 \times 0.35 = 0.40$ m.

The nominal value of the square pile base area in this case is:
\[ A_b = a^2 = 0.35^2 = 0.123 \, m^2. \]

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

\[ q_{c,m} = \frac{1.175 + 4.150}{5} = 15.5 \, MN/m^2 \]

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

<table>
<thead>
<tr>
<th>Relative settlement s/D</th>
<th>( q_{b,k} ) [MN/m²]</th>
<th>( \eta_b ) [-]</th>
<th>( R_{b,k}(s) ) [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.035</td>
<td>4.025 – 6.550</td>
<td>1.0</td>
<td>0.495 – 0.806</td>
</tr>
<tr>
<td>0.100</td>
<td>7.658 – 10.265</td>
<td>1.0</td>
<td>0.942 – 1.263</td>
</tr>
</tbody>
</table>

**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)

<table>
<thead>
<tr>
<th>Relative settlement s/D</th>
<th>Pile head settlement [cm]</th>
<th>( R_{c,k}(s) ) [MN]</th>
<th>( R_{b,k}(s) ) [MN]</th>
<th>( R_{c,k}(s) ) [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s_{g*} )</td>
<td>0.4</td>
<td>0.715</td>
<td>0.141</td>
<td>0.856</td>
</tr>
<tr>
<td>0.035</td>
<td>1.4</td>
<td>0.809</td>
<td>0.495</td>
<td>1.304</td>
</tr>
<tr>
<td>0.100</td>
<td>4.0</td>
<td>1.053</td>
<td>0.942</td>
<td>1.995</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Relative settlement s/D</th>
<th>Pile head settlement [cm]</th>
<th>( R_{c,k}(s) ) [MN]</th>
<th>( R_{b,k}(s) ) [MN]</th>
<th>( R_{c,k}(s) ) [MN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( s_{g*} )</td>
<td>0.5</td>
<td>1.002</td>
<td>0.288</td>
<td>1.290</td>
</tr>
<tr>
<td>0.035</td>
<td>1.4</td>
<td>1.097</td>
<td>0.806</td>
<td>1.903</td>
</tr>
<tr>
<td>0.100</td>
<td>4.0</td>
<td>1.370</td>
<td>1.263</td>
<td>2.633</td>
</tr>
</tbody>
</table>

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


BGR, Bundesanstalt für Geowissenschaften und Rohstoffe (2008): Geologische Karte der Bundesrepublik Deutschland 1:1.000.000 (GK 1000). Revised ed., Hannover (online).


