1 Subsoil

1.1 Mean characteristic values of soil parameters (R 9)

1.1.1 General

For preliminary designs, the characteristic values (index *k*) given in Table R 9-1 may be used as empirical values for a larger body of soil. Without verification, the values in the table may only be assumed for low penetration resistance or soft consistency.

Detailed and final designs should always be based on the soil parameter values determined by way of soil investigations and laboratory tests (R 88, section 1.4). Wherever possible, the effective shear parameters φ' and c' of cohesive soils should be ascertained in triaxial tests on undisturbed soil samples.

According to Wroth [228], the angle of internal friction φ' for non-cohesive, densely bedded, compact soils in the plane strain state amounts to 9/8 of the angle of internal friction measured in a triaxial test. Therefore, this can be increased by up to 10% in calculations for long waterfront structures with the consent of the geotechnical expert.

The characteristic values of the shear parameters φ'_k and c'_k for cohesive soils apply to calculations for final stability (consolidated state, final strength).

Empirical values for the shear parameters of the undrained, initially loaded soil $c_{u,k}$ are specified in DIN 1055-2:2010-11.

1.2 Layout and depths of boreholes and penetrometer tests (R 1)

1.2.1 General

The nature and extent of soil investigations, their layout and the depth of any such investigations must be determined by a geotechnical expert according to the provisions of DIN EN 1997-2 and DIN 4020.

The aim of boreholes is to investigate the stratification and obtain soil samples for soil mechanics tests in the laboratory. For investigation and monitoring of groundwater conditions, boreholes can be upgraded to groundwater monitoring wells.

Penetrometer tests allow the strength properties of the in situ soil types to be determined. With the help of empirical correlations, the soil types can be identified and the values of soil properties derived.

Recommendations of the Committee for Waterfront Structures Harbours and Waterways – EAU 2012, 9th Edition. Issued by the Committee for Waterfront Structures of the German Port Technology Association and the German Geotechnical Society.

Table R 9-1. Characteristic values of soil parameters (empirical values)

No.	1	2	3	4	5		6		7	8	9
	Soil type	Soil group to DIN 18196[1)]	Penetration resistance	Consistency in initial state, i.e. to DIN 14688-1	Unit weight		Compressibility[2)] Initial loading[3)] $E_S = v_e \sigma_{at} (\sigma/\sigma_{at})^{we}$		Shear parameters of drained soil		Hydraulic conductivity
			q_c		γ_k	γ'_k	v_e	w_e	φ'_k	c'_k	K_k
			MN/m^2		kN/m^3	kN/m^3			degree	kN/m^2	m/s
1	Gravel, uniformly graded	GE U[4)] < 6	< 7.5 7.5–15 > 15		16.0 17.0 18.0	8.5 9.5 10.5	400 900	0.6 0.4	30.0–32.5 32.5–37.5 35.0–40.0		2×10^{-1} to 1×10^{-2}
2	Gravel, well- or gap-graded	GW, GI 6 ≤ U[4)] ≤ 15	< 7.5 7.5–15 > 15		16.5 18.0 19.5	9.0 10.5 12.0	400 1100	0.7 0.5	30.0–32.5 32.5–37.5 35.0–40.0		1×10^{-2} to 1×10^{-6}
3	Gravel, well- or gap-graded	GW, GI U[4)] > 15	< 7.5 7.5–15 > 15		17.0 19.0 21.0	9.5 11.5 13.5	400 1200	0.7 0.5	30.0–32.5 32.5–37.5 35.0–40.0		1×10^{-2} to 1×10^{-6}
4	Sandy gravel with proportion $d < 0.06$ mm < 15%	GU, GT	< 7.5 7.5–15 > 15		17.0 19.0 21.0	9.5 11.5 13.5	400 800 1200	0.7 0.6 0.5	30.0–32.5 32.5–37.5 35.0–40.0		1×10^{-5} to 1×10^{-6}

5	Gravel/sand/ fine grain mixture with proportion $d < 0.06$ mm > 15%	$G\overline{U}$, $G\overline{T}$	< 7.5 7.5–15 > 15		16.5 18.0 19.5	9.0 10.5 12.0	150 275 400	0.9 0.8 0.7	30.0–32.5 32.5–37.5 35.0–40.0		1×10^{-7} to 1×10^{-11}
6	Sand, uniformly graded, coarse sand	SE $U^{4)} < 6$	< 7.5 7.5–15 > 15		16.0 17.0 18.0	8.5 9.5 10.5	250 475 700	0.75 0.60 0.55	30.0–32.5 32.5–37.5 35.0–40.0		5×10^{-3} to 1×10^{-4}
7	Sand, uniformly graded, fine sand	SE $U^{4)} < 6$	< 7.5 7.5–15 > 15		16.0 17.0 18.0	8.5 9.5 10.5	150 225 300	0.75 0.65 0.60	30.0–32.5 32.5–37.5 35.0–40.0		1×10^{-4} to 2×10^{-5}
8	Sand, well- or gap-graded	SW, SI $6 \leq U^{4)} \leq 15$	< 7.5 7.5–15 > 15		16.5 18.0 19.5	9.0 10.5 12.0	200 400 600	0.70 0.60 0.55	30.0–32.5 32.5–37.5 35.0–40.0		5×10^{-4} to 2×10^{-5}
9	Sand, well- or gap-graded	SW, SI $U^{4)} > 15$	< 7.5 7.5–15 > 15		17.0 19.0 21.0	9.5 11.5 13.5	200 400 600	0.70 0.60 0.55	30.0–32.5 32.5–37.5 35.0–40.0		1×10^{-4} to 1×10^{-5}
10	Sand $d < 0.06$ mm < 15%	SU, ST	< 7.5 7.5–15 >15		16.0 17.0 18.0	8.5 9.5 10.5	150 350 500	0.80 0.70 0.65	30.0–32.5 32.5–37.5 35.0–40.0		2×10^{-5} to 5×10^{-7}
11	Sand $d < 0.06$ mm > 15%	$S\overline{U}$, $S\overline{T}$	< 7.5 7.5–15 >15		16.5 18.0 19.5	9.0 10.5 12.0	50 250	0.90 0.75	30.0–32.5 32.5–37.5 35.0–40.0		2×10^{-6} to 1×10^{-9}

Table R 9-1. (*Continued*)

No.	1	2	3	4	5		6		7	8	9
	Soil type	Soil group to DIN 18196[1)]	Penetration resistance	Consistency in initial state, i.e. to DIN 14688-1	Unit weight		Compressibility[2)] Initial loading[3)] $E_S = v_e \sigma_{at} (\sigma/\sigma_{at})^{we}$		Shear parameters of drained soil		Hydraulic conductivity
			q_c		γ_k	γ'_k	v_e	w_e	φ'_k	c'_k	K_k
			MN/m^2		kN/m^3	kN/m^3			degree	kN/m^2	m/s
12	Inorganic cohesive soils with low plasticity ($w_L < 35\%$)	UL		soft firm stiff	17.5 18.5 19.5	9.0 10.0 11.0	40 110	0.80 0.60	27.5–32.5	0 2–5 5–10	1×10^{-5} to 1×10^{-7}
13	Inorganic cohesive soils with medium plasticity ($35\% < w_L < 50\%$)	UM		soft firm stiff	16.5 18.0 19.5	8.5 9.5 10.5	30 70	0.90 0.70	25.0–30.0	0 5–10 10–15	2×10^{-6} to 1×10^{-9}

14	Inorganic cohesive soils with low plasticity (w_L < 35%)	TL		soft firm stiff	19.0 20.0 21.0	9.0 10.0 11.0	20 50	1.0 0.90	25.0–30.0	0 5–10 10–15	1×10^{-7} to 2×10^{-9}
15	Inorganic cohesive soils with medium plasticity (35% < w_L < 50%)	TM		soft firm stiff	18.5 19.5 20.5	8.5 9.5 10.5	10 30	1.0 0.95	22.5–27.5	5–10 10–15 15–20	5×10^{-8} to 1×10^{-10}
16	Inorganic cohesive soils with high plasticity (50% < w_L)	TA		soft firm stiff	17.5 18.5 19.5	7.5 8.5 9.5	6 20	1.0 1.0	20.0–25.0	5–15 10–20 15–25	1×10^{-9} to 1×10^{-11}
17	Organic silt, organic clay	OU and OT		very soft soft firm	14.0 15.5 17.0	4.0 5.5 7.0	5 20	1.00 0.85	17.5–22.5	0 2–5 5–10	1×10^{-9} to 1×10^{-11}
18	Peat[5)]	HN, HZ		very soft soft firm stiff	10.5 11.0 12.0 13.0	0.5 1.0 2.0 3.0	[5)]	[5)]	[5)]	[5)]	1×10^{-5} to 1×10^{-8}

Table R 9-1. (*Continued*)

No.	1	2	3	4	5		6		7	8	9
	Soil type	Soil group to DIN 18196[1]	Penetration resistance	Consistency in initial state, i.e. to DIN 14688-1	Unit weight		Compressibility[2] Initial loading[3] $E_S = v_e \sigma_{at}(\sigma/\sigma_{at})^{we}$		Shear parameters of drained soil		Hydraulic conductivity
			q_c		γ_k	γ'_k	v_e	w_e	φ'_k	c'_k	K_k
			MN/m^2		kN/m^3	kN/m^3			degree	kN/m^2	m/s
19	Mud[6] Digested sludge	F		very soft soft	12.5 16.0	2.5 6.0	4 15	1.0 0.9	[6]	0	1×10^{-7} 1×10^{-9}

Explanatory notes:

1) Code letters for primary and secondary components:

F mud
G gravel
H peat (humus)
O organic inclusions
S sand
T clay
U silt

Code letters for characteristic physical soil properties:
Particle size distribution:

W wide-graded particle size distribution
E narrow-graded particle size distribution
I gap-graded particle size distribution

Plasticity:

L low plasticity
M medium plasticity
A high plasticity

Degree of decomposition of peat:

N not decomposed or scarcely decomposed peat
Z decomposed peat

2) Symbols:

v_e stiffness factor, empirical parameter
w_e stiffness exponent, empirical parameter
w_e empirical parameter
σ load in kN/m^2
σ_{at} atmospheric pressure (= 100 kN/m^2)

3) v_e values for repeated load up to 10 times higher, w_e values tend towards 1

4) U uniformity coefficient

5) The compressibility and shear parameter values for peat exhibit such a wide scatter that empirical values cannot be given.

6) The effective angle of internal friction of fully consolidated mud can be very high, but the value corresponding to the true degree of consolidation, which can only be determined reliably in laboratory tests, always governs.

The number and layout of boreholes and penetrometer tests must always be such that all the characteristics of the subsoil relevant to the planning are established and a sufficient number of suitable soil samples is obtained for the laboratory tests. When determining the number and type of boreholes and penetrometer tests, the results of earlier surveys in the form of geological maps and, where applicable, the findings of earlier boreholes and penetrometer tests should also be taken into account.
Geophysical surface measurements in conjunction with the boreholes and penetrometer tests can supply two-dimensional data on the geological profile, groundwater level and indications regarding any large obstacles in the subsoil.
Where major construction projects are involved, it can be useful to begin with principal boreholes and penetrometer tests to gain an overall picture and then to supplement these with intermediate boreholes and further penetrometer tests during the planning phase.

1.2.2 Principal boreholes

Principal boreholes should preferably lie on the later axis of the structure (waterfront). For cantilever walls they should be drilled to a depth equal to approximately twice the difference in ground levels or as far as a known geological stratum. As a guide, the recommended borehole spacing is approx. 50 m; recommendations regarding their location and depth are specified in DIN EN 1997-2 (2.4.1.3) and DIN 4020. In specific cases, the positions and spacings of the boreholes must be adapted in line with the geological and constructional boundary conditions. Given that soil samples for soil mechanics tests in the laboratory must be at least grade 2 according to DIN EN ISO 22475-1, the principal boreholes must be designed as boreholes suitable for obtaining samples in liners.

1.2.3 Intermediate boreholes

Depending on the findings of the principal boreholes or the earlier penetrometer tests, intermediate boreholes are also sunk to the depth of the principal boreholes, or to a depth at which a known, homogeneous soil stratum is encountered. The typical borehole spacing is again approx. 50 m; in some cases 25 m is necessary.

1.2.4 Penetrometer tests

Penetrometer tests are generally executed according to the layout in Fig. R 1-1. As far as possible, they are sunk to the same depth as the principal boreholes. The relevant standards should be consulted regarding details of the equipment for and execution of penetrometer tests, along with their application.
In order to interpret the results of penetrometer tests, individual tests must be carried out directly adjacent to boreholes. In such cases the

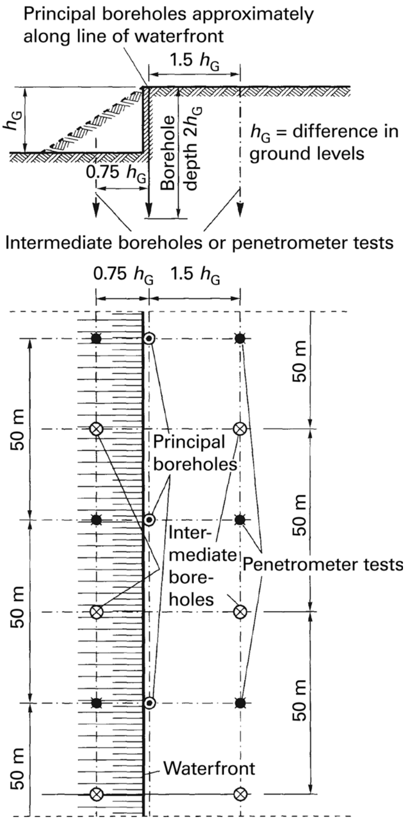

Fig. R 1-1. Example of layout of boreholes and penetrometer tests for waterfront structures

penetrometer tests must be performed prior to drilling the boreholes in order to prevent the results of the penetrometer test from being influenced due to any loosening of the soil during drilling.

1.3 Geotechnical report (R 150)

The results of the soil investigations are compiled in a geotechnical report according to DIN EN 1997-1 (3.4) or DIN EN 1997-2 (6). The nature and extent of such investigations along with their results must be recorded in that document.
The geotechnical report contains the characteristic design values of the soil parameters, and might also include references to the proposed methods of calculation. An assessment of the subsoil should also include investigations regarding chemical constituents that could damage concrete and/or steel and details of any contamination.

The findings gleaned from the geotechnical report are summarised as foundation recommendations for the specific structure. For waterfront structures, this also includes information on the installation of piles and sheet piles as well as any obstacles to driving.
The soil investigations can be supplemented by loading tests and trial embankments in order to be able to make a proper assessment of the loadbearing behaviour of foundation elements and soil compaction options. If required, a number of model tests can be carried out to assess soil-structure interaction. The execution of and results from loading tests, trial embankments and model tests must be recorded in the geotechnical report.
Together with the verification of stability and serviceability, the aforementioned contents form the basis of the draft geotechnical report according to DIN EN 1997-2 (section 2.8).

1.4 Determining the shear strength c_u of saturated, undrained cohesive soils (R 88)

If saturated cohesive soil is loaded without being able to consolidate (undrained conditions), its change in volume is negligible due to the low compressibility of the pore water at loads below its strength. The load generates excess pore water pressure only and no additional effective stresses in the soil skeleton. As a result, the angle of internal friction for saturated cohesive soils in undrained conditions is $\varphi_u = 0$. The strength is only described by the cohesion of the undrained soil c_u. In the case of partial saturation, part of the load can generate additional effective stresses in the soil skeleton; in such cases $\varphi' > \varphi_u > 0$.

1.4.1 Cohesion c_u of undrained soil

The cohesion c_u of undrained cohesive soils essentially depends on the following conditions:

- For normally consolidated soils, c_u is proportional to the effective vertical stress σ'_v, i.e. c_u increases linearly with depth:

  $$\frac{c_u}{\sigma'_v} = \lambda_{cu}$$

  According to Jamiolowski et al. [110], the cohesion constant is $\lambda_{cu} = 0.23 \pm 0.04$, although Gebreselassie [72] states that values as low as $\lambda_{cu} = 0.18$ and even lower are possible.
  For north German marine clay, c_u is often very low and any dependency on the vertical stress σ'_v is hard to measure with any certainty.

- For overconsolidated soil, c_u is likewise proportional to the effective vertical stress σ'_v, but is also determined from the stress history:

$$\frac{c_u}{\sigma'_v} = \lambda_{cu} OCR^{\alpha}$$

The overconsolidation ratio (OCR) is the ratio of the stress σ'_{vc}, for which the soil is consolidated, and the current stress σ'_v:

$$OCR = \frac{\sigma'_{vc}}{\sigma'_v}$$

Reference values for exponent α lie between 0.8 and 0.9.

- A number of different authors demonstrate that the cohesion c_u of an undrained soil depends on the stress path. Under triaxial compression (tc), c_u is greater than for triaxial extension (te) Bjerrum [20]; Jamiolowski et al. [110]; Scherzinger [190]; $c_{u,tc}$ can be approx. 50% greater than $c_{u,te}$. Values for direct simple shear $c_{u,dss}$ lie in between for the same pore volume:

$$c_{u,tc} > c_{u,dss} > c_{u,te}$$

- Owing to the viscosity of cohesive soils, the cohesion c_u of an undrained soil depends on the rate of load application. This can be described using the shear rules of Leinenkugel [132] or Randolph [168], for instance. Leinenkugel's relationship is

$$\frac{c_u}{c_{u\alpha}} = \left[1 + I_{v\alpha} \ln\left(\frac{\dot{\gamma}}{\dot{\gamma}_\alpha}\right)\right]$$

The viscosity index $I_{v\alpha}$ for the reference strain rate $\dot{\gamma}_\alpha$ can be determined, for example, by means of CU triaxial tests with an abruptly varying strain rate (step test) or using one-dimensional creep tests. Reference values for $I_{v\alpha}$ can be found, for example, in Leinenkugel [132] and Gudehus [78].

1.4.2 Determining the cohesion c_u of an undrained soil

The cohesion c_u of an undrained soil can be determined in laboratory or field tests. There are two essentially different methods for such investigations: the recompression method and the stress history method.

1.4.2.1 Recompression method

In this method, c_u is determined by way of triaxial tests on soil samples that are reconsolidated prior to shearing with the stress that acts on the soil in situ Bjerrum [20]. According to Seah und Lai [197], however, this method overestimates the cohesion c_u of normally consolidated soil. Therefore, the recompression method is preferred for highly structured, brittle soils, e.g. sensitive clays, cemented soils and severely overconsolidated soils. The results of shear tests should always be checked by comparing them with the stress history.

1.4.2.2 The stress history and normalised soil engineering properties method (SHANSEP)

This method enables the cohesion c_u to be determined while taking into account the sample disorder, the anisotropy to a limited extent and the rate of load application. It is based on investigations carried out at MIT during the 1960s and was initially published by Ladd and Foott [129]. A revised version can be found in Ladd and DeGroot [128]. The SHANSEP method involves the following steps for specifying the soil model:

- Carrying out a soil investigation, taking special samples (undisturbed soil samples) and compiling a soil profile based on the results of cone penetration tests and field vane shear tests.
- Determining the degree of overconsolidation in the laboratory based on compression tests and deriving the overconsolidation ratio (OCR).
- Determining the effective shear parameters φ'/c' and c_u in laboratory tests, normally by way of triaxial tests. Triaxial tests with anisotropic consolidation (CK_0) and subsequent undrained triaxial compression (UC with $\sigma_1 > \sigma_3$) and triaxial extension (UE with $\sigma_1 < \sigma_3$) are recommended. Stipulation of the reconsolidation stress corresponding to the calculated OCR.
- Performing shear tests to determine the relationship between the OCR and the normalised shear strength c_u/σ'_v.
- Stipulating a c_u design profile for the cohesive strata which lies on the safe side.

1.4.3 Determining c_u in laboratory tests

The advantage of determining c_u in laboratory tests is that the test conditions can be reproduced in an ideal manner within larger test series. However, the disadvantage is that test specimens can never be obtained from boreholes without disturbing their structure and strength. In addition, test specimens are not continuous, meaning that the distribution of c_u over the stratum thickness is only ascertained at discrete points. The test conditions are best controlled in triaxial tests. CU triaxial tests supply both drained and undrained shear parameters because the pore water pressure is measured. When determining c_u by way of laboratory vane shear tests as well as unconfined compression tests, a distorted capillarity influence cannot be ruled out. For soft soils, c_u can also be determined by way of various pressure and fall cone tests.

1.4.4 Field tests

Determining c_u by way of cone penetration tests to DIN 4094-1 and vane shear tests to DIN 4094-4 supplies a profile of the cohesion c_u over the depth. Owing to the high shearing rate, the shear resistance τ_{fvt} in the vane shear test must be reduced by a factor μ, which depends on the plasticity index I_P:

$$c_{u,fvt} = \mu \tau_{fvt}$$

Details of the correction factor μ can be found in DIN EN 1997-2, annex I. The derivation of c_u from the penetration resistance in cone penetration tests requires knowledge of the OCR of the soil. For example, the following applies for the CPTU test:

$$c_{u,cptu} = \frac{q_c - \sigma_v}{N_{kt}}$$

The factor N_{kt} depends on the cone geometry and OCR, and lies between 10 and 20.
The derivation of c_u from borehole-widening tests to DIN 4096 is less common.
Plate load tests to DIN 18134 only supply a c_u value for soils near the surface.

1.4.5 Correlations

A number of authors have suggested correlations between c_u and the water content w, the consistency index I_C, the plasticity index I_P and the liquidity index I_L; Gebreselassie [72] provides a detailed overview of this. It should be noted that these correlations apply, at best, to the soils examined and the test conditions, and can therefore only be used as reference values.

1.5 Assessing the subsoil for the installation of piles and sheet piles and for selecting the installation method (R 154)

1.5.1 General

In the first place, the material, form, size, length and angle of piles and sheet piles play a decisive role with respect to the installation of piles and sheet piles and the selection of the installation method. Important information on this can be found in:

R 21, section 8.1.2,	Design and installation of reinforced concrete sheet pile walls
R 22, section 8.1.1,	Design and installation of timber sheet pile walls
R 34, section 8.1.3,	Design and installation of steel sheet pile walls
R 104, section 8.1.12,	Driving combined steel sheet piling
R 105, section 8.1.13,	Monitoring during the installation of sheet piles, tolerances
R 118, section 8.1.11,	Driving steel sheet piles
R 217, section 9.2.2.1,	Tension piles and anchors

In connection with these recommendations it is especially important to note that when selecting the type of pile section (material, form), it is essential to take into account the stresses due to the installation procedure in the respective subsoil in addition to the structural requirements and economic issues. As a result, the geotechnical report must also include an evaluation of the in situ subsoil with respect to the installation of piles and sheet piles (see also R 150, section 1.3)

1.5.2 Assessment of soil types with respect to installation methods

1.5.2.1 General

The shear parameters have only a limited significance when it comes to describing the behaviour of the subsoil during the installation of piles and sheet piles. For example, rocky calcareous marl can exhibit relatively low shear parameters due to its fissuring, but may in fact present difficult conditions in terms of piling.

1.5.2.2 Impact driving

Easy driving conditions are to be expected in soft or very soft soils such as moorland, peat, silt, marine clay, etc. Easy driving conditions are also to be generally expected in loosely bedded medium and coarse sands and gravels with no rock inclusions, unless there are embedded cemented strata.

Moderate driving conditions are to be expected in moderately densely bedded medium and coarse sands, fine gravel soils and firm clays and loams.

Difficult to very difficult driving conditions are to be expected in most instances of densely bedded medium and coarse gravels, densely bedded fine sandy and silty soils, embedded cemented strata, stiff to very stiff clays, cobbles and moraine strata, glacial till and weathered and soft to medium-hard rock. Earth-moist or dry soils present a greater resistance

to penetration during impact driving than those subject to buoyancy. This does not apply to saturated cohesive soils, and silts especially.
With a number of blows $N_{10} > 30$ per 10 cm penetration in the heavy dynamic penetration test (DPH, DIN EN ISO 22476-2) or $N_{30} > 50$ per 30 cm penetration in borehole dynamic probing (BDP, DIN 4094-2), an increasingly high penetration resistance during driving must be reckoned with. It can generally be assumed that driving is possible up to a number of blows $N_{10} = 80$–100 per 10 cm penetration (DPH). Driving with a higher number of blows can be possible in individual cases. For more information see Rollberg [176,177].

1.5.2.3 Vibratory driving

The skin friction and base resistance of the pile being installed are greatly reduced when using vibratory driving methods. As a result, the piles or sheet piles can quickly reach their required depth compared with impact driving. For more information see R 202, section 8.1.23.
Vibratory driving is particularly successful in sands and gravels with a rounded grain shape and in very soft or soft soil types with low plasticity. Vibrating is much less suitable for highly cohesive soils or sands and gravels with an angular grain shape. Dry fine sands and firm marl and clay soils are particularly critical as they absorb the energy of the vibrator without reducing the skin friction and base resistance.
If the subsoil is compacted during vibration, then its penetration resistance can increase to such an extent that the pile being installed can no longer reach the required depth. This risk arises, in particular, when piles and sheet piles are installed at close spacings and when using vibration in non-cohesive soils. Vibratory driving must be stopped in such cases, see R 202. The use of auxiliary driving measures according to section 1.5.2.5 may represent an option.
Above all, vibratory driving in non-cohesive soils may lead to localised settlement, the magnitude and extent of which depend on the power output of the vibrator, the section being driven, the duration of the vibratory driving and the soil. When working close to existing structures, checks must be made to establish whether any such settlement could cause damage. If required, the installation procedure must be adjusted accordingly.

1.5.2.4 Pressing

For pressing to be used, there should be no obstacles in the soil, or if there are any, they must be removed prior to driving.
Slender sections can generally be pressed hydraulically into cohesive soils without obstacles or into loosely bedded non-cohesive soils. Sections can only be pressed into densely bedded non-cohesive soils

Table R 154-1. Pressing limits for steel sheet piles

Soil parameter			Without driving aid	With driving aid
CPT peak pressure	q_b	MN/m^2	< 20	< 35
CPT skin friction	q_s	MN/m^2	< 0.1	< 0.3
DPH	N_{10}	-	< 25	< 40
Consistency index	I_c	-	< 1.0	> 1.0
Plasticity index	I_P	-	> 10	
Ratio*)	I_f	-	< 1.0	> 1.0
Angle of friction	φ'	°	< 35	< 45

*) $I_f = (\max e - \min e)/\min e$ (higher compactability in the event of decreasing I_f)

if the soil has been loosened beforehand. Empirical values according to Busse [33] are given in Table R 154-1.

1.5.2.5 Auxiliary driving measures

Water-jetting can ease driving in densely bedded sands and gravels as well as in firm and stiff clays in particular – indeed, may be essential to enable driving in the first place.
Additional auxiliary driving measures include pre-drilling to loosen the soil or local soil replacement, etc. Rocky soils can be loosened by way of local blasting in such a way that the required depth can be reached using conventional impact driving and appropriate pile sections. For more information see R 183, section 8.1.10.

1.5.2.6 Driving plant, pile sections, installation methods

Driving plant, pile sections and installation methods must be suited to the subsoil through which the pile sections are being driven, see: R 104, section 8.1.12; R 118, section 8.1.11; R 202, section 8.1.23; R 210, section 7.13.
Slow-acting drop hammers, diesel hammers and hydraulic hammers are suitable for both cohesive and non-cohesive soils. Rapid-acting hammers and vibration plant place less stress on the pile section, but generally are only effective in non-cohesive soils with a rounded grain shape. Rapid-acting hammers or heavy hammers with short drop heights should be preferred when driving in rocky soils, even when using pre-blasting to loosen the ground.
Interruptions during driving a pile, e.g. between initial and final driving, can make subsequent driving easier or harder depending on the soil type

and water saturation as well as the length of the interruption. Any changes to the penetration resistance should generally be identified and quantified by way of tests in advance.

Assessing the subsoil for the installation of piles and sheet piles presumes appropriate experience and specialised knowledge of the installation methods. Experience of construction projects with similar subsoil conditions can indeed be very beneficial.

1.5.2.7 Testing installation methods and loadbearing behaviour in difficult conditions

If on construction projects with considerable embedment depths there are concerns that sheet piles cannot be driven to the depth required to satisfy the structural design without being damaged or other piles cannot reach the intended embedment depth to carry the loads, then test piles must be driven and pile loading tests carried out beforehand. At least two test piles should be driven for each installation method in order to obtain accurate information.

Testing the installation method may also be necessary in order to predict any settlement of the soil as well as the spread and impact of vibrations as a result of the installation method.