1
Safety and verification concept

1.1 Principles of the safety and verification concept for waterfront structures

1.1.1 General

A structure can fail as a result of exceeding the ultimate limit state of bearing capacity ("ultimate limit state – ULS", failure of the soil or the structure, loss of static equilibrium) or the limit state of serviceability ("serviceability limit state – SLS", excessive deformations).

1.1.2 Normative regulations for waterfront structures

The "Eurocodes" (EC) – harmonised directives specifying fundamental safety requirements for buildings and structures – were drawn up as part of the realisation of the European Single Market. Those Eurocodes are as follows:

DIN EN 1990:	Basis of structural design ("EC 0")
DIN EN 1991, EC 1:	Actions on structures
DIN EN 1992, EC 2:	Design of concrete structures
DIN EN 1993, EC 3:	Design of steel structures
DIN EN 1994, EC 4:	Design of composite steel and concrete structures
DIN EN 1995, EC 5:	Design of timber structures
DIN EN 1996, EC 6:	Design of masonry structures
DIN EN 1997, EC 7:	Geotechnical design
DIN EN 1998, EC 8:	Design of structures for earthquake resistance
DIN EN 1999, EC 9:	Design of aluminium structures

The Eurocodes "Basis of structural design" (DIN EN 1990) and "Actions on structures" (DIN EN 1991) with their various parts and annexes form the basis of European construction standards, the starting point for building designs throughout Europe. The other eight Eurocodes, along with their respective parts, relate to these two basic standards.

Verification of safety must always be carried out according to European standards. However, in some instances such verification is not possible with these standards alone; a number of parameters, e.g. numerical values for partial safety factors, have to be specified on a

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national level. Furthermore, the Eurocodes do not cover the entire range of German standards, meaning that a comprehensive set of national standards has been retained in Germany. However, this set of German standards along with its requirements may not contradict the regulations contained in the European standards, which in turn necessitated the revision of national standards.

For proof of stability according to the EAU, the standards DIN EN 1990, DIN EN 1991, DIN EN 1992, DIN EN 1993, DIN EN 1994, DIN EN 1995, DIN EN 1996, DIN EN 1997, DIN EN 1998, DIN EN 1999, and especially DIN EN 1997 (Geotechnical design), are of particular importance. DIN EN 1997-1 defines a number of terms and describes and stipulates limit state verification procedures. The various earth pressure design models for stability calculations are also included in the annexes for information purposes. A particular feature here is that three methods of verification using the partial safety factor concept are available for use throughout Europe.

The publication of DIN 1054:2010 ensured that any duplication of DIN EN 1997-1 was avoided, but specific German experience has been retained. This standard was combined with DIN EN 1997-1:2010 and the National Annex (DIN EN 1997-1/NA:2010) to create the EC 7-1 manual (2015).

The many years of experience with the specific boundary conditions of waterfront structures (e.g. greater tolerances for deformations compared with other engineering works) have led to the EAU containing a number of specific stipulations for the design of such structures that can deviate from those given in DIN EN 1997-1 and DIN 1054.

Those specific stipulations include, for example, the following:

- In some cases, lower partial safety factors for actions, action effects and resistances for the limit state of failure (Section 1.2.4, Tables 1.1 and 1.3).
- The determination of a characteristic resultant hydrostatic pressure by offsetting favourable and unfavourable hydrostatic pressures against each other where this is realistic (see Section 3.3.1).
- Simplified assumptions for hydrostatic pressure (see Section 3.3.2).
- Redistribution of active earth pressure independently of the method of construction for sheet pile walls (see Section 8.2.3.2).
- Increasing the theoretical anchor force by 15% for the robust construction of sheet pile wall components (see Section 9.2).

DIN EN 1997-2 covers the planning, execution and evaluation of soil investigations. As for part 1, this standard has been published together with DIN 4020:2010 and the National Application Document in the EC 7-2 manual (2011).

The execution of special civil engineering works is covered by European standards. In Germany, more specific information for such work is laid out in DIN SPEC publications.

Calculations for large-scale soil stabilisation measures (e.g. jet grouting, grout injection) in Germany are covered by DIN 4093.

Where standards are cited in the recommendations, the current version applies, unless stated otherwise. Standards quoted in the text are listed at the end of each chapter.

1.1.3 Geotechnical categories

The minimum requirements in terms of scope and quality of geotechnical investigations, calculations and monitoring measures are described by three geotechnical categories in accordance with EC 7: low (category 1), normal (category 2) and high (category 3) geotechnical difficulty. These are reproduced in DIN 1054, A 2.1.2. Waterfront structures should be allocated to category 2, or category 3 in the case of difficult subsoil conditions. A geotechnical expert should always be consulted.

1.1.4 Design situations

Load cases for verifying stability and allocating partial safety factors are defined in DIN 1054, Section 6.3.3. These result from the combinations of actions in conjunction with the safety categories for resistances. The following classifications apply to waterfront structures:

1.1.4.1 Design situation DS-P (persistent)

This design situation covers loads due to active earth pressures (separately for the initial and final states in the case of unconsolidated, cohesive soils) and excess water pressure in the case of the frequent occurrence of unfavourable inner and outer water levels (see Section 3.3.2), active earth pressure influences due to normal imposed loads and normal crane and pile loads, instantaneous surcharges due to self-weight and normal imposed loads.

1.1.4.2 Design situation DS-T (transient)

Transient situations, i.e. those related to a certain period of time, are allocated to design situation DS-T. They include, for example, situations during construction or repairs. For hydraulic engineering works, besides permanent actions and variable actions that occur regularly during the service life of the structure, which are all allocated to DS-P, transient actions include limited scour due to currents or ship propellers, excess water pressure in the case of rare occurrences of unfavourable inner and outer water levels (see Section 3.3.2) or wave loads according to Section 4.3.

1.1.4.3 Design situation DS-A (accidental)

This is as for design situation DS-T, but with extraordinary design situations such as unscheduled surcharges over a larger area, unusually extensive flattening of an underwater slope in front of the base of a sheet pile wall, unusual scour due to currents or ship propellers, excess water pressure following extreme water levels (see Section 3.3.2 or 6.2), excess water pressure following exceptional flooding of the waterfront structure, combinations of earth and hydrostatic pressures with wave loads resulting from waves that occur only rarely (see Section 4.3), combinations of earth and hydrostatic pressures with flotsam impact according to Section 6.2.5, all load combinations in conjunction with ice states or ice pressures.

1.1.4.4 Extreme case

When extremely improbable combinations of actions occur concurrently, then DIN 1054, Section A 2.4.7.6.1, A(4), A 2.4.7.6.3 and A(5), permit partial safety factors for actions and resistances to be taken as $\gamma_F = \gamma_R = 1.0$. The combination factors are set to $\psi = 1.0$ according to Section 1.2.4.

Examples of this are the simultaneous occurrence of extreme water levels and extreme wave loads due to plunging breakers according to Section 4.3.6, extreme water levels and the simultaneous, total failure of a drainage system (see Section 6.2), combinations of three short-term events acting simultaneously, e.g. high water (highest astronomical tide, see Section 6.2), waves that occur only rarely (see Section 4.3) and flotsam impact (see Section 6.2).

1.2 Verification for waterfront structures

1.2.1 Principles for verification

A stability analysis of a waterfront structure must include the following in particular:

- Details of the use of the facility.
- Drawings of the structure with all essential, planned structural dimensions.
- Brief description of the structure including, in particular, all details that are not readily identifiable from the drawings.
- Design value of bottom depth.
- Characteristic values of all actions.
- Soil strata and associated characteristic values of soil parameters.
- Critical water levels related to the German NHN height reference system (previously mean sea level) or a local gauge datum, together with corresponding groundwater levels (no high water, no flooding).
- Combinations of actions, i.e. load cases.
- Partial safety factors necessary/used.
- Intended building materials and their strengths or resistances.
- All data regarding construction timetables and construction operations, with critical temporary states.
- Description of and justification for the intended verification procedures.
- Information about literature used and other calculation aids.

1.2.2 Design approaches

1.2.2.1 Analytical method

Geotechnical analyses according to the relevant standards generally make use of analytical models based on failure mechanisms. The critical failure planes in the subsoil are either specified or determined by examining variations. The level of safety to be verified should take into account the uncertainties of the earth pressure analysis, the soil investigation and the type of construction. Implicitly, maximum deformations, i.e. serviceability requirements, often have to be considered as well.

1.2.2.2 Numerical simulations

In the meantime, numerical methods of calculation, e.g. the finite element method (FEM), have become established for calculations for the limit state of serviceability (deformations).

An example of a comprehensive numerical simulation for the deformations of a quay structure caused by backfilling can be found in Mardfeld (2005). For earth structure, the analysis of the ultimate limit state can be carried out using the $\varphi' - c'$ reduction. Compared with conventional approaches such as the slip circle method, FEM has the advantage that the shear joint can be in any position, which means that more relevant results can be obtained than is the case when assuming planar or curved failure body geometries. The Z^* method is a good choice when assessing the load-bearing capacity in soil-structure interaction problems because the stresses in the components are determined for the serviceability limit state and then transferred to a conventional analysis. An analysis of the limit state based solely on FEM is currently the subject of debate. When it comes to modelling the ultimate limit state and integrating the safety factors, there are still no stipulations that apply to all cases. Numerical simulations call for modelling based on correct states of stresses and deformations, an adequately large section of the subsoil, the drainage conditions of the soil and, above all, material models for the undisturbed soil types that model the stress–strain behaviour phenomena relevant for the structure. For more information on this topic, please refer to *Numerik in der Geotechnik* (EANG 2014).

1.2.2.3 Observational method

The monitoring method according to DIN EN 1997-1 should be employed for complex structures in which the structural behaviour cannot be modelled with sufficient reliability or accuracy during the design. This involves taking measurements on the structure or in the subsoil and comparing these with predicted or warning values. Countermeasures or safety measures that are to be implemented if warning values are exceeded are inherent to the monitoring method. Deformations and forces obtained from numerical simulations form the basis for assessing in situ measurements.

1.2.2.4 Experiments

Experiments, trials and tests can be used to determine the structural behaviour of individual geotechnical elements and even complex geotechnical load-bearing structures. Tests can be carried out on full-size elements (e.g. trial loadings on piles or pull-out tests on anchors) or on scale models. The latter require compliance with the model laws that ensue from the similitude concept for engineering models if the observations made using the model are to be transferred to full-size components. The various laws of the different physical variables limit the transferability. This is particularly true for geotechnical applications, where the stress state in the subsoil has a crucial influence on the stress–strain behaviour but is very difficult to model. Tests on models can be carried out in a geotechnical centrifuge so that the soil is subjected to a realistic stress state, thus providing a correct representation of the pressure-dependent stress–strain behaviour of the subsoil. Further details of geotechnical centrifuge modelling can be obtained from Technical Committee TC 104 “Physical Modelling in Geotechnics” of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE).

1.2.3 Analysis of the serviceability limit state

Deformation analyses must be carried out for all structures whose function can be impaired or rendered ineffective through deformations. The deformations are calculated with the characteristic values of actions and soil reactions and must be less than the deformations permissible for correct functioning of the component or whole structure. Where applicable, the calculations should include the upper and lower bounds of the characteristic values.

In particular, deformation analyses must consider the course of actions over time in order to allow for critical deformation states during various operating and construction stages.

1.2.4 Analysis of the ultimate limit state

Numerical proof of adequate stability is carried out for limit states STR and GEO-2 with the help of design values (index d) for actions or action effects and resistances, and for limit state GEO-3 with the help of design values for actions or action effects and soil properties.

Verification of safety is assessed according to the following fundamental equation:

$$E_d \leq R_d$$

where

E_d Design value of sum of actions or action effects
R_d Design value of resistances derived from sum of resistances of soil or structural elements

When analysing the limit state of loss of equilibrium (EQU) or failure due to hydraulic heave (HYD) or buoyancy (UPL), it is necessary to compare the design values for favourable and unfavourable or stabilising and destabilising actions and to verify compliance with the respective limit state condition. Resistances do not play a role in these analyses.

Six cases apply for analyses of the ultimate limit state of bearing capacity:

Loss of equilibrium of structure or ground	EQU
Loss of equilibrium of structure or ground due to uplift by water pressure (buoyancy)	UPL
Hydraulic heave, internal erosion or piping in the ground due to hydraulic gradients	HYD
Internal failure or very large deformation of the structure or its components	STR
Failure or very large deformation of the ground	GEO-2
Loss of overall stability	GEO-3

DIN EN 1997-1 permits three options for verifying safety, designated “design approaches 1 to 3”. For approach 1, two groups of partial safety factors are taken into account and are used in two separate analyses. For approaches 2 and 3, a single analysis with one group of partial safety factors suffices.

In approaches 1 and 2, the partial safety factors are applied, in principle, to either actions or action effects and to resistances. However, DIN 1054 stipulates that the characteristic, or representative, effects $E_{Gk,i}$ or $E_{Qrep,i}$ (e.g. shear forces, reactions, bending moments, stresses in the relevant sections of the structure and at interfaces between structure and subsoil) are determined first and then the partial safety factors are applied. This is also referred to as design approach 2*.

In approach 3, the partial safety factors are applied to the soil parameters and to actions or action effects not related to the subsoil. Actions or action effects induced by the subsoil are derived from the factored soil parameters.

According to DIN 1054, design approach 2 (2*) should be used for the geotechnical analysis of limit states STR and GEO-2, and design approach 3 for analysing limit state GEO-3.

The partial safety factors specified in DIN 1054 are reproduced in Tables 1.1–1.3.

Remarks:

- For the limit state of failure due to loss of overall stability GEO-3, the partial safety factors for shear strength are to be taken from Table 1.2, and pull-out resistances are multiplied by partial safety factors according to STR and GEO-2.
- The partial safety factor for the material resistance of steel tension members made from reinforced and prestressed steel for limit states GEO-2 and GEO-3 is given in DIN EN 1992-1-1 as $\gamma_M = 1.15$.
- The partial safety factor for the material resistance of flexible reinforcing elements for limit states GEO-2 and GEO-3 is given in EBGEO (2010).

Provided that greater displacements and deformations of the structure do not impair the stability and serviceability of the structure, as can be the case for waterfront structures, ports, harbours and waterways, the partial safety factor γ_G can be reduced for earth and water pressures in justified cases (DIN 1054, A 2.4.7.6.1, A(3)). This is exploited in the EAU by using the partial safety factors in the form of $\gamma_{G,red}$ (Table 1.1) and $\gamma_{R,e,red}$ (Table 1.3). Furthermore, a partial safety factor $\gamma_G = \gamma_Q = 1.00$ is used for action effects due to permanent and unfavourable variable actions in design situation DS-A.

When calculating a design value for actions F_d according to EN 1990, this value must either be stipulated directly or derived from representative values:

$$F_d \leq \gamma_F \cdot F_{rep}$$

where

$$F_{rep} = \psi \cdot F_k$$

γ_F Partial safety factor
ψ Combination factor

For permanent actions and the leading action of variable actions,

$$F_{rep} = F_k$$

applies.

A combination factor $\psi = 1.00$ is usually used for waterfront structures. To verify safety against buoyancy (UPL) and safety against hydraulic heave (HYD) the design values F_d are always calculated without considering combination factors.

Table 1.1 Partial safety factors for actions and action effects (to DIN 1054:2010, Table A2.1, with additions) for the ultimate and serviceability limit states.

Action or action effect	Symbol	Design situation		
		DS-P	DS-T	DS-A
HYD and UPL: limit state of failure due to hydraulic failure and buoyancy				
Destabilising permanent actions[a)]	$\gamma_{G,dst}$	1.05	1.05	1.00
Stabilising permanent actions	$\gamma_{G,stb}$	0.95	0.95	0.95
Destabilising variable actions	$\gamma_{Q,dst}$	1.50	1.30	1.00
Stabilising variable actions	$\gamma_{Q,stb}$	0	0	0
Flow force in favourable subsoil	γ_H	1.45	1.45	1.25
Flow force in unfavourable subsoil	γ_H	1.90	1.90	1.45
EQU: limit state of loss of equilibrium				
Unfavourable permanent actions	$\gamma_{G,dst}$	1.10	1.05	1.00
Favourable permanent actions	$\gamma_{G,stb}$	0.90	0.90	0.95
Unfavourable variable actions	γ_Q	1.50	1.25	1.00
STR and GEO-2: limit state of failure of structures, components and subsoil				
Action effects from permanent actions generally[a)]	γ_G	1.35	1.20	1.00
Action effects from permanent actions for calculating anchorage[b)]	γ_G	1.35	1.20	1.10
Action effects from favourable permanent actions[c)]	$\gamma_{G,inf}$	1.00	1.00	1.00
Action effects from permanent actions due to earth pressure at rest	$\gamma_{G,EO}$	1.20	1.10	1.00
Water pressure in certain boundary conditions[d)]	$\gamma_{G,red}$	1.20	1.10	1.00
Water pressure in certain boundary conditions for calculating anchorage[b)]	$\gamma_{G,red}$	1.20	1.10	1.10
Action effects from unfavourable variable actions[e)]	γ_Q	1.50	1.30	1.00
Action effects from unfavourable variable actions[f)] for calculating anchorage[b)]	γ_Q	1.50	1.30	1.10
Action effects from favourable variable actions	γ_Q	0	0	0
GEO-3: limit state of failure due to loss of overall stability				
Permanent actions	γ_G	1.00	1.00	1.00
Unfavourable variable actions	γ_Q	1.30	1.20	1.00
SLS: limit state of serviceability				
$\gamma_G = 1.00$ for permanent actions or action effects				
$\gamma_Q = 1.00$ for variable actions or action effects				

a) The permanent actions are understood to include permanent and variable water pressure. Differing from DIN 1054:2010-12, $\gamma_G = 1.00$ applies in DS-A except when verifying anchorage.
b) The design of anchorages (grouted anchors, micropiles, tension piles) also includes verifying stability at the lower failure plane when dealing with retaining structures (Section 9.3).
c) If during the determination of the design values of the tensile action effect a characteristic compressive action effect from favourable permanent actions is assumed to act simultaneously, then this should be considered with the partial safety factor $\gamma_{G,inf}$ (DIN 1054, 7.6.3.1, A(2)).
d) For waterfront structures in which larger displacements can be accommodated without damage, the partial safety factors $\gamma_{G,red}$ for water pressure may be used if the conditions according to Section 8.2.1.3 are complied with (DIN 1054, A 2.4.7.6.1, A(3)).
e) Differing from DIN 1054:2010-12, $\gamma_Q = 1.00$ applies in DS-A except when verifying anchorage.
f) The permanent actions are understood to include permanent and variable water pressures.

Table 1.2 Partial safety factors for geotechnical parameters (DIN 1054:2010, Table A 2.2).

Soil parameter	Symbol	Design situation DS-P	DS-T	DS-A
HYD and UPL: limit state of failure due to hydraulic failure and buoyancy				
Friction coefficient tan φ' of drained soil and friction coefficient tan φ_u of undrained soil	$\gamma_{\varphi'}, \gamma_{\varphi_u}$	1.00	1.00	1.00
Cohesion c' of drained soil and shear strength c_u of undrained soil	$\gamma_{c'}, \gamma_{c_u}$	1.00	1.00	1.00
GEO-2: limit state of failure of structures, components and subsoil				
Friction coefficient tan φ' of drained soil and friction coefficient tan φ_u of undrained soil	$\gamma_{\varphi'}, \gamma_{\varphi_u}$	1.00	1.00	1.00
Cohesion c' of drained soil and shear strength c_u of undrained soil	$\gamma_{c'}, \gamma_{c_u}$	1.00	1.00	1.00
GEO-3: limit state of failure due to loss of overall stability				
Friction coefficient tan φ' of drained soil and friction coefficient tan φ_u of undrained soil	$\gamma_{\varphi'}, \gamma_{\varphi_u}$	1.25	1.15	1.10
Cohesion c' of drained soil and shear strength c_u of undrained soil	$\gamma_{c'}, \gamma_{c_u}$	1.25	1.15	1.10

Table 1.3 Partial safety factors for resistances (according to DIN 1054:2010-12, Table A 2.3, with additions).

Resistance	Symbol	Design situation DS-P	DS-T	DS-A
STR and GEO-2: limit state of failure of structures, components and subsoil				
Soil resistances				
Passive earth pressure and ground failure resistance	$\gamma_{R,e}, \gamma_{R,v}$	1.40	1.30	1.20
Passive earth pressure when determining bending moment[a)]	$\gamma_{R,e,red}$	1.20	1.15	1.10
Sliding resistance	$\gamma_{R,h}$	1.10	1.10	1.10
Pile resistances from static and dynamic pile loading tests				
Base resistance	γ_b	1.10	1.10	1.10
Skin resistance (compression)	γ_s	1.10	1.10	1.10
Total resistance (compression)	γ_t	1.10	1.10	1.10
Skin resistance (tension)	$\gamma_{s,t}$	1.15	1.15	1.15
Pile resistances based on empirical values				
Compression piles	$\gamma_b, \gamma_s, \gamma_t$	1.40	1.40	1.40
Tension piles (in exceptional cases only)	$\gamma_{s,t}$	1.50	1.50	1.50
Pull-out resistances				
Ground or rock anchors	γ_a	1.40	1.30	1.20
Grout body of grouted anchors	γ_a	1.10	1.10	1.10
Flexible reinforcing elements	γ_a	1.40	1.30	1.20

a) Reduction for calculating the bending moment only. For waterfront structures in which larger displacements can be accommodated without damage, the partial safety factors $\gamma_{R,e,red}$ for passive earth pressure may be used if the conditions according to Section 8.2.1.2 are complied with (DIN 1054, A 2.4.7.6.1, A(3)).

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DIN 4020: Geotechnical investigations for civil engineering purposes – Supplementary rules to DIN EN 1997-2.

DIN 4093 (2015): Design of strengthened soil – Set up by means of jet grouting, deep mixing or grouting.

DIN EN 1990 Eurocode: Basis of structural design.

DIN EN 1991 Eurocode 1: Actions on structures.

DIN EN 1992 Eurocode 2: Design of concrete structures.

DIN EN 1993 Eurocode 3: Design of steel structures.

DIN EN 1994 Eurocode 4: Design of composite steel and concrete structures

DIN EN 1995 Eurocode 5: Design of timber structures.

DIN EN 1996 Eurocode 6: Design of masonry structures.

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