CEN/TC 250

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Eurocode 7 — Geotechnical design — Part 3: Geotechnical structures

Eurocode 7 - Entwurf, Berechnung und Bemessung in der Geotechnik — Teil 3: Geotechnische Bauten

Eurocode 7 - Calcul géotechnique — Partie 3: Constructions géotechniques

ICS:

Descriptors:

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European foreword

This document (prEN 1997-3:2022) has been prepared by Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will partially supersede EN 1997-1:2004.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognise the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

0 Introduction

0.1 Introduction to the Eurocodes

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

- *EN 1990 Eurocode: Basis of structural and geotechnical design*
- *EN 1991 Eurocode 1: Actions on structures*
- *EN 1992 Eurocode 2: Design of concrete structures*
- *EN 1993 Eurocode 3: Design of steel structures*
- *EN 1994 Eurocode 4: Design of composite steel and concrete structures*
- *EN 1995 Eurocode 5: Design of timber structures*
- *EN 1996 Eurocode 6: Design of masonry structures*
- *EN 1997 Eurocode 7: Geotechnical design*
- *EN 1998 Eurocode 8: Design of structures for earthquake resistance*
- *EN 1999 Eurocode 9: Design of aluminium structures*
- New parts are under development, e.g. Eurocode for design of structural glass.

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1997 Eurocode 7

EN 1997 consists of a number of parts:

- EN 1997-1, *Geotechnical design — Part 1: General rules*
- EN 1997-2, *Geotechnical design — Part 2: Ground properties*
- EN 1997-3, *Geotechnical design — Part 3: Geotechnical structures*

EN 1997 standards establish additional principles and requirements to those given in EN 1990 for the safety, serviceability, robustness, and durability of geotechnical structures.

Design and verification in EN 1997 (all parts) are based on the partial factor method or other reliability-based methods, prescriptive rules, testing, or the observational method.

0.3 Introduction to prEN 1997-3

This document establishes principles and requirements for the design and verification of the following of geotechnical structures, including temporary geotechnical structures: slopes, cuttings, embankments, shallow foundation, piled foundation and retaining structures.

This document establishes principles and requirements for the design and verification of supporting elements: anchors, reinforcing element in reinforced fill structures, soil nails, rock bolts and facing.

This document establishes principles and requirements for the design and verification of groundwater control including reduction of hydraulic conductivity, dewatering and infiltration, and the use of impermeable barriers

0.4 Verbal forms used in the Eurocodes

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may” expresses a course of action permissible within the limits of the Eurocodes.

The verb “can” expresses possibility and capability; it is used for statements of fact and clarification of concepts.

0.5 National Annex for prEN 1997-3

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing prEN 1997-3:2022 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this standard, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in prEN 1997-3:2022 through notes to the following:

Table 4.1 (NDP)	Table 4.2 (NDP)	Table 5.1 (NDP)	Table 5.2 (NDP)
Table 5.3 (NDP)	Table 6.1 (NDP)	Table 6.2 (NDP)	Table 6.3 (NDP)
Table 6.4 (NDP)	Table 6.5 (NDP)	Table 6.6 (NDP)	Table 6.7 (NDP)
Formula (6.18)	Table 7.1 (NDP)	Table 8.1 (NDP)	Table 8.2 (NDP)
Table 8.3 (NDP)	Table 9.1 (NDP)	Table 9.2 (NDP)	Table 9.3 (NDP)
Table 10.1 (NDP)	Table 10.2 (NDP)	Table 10.3 (NDP)	Table 10.4 (NDP)
Table 10.5 (NDP)	Table 11.1 (NDP)	Table 11.2 (NDP)	Table 11.3 (NDP)
Table 11.4 (NDP)	Table 11.5 (NDP)	Table 12.1 (NDP)	A.1(1) NOTE 1
G.1(1) NOTE 1			

National choice is allowed in prEN 1997-3:2022 on the application of the following informative annexes.

Annex A	Annex B	Annex C	Annex D
Annex E	Annex F	Annex G	

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

1 Scope

1.1 Scope of prEN 1997-3

(1) This document provides specific rules to be applied for design and verification of geotechnical structures.

1.2 Assumptions

(1) This document is intended to be used in conjunction with prEN 1990:2021, which establishes principles and requirements for the safety, serviceability, robustness, and durability of structures, including geotechnical structures, and other construction works.

(2) This document is intended to be used in conjunction with prEN 1997-1:2022, which provides general rules for design and verification of geotechnical structures.

(3) This document is intended to be used in conjunction with prEN 1997-2:2022, which gives provisions rules for determining ground properties from ground investigation.

(4) This document is intended to be used in conjunction with the other Eurocodes for the design of geotechnical structures, including temporary geotechnical structures.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in 'should' clauses), permissions ('may' clauses), possibilities ('can' clauses), and in notes.

EN 1537, *Execution of special geotechnical works — Ground anchors*

prEN 1990:2021, *Eurocode — Basis of structural and geotechnical design*

prEN 1992 (all parts), *Eurocode 2 — Design of concrete structures*

prEN 1993 (all parts), *Eurocode 3 — Design of steel structures*

prEN 1993-1-1:2022, *Eurocode 3 — Design of steel structures — Part 1-1: General rules and rules for buildings*

EN 1993-5:2007, *Eurocode 3 — Design of steel structures — Part 5: Piling*

prEN 1994 (all parts), *Eurocode 4 — Design of composite steel and concrete structures*

prEN 1995 (all parts), *Eurocode 5 — Design of timber structures*

prEN 1996 (all parts), *Eurocode 6 — Design of masonry structures*

prEN 1997-1:2022, *Eurocode 7 — Geotechnical design — Part 1: General rules*

prEN 1997-2:2022, *Eurocode 7 — Geotechnical design — Part 2: Ground properties*

EN 10025 (all parts), *Hot rolled products of structural steel*

EN 10080, *Steel for the reinforcement of concrete — Weldable reinforcing steel — General*

EN 10244-2:2009, *Steel wire and wire products — Non-ferrous metallic coatings on steel wire — Part 2: Zinc or zinc alloy coatings*

EN 10245-2, *Steel wire and wire products — Organic coatings on steel wire — Part 2: PVC finished wire*

EN 10245-3, *Steel wire and wire products — Organic coatings on steel wire — Part 3: PE coated wire*

EN 10245-4, *Steel wire and wire products — Organic coatings on steel wire — Part 4: Polyester coated wire*

EN 10245-5, *Steel wire and wire products — Organic coatings on steel wire — Part 5: Polyamide coated wire*

EN 13738, *Geotextiles and geotextile-related products — Determination of pullout resistance in soil*

EN 14475:2006, *Execution of special geotechnical works — Reinforced fill*

EN 14488-4, *Testing sprayed concrete — Part 4: Bond strength of cores by direct tension*

EN 14488-5, *Testing sprayed concrete — Part 5: Determination of energy absorption capacity of fibre reinforced slab specimens*

EN 14490, *Execution of special geotechnical works — Soil nailing*

EN ISO 1461, *Hot dip galvanized coatings on fabricated iron and steel articles — Specifications and test methods (ISO 1461)*

EN ISO 12957-1, *Geosynthetics — Determination of friction characteristics — Part 1: Direct shear test (ISO 12957-1)*

EN ISO 12957-2, *Geosynthetics — Determination of friction characteristics — Part 2: Inclined plane test (ISO 12957-2)*

EN ISO 10319, *Geosynthetics — Wide-width tensile test (ISO 10319)*

EN ISO 22477-5, *Geotechnical investigation and testing — Testing of geotechnical structures — Part 5: Testing of grouted anchors (ISO 22477-5)*

3 Terms, definitions, and symbols

3.1 Terms and definitions

For purposes of this document, the following terms and definitions apply.

3.1.1 Common terms used in prEN 1997-3

3.1.1.1
foundation
construction for transmitting forces to the supporting ground

[SOURCE: ISO 6707-1:2020]

3.1.1.2
deep foundation
foundation consisting of a pile or caisson that transfers loads below the surface stratum to a deeper stratum or series of strata at a range of depths

3.1.1.3
caisson
hollow construction with substantial impervious walls that comprises one or more cells and is sunk into the ground or water to form the permanent shell of a deep foundation

[SOURCE: ISO 6707-1:2020]

3.1.1.4
frost heave
swelling of soil due to formation of ice within it

[SOURCE: ISO 6707-1:2020]

3.1.1.5
ground heave
upward movement of the ground caused by either failure in the ground or by deformations due to stress relief, creep, or swelling

3.1.1.6
secondary compression
slow deformation of soil and rock mass because of prolonged pressure and stress; synonym for 'creep' in fine soils

3.1.1.7
competent rock
rock with sufficient strength and stiffness to withstand applied actions without failure or any significant permanent movement

3.1.2 Terms relating to slopes, cuttings, and embankments

3.1.2.1
earth-structure
civil engineering structure, made of fill material or as a result of excavation

3.1.2.2
cut
void that results from excavation of the ground

3.1.2.3
cutting
earth-structure created by excavation of the ground

3.1.2.4
cut slope
slope that results from excavation

3.1.2.5
embankment
earth-structure formed by the placement of fill

3.1.2.6
embankment slope
slope that results from the placement of fill

3.1.2.7
earthworks
civil engineering process that modifies the geometry of ground surface, by creating stable and durable earth-structures

3.1.2.8
excavation
result of removing material from the ground

3.1.2.9
levee
embankment for preventing flooding

3.1.2.10
load transfer platform
layer of coarse fill constructed with or without reinforcing element used to spread the load from an overlying structure such as a spread foundation, raft or embankment to improved ground or piles

3.1.3 Terms relating to spread foundations

3.1.3.1
spread foundation
foundation that transmits forces to the ground mainly by compression on its base

3.1.3.2
footing
stepped construction that spreads the load at the foot of a wall or column

[SOURCE: ISO 6707-1:2020]

3.1.3.3
pad foundation
spread foundation with usually rectangular or circular footprint

3.1.3.4
strip foundation
long, narrow, usually horizontal foundation

[SOURCE: ISO 6707-1:2020]

3.1.3.5
raft foundation
spread foundation in the form of a continuous structural concrete slab that extends over the whole base of a structure

[SOURCE: ISO 6707-1:2020]

3.1.3.6
adjusted elasticity method
method to evaluate the settlement of a spread foundation assuming the ground beneath the foundation is homogeneous and linear elastic

3.1.4 Terms relating to piled foundations

3.1.4.1
pile
slender structural member, substantially underground, intended to transmit forces into load-bearing strata below the surface of the ground

[SOURCE: ISO 6707-1:2020]

3.1.4.2
bored cast-in-place pile
bored pile formed by continuous or discontinuous earthwork methods where the hole is subsequently filled with concrete

[SOURCE: ISO 6707-1:2020]

3.1.4.3
displacement pile
pile which is installed in the ground without excavation of material from the ground, except for limiting heave, vibration, removal of obstructions, or to assist penetration

[SOURCE: ISO 6707-1:2020]

3.1.4.4
driven pile
displacement pile forced into the ground by hammering, vibration or static pressure

[SOURCE: modified from ISO 6707-1:2020]

3.1.4.5
end bearing pile
pile that transmits forces to the ground mainly by compression on its base

Note 1 to entry: The term 'mainly' implies at least 70 % to 80 % of the compression force applied to the pile is transmitted to the ground via its base.

[SOURCE: ISO 6707-1:2020]

3.1.4.6
friction pile
pile transmitting forces to the ground mainly by friction between the surface of the pile and the adjacent ground

Note 1 to entry: The term 'mainly' implies at least 70 % to 80 % of the compression or tension force applied to the pile is transmitted to the ground by friction between the pile shaft and the ground.

[SOURCE: ISO 6707-1:2020]

3.1.4.7
replacement pile
pile installed in the ground after excavation of material

3.1.4.8
tension pile
vertical or inclined pile used to transfer axial tension force by friction between the surface of the pile and the adjacent ground

3.1.4.9
pile cap
construction at the head of one or more piles that transmits forces from a structure to one or several piles

[SOURCE: ISO 6707-1:2020]

3.1.4.10
piled foundation
foundation that incorporates one or more piles

[SOURCE: ISO 6707-1:2020]

3.1.4.11
pile group
foundation that incorporates piles arranged in a grid

3.1.4.12
piled raft
combined foundation that incorporates a ground bearing raft foundation and a pile group

3.1.4.13
ground model method
calculation method to determine the pile axial resistance based on a Geotechnical Design Model comprising various strata with assigned ground parameters that can be ascribed to either the whole or part of the project site area

3.1.4.14
model pile method
calculation method to determine the pile axial resistance based on a single profile of field tests with assigned ground parameters relevant just to the local profile and not to the whole project site area

3.1.4.15
downdrag (negative shaft friction)
situation where the ground surrounding a pile settles more than the pile shaft sufficient to induce a downward drag force that potentially results in drag settlement

3.1.4.16
drag force
additional axial force acting on a pile due to downdrag

3.1.4.17
drag settlement
additional settlement of a pile due to downdrag

3.1.4.18
neutral plane
depth at which there is no relative movement between the pile and the surrounding ground

3.1.4.19
pile heave
upward movement of the ground surrounding a pile that can result in a heave force developing on the pile shaft, tension within the pile shaft, and upward movement of part or all of the pile

3.1.4.20
trial pile
pile that will not form part of the foundation, installed before the commencement of the piling works, and used to investigate the appropriateness of the chosen type of pile and method of execution and to confirm its design, dimensions, and resistance

3.1.4.21
working pile
pile that will form part of the foundation of the structure

3.1.4.22
test pile
trial pile or working pile to which loads are applied to determine the load-displacement behaviour of the pile and the surrounding ground at the time of construction

3.1.4.23
ultimate control test
load test carried out on a test pile to determine its resistance at the ultimate limit state

3.1.4.24
serviceability control test
load test carried out on a test pile to determine its load-displacement behaviour and resistance at the serviceability limit state

3.1.4.25
inspection test
test used to verify acceptance of a working pile

Note 1 to entry: Pile inspection tests include non-destructive integrity tests (to confirm the as-built condition, length, and cross-sectional area of the pile shaft) and concrete or grout tests (such as cube or cylinder strength tests to confirm that the pile materials comply with acceptance criteria).

3.1.4.26
integrity test
test carried out on an installed pile for the verification of soundness of materials and of the pile geometry

3.1.4.27
pile load
axial compressive, tensile, or transverse load (or force) applied to the head of the pile

3.1.4.28
pile test proof load
maximum proposed test load which includes imposed actions from the superstructure or the ground

3.1.4.29
temporary support load
load representing the temporary axial or transverse support from the ground to a pile under load test resulting from particular conditions of the test such as variations in groundwater, pile head level or pile head restraint that may reverse, reduce or change under service conditions

3.1.4.30
static load test
load test in which a single pile is subject to a series of static loads in order to define its load-displacement behaviour

[SOURCE: adapted from EN ISO 22477-1:2018]

3.1.4.31
dynamic load
axial compressive impact load (or force) applied to the head of a pile by a driving hammer or drop mass

[SOURCE: EN ISO 22477-4:2018, 3.1.5]

3.1.4.32
dynamic load test
test where a pile is subjected to chosen axial dynamic load at the pile head to allow the determination of its compressive resistance

[SOURCE: EN ISO 22477-4:2018, 3.1.7]

3.1.4.33
dynamic impact test
pile test with measurement of strain, acceleration and displacement versus time during the impact event

Note 1 to entry: Dynamic impact tests are often referred to as dynamic load tests

[SOURCE: EN ISO 22477-4:2018, 3.1.8]

3.1.4.34
rapid load test
pile load test where a pile is subjected to chosen axial rapid load at the pile head for the analysis of its capacity (compression resistance)

[SOURCE: EN ISO 22477-10:2016, 3.1.8]

3.1.4.35
bi-directional load test
static load test using an embedded jack where a section of the pile is used as reaction to load another section

Note 1 to entry: It is possible to install one or more levels of jacks in the pile shaft

3.1.4.36
ultimate resistance of a pile
corresponding state in which the piled foundation displaces significantly with negligible increase of resistance

3.1.4.37
driving formulae
formula that relates impact hammer energy and number of blows for a unit distance or permanent set for a single blow to pile compressive resistance

[SOURCE: EN ISO 22477-4:2018, 3.1.9]

3.1.4.38
wave equation analysis
analysis of a dynamically loaded pile by a mathematical model that can represent the dynamic behaviour of the pile by the progression of stress waves in the pile and the resulting response of the ground

[SOURCE: EN ISO 22477-4:2018, 3.1.10]

3.1.4.39
closed form solution
mathematical analysis of the dynamic load test data based on closed form wave analysis equations to derive a mobilised load

3.1.4.40
signal matching
numerical analysis to evaluate the shaft and base resistance of the test pile by modelling the pile and ground with assumed parameters to closely match the measured signals of pile head strain, displacement and acceleration obtained during a dynamic load test

[SOURCE: EN ISO 22477-4:2018, 3.1.11]

3.1.4.41
re-driving
process of re-initiating movement of a driven pile carried out some time after pile installation, used to check or determine any change in pile set or resistance

3.1.4.42
pile set
permanent pile settlement after one hammer impact blow during driving

3.1.4.43
pile set-up
time-dependent increase in pile resistance

3.1.5 Terms relating to retaining structures

3.1.5.1
retaining structure
structure that provides lateral support to the ground or that resists pressure from a mass of other material

3.1.5.2
gravity wall
retaining structure of stone or plain or reinforced concrete having a base footing with or without a heel, ledge or buttress

Note 1 to entry: The weight of the wall itself, sometimes including stabilizing masses of soil, rock or backfill, plays a dominant role in the support of the retained material.

3.1.5.3
embedded wall
relatively thin retaining structure of steel, reinforced concrete, or timber that is supported by anchors, struts or passive earth pressure

Note 1 to entry: The bending stiffness of such walls plays a significant role in the support of the retained material while the role of the weight of the wall is insignificant.

Note 2 to entry: This definition includes structures that do not reach below the final excavation level, even if they cannot formally be considered as embedded.

3.1.5.4
composite retaining structure
retaining structure composed of elements of gravity and embedded walls

Note 1 to entry: A large variety of such structures exists and examples include double sheet pile wall cofferdams, gabion walls, crib walls, earth structures reinforced by grouting.

Note 2 to entry: Earth structures reinforced by tendons, geotextiles, and structures with multiple rows of soil nails are considered as soil reinforcement (see 3.1.7).

3.1.5.5
combined wall
embedded wall composed of primary and secondary steel elements, placed in the ground before excavation begins

3.1.6 Terms relating to anchors

3.1.6.1
anchor
structural element capable of transmitting an applied tensile load from the anchor head through a free anchor length to a resisting element and finally into the ground

3.1.6.2
grouted anchor
anchor that uses a bonded length formed of cement grout, resin or similar material to transmit the tensile force to the ground

Note 1 to entry: A ‘grouted anchor’ in prEN 1997-3:2022 is termed a ‘ground anchor’ in EN 1537.

3.1.6.3
permanent anchor
anchor with a design service life which is in excess of two years

3.1.6.4
temporary anchor
anchor with a design service life of two years or less

3.1.6.5
tendon
part of an anchor that is capable of transmitting the tensile load from the anchor head to the resisting element in the ground

3.1.6.6
fixed anchor length
designed length of an anchor over which the load is transmitted to the surrounding ground through a resisting element

3.1.6.7
free anchor length
distance between the proximal end of the fixed anchor length and the tendon anchorage point at the anchor head

3.1.6.8
tendon bond length
(for grouted anchors only) length of the tendon that is bonded directly to the grout and capable of transmitting the applied tensile load

3.1.6.9
tendon free length
length of the tendon between the anchorage point at the anchor head and the proximal end of the tendon bond length

3.1.6.10
apparent tendon free length
(for grouted anchors only) length of tendon which is estimated to be fully decoupled from the surrounding grout and is determined from the load-elastic displacement data following testing

3.1.6.11
investigation test
load test to establish the geotechnical ultimate load resistance of an anchor at the interface of the resisting element and the ground and to determine the characteristics of the anchor in the working load range

[SOURCE: EN ISO 22477-5:2018, 3.1.6]

3.1.6.12
suitability test
load test to confirm that a particular anchor design will be adequate in particular ground conditions

[SOURCE: EN ISO 22477-5:2018, 3.1.9]

3.1.6.13
acceptance test
load test to confirm that an individual anchor conforms with its acceptance criteria

[SOURCE: EN ISO 22477-5:2018, 3.1.1]

3.1.6.14
lock-off load
load with which pre-stressible anchors are fixed to realise an active force to limit deformation

3.1.6.15
test method 1
test in which the anchor is loaded stepwise by one or more load cycles increasing from the datum load to the proof load

[SOURCE: EN ISO 22477-5:2018, Test Method 1]

3.1.6.16
test method 2
test in which the anchor is loaded stepwise by load cycles increasing from a datum load to the proof load

Note 1 to entry: At each load step the load loss in the anchor is measured during a fixed time period.

[SOURCE: EN ISO 22477-5:2018, Test Method 2]

3.1.6.17
test method 3
test in which the anchor is loaded in incremental steps from a datum load to a maximum load

Note 1 to entry: The displacement of the tendon end is measured under maintained load at each loading step.

[SOURCE: EN ISO 22477-5:2018, Test Method 3]

3.1.7 Terms relating to reinforced fill structures

3.1.7.1
reinforced fill structures
engineered fill incorporating discrete layers of soil reinforcement, generally placed horizontally, which are arranged between successive layers of fill during construction

3.1.7.2
soil nailed structures
engineered cut-faced or existing structures incorporating layers of soil reinforcements which are installed into the ground, usually at a sub-horizontal angle, and that mobilise resistance with the soil along their entire length

Note 1 to entry: They are typically arranged in rows. For cut-faced applications the rows are usually placed between successive passes of soil excavation in front of one face of the structure.

3.1.7.3
basal reinforcement to embankments
fill structures incorporating at their base level at least one layer of soil reinforcements, commonly used for fills founded on weak or soft soils and fills founded on inclusion networks, or for fills overbridging voids

3.1.7.4
soil veneer reinforcement
use of soil reinforcement to prevent the sliding of the cover soil layer over a landfill lining or cover system, or any other low friction interface

3.1.7.5
tie back wedge method
method of analysis of reinforced soil structures that follows basic design principles currently employed for classical or anchored retaining walls

3.1.7.6
coherent gravity method
method of analysis of reinforced soil structures based on the monitored behaviour of a large number of structures using inextensible reinforcements, corroborated by theoretical analysis

3.1.7.7
isochronous creep curves
load/strain creep curves plotted at fixed times (0,1 h, 1 h, 10 h, etc.)

Note 1 to entry: The load at which there is a specified difference in strain for a specified time interval can then be defined. The procedure how to generate the isochronous creep curves is given in ISO TR 20432.

3.1.7.8
equivalent constant in-soil temperature
temperature that causes, during one year, the same rate of reinforcing element degradation as the actual in-soil temperature variation at the location of the reinforcing element

3.1.8 Terms relating to ground reinforcing element

3.1.8.1
rock bolt
rock reinforcing element for stabilizing rock excavations, transferring load from the unstable exterior to the confined interior of the rock mass

3.1.8.2
rock anchor
rock reinforcing element capable of imposing a pre-tensile load via the anchor from the unstable exterior to the confined interior of the rock mass to enhance the shear capacity of potential slip surfaces inside the rock mass

Note 1 to entry: A rock anchor differs from a “regular” anchor, that it is not transmitting external loads into the ground from e.g. retaining walls, but to impose internal pretension load to stabilize the rock itself. Many of the anchor characteristics may be the same or similar, such as an anchor head, grouting, anchor length.

3.1.8.3
soil nails
soil reinforcing element to treat unstable natural soil slopes or as a construction technique that allows the safe over-steepening of soil slopes

3.1.8.4
sprayed concrete
concrete that is conveyed through a hose and pneumatically sprayed at high velocity onto a surface

3.1.8.5
wire mesh
arrangement of bidirectional interlocking metal wires with spaced, small openings between

3.1.8.6
facing element
modular precast panel embedding the connections for soil reinforcements

3.1.9 Terms relating to ground improvement

3.1.9.1
ground improvement
modification of the ground or its hydraulic conductivity in order to bring the effects of actions within ultimate and serviceability requirements

Note 1 to entry: Ground improvement can be achieved by reducing or increasing hydraulic conductivity, binding or densifying the ground, filling voids, or creating inclusions in the ground.

3.1.9.2
ground improvement zone
volume of ground within which ground improvement is installed and results in modified ground properties

3.1.9.3
inclusion
elements installed in the ground with defined geometry and material properties sufficiently different from the surrounding ground as to modify the distribution of load, stress and groundwater flow within the ground improvement zone

3.1.9.4
rigid inclusion
inclusions with higher stiffness and a measurable unconfined compressive strength

3.1.9.5
discrete ground improvement
ground improvement zone comprising inclusions created in the ground with properties differing from the surrounding ground

3.1.9.6
diffused ground improvement
ground improvement where the ground improvement zone is be modelled with a single set of parameters

3.1.9.7
structural connection
mechanical connection between the ground improvement and the structure, capable of transferring compressive, tensile, shear, and bending actions directly

3.1.9.8
contact
physical contact between the ground improvement and the structure, capable of transferring only compressive and limited shear loads

Note 1 to entry: The transferable shear load typically depends on the size of the compressive load and the activated friction.

3.1.9.9
load distribution
subdivision of the total load into the share transferred by the inclusion and the share transferred by the soil

Note 1 to entry: The load distribution is determined by calculation and is an integral part of the design of discrete ground improvement.

3.2 Symbols and abbreviations

The symbols in prEN 1997-1:2022 and the following apply to this document.

3.2.1 Latin upper-case letters

A	plan area of the foundation base; and
A	loss of metal (incl. zinc) per face over the first year (in reinforcement elements)
A'	effective foundation area (= $B' \times L'$)
A_0	initial cross-sectional area of steel reinforcement
$A_{0,con}$	initial cross-sectional area of steel reinforcement at a connection
A_b, A_s	cross sectional area of the pile base and shaft, respectively
$A'_{gs,d}$	design value of the effective adhesion between the ground and geosynthetic reinforcement (also covers apparent adhesion caused by interlocking mechanism)
A_r	reduced cross-sectional area of steel reinforcement, taking account of the maximum anticipated loss of steel during the design service life of the structure ($A_r = A_0 - \Delta_{Ar}$)
$A_{r,con}$	reduced cross-sectional area of steel reinforcement at a connection, taking account of the maximum anticipated loss of steel along the design service life of the structure ($A_{r,con} = A_{0,con} - \Delta A_{r,con}$)
A_{red}	plan area of the foundation base not including any area where there is no positive contact pressure between the foundation and the underlying ground
A_{ru}	reduced cross-sectional area of the reinforcing element at ultimate resistance, allowing for the effects of potential corrosion
A_{ry}	reduced cross-sectional area of the reinforcing element at yield, allowing for the effects of potential corrosion.
$A'_{sn,d}$	design value of the effective adhesion between the ground and a soil nail
$A'_{st,d}$	design value of the effective adhesion between the ground and steel reinforcement
B	foundation width (shorter dimension on plan); and
B	breadth of the reinforcing element
B'	effective foundation width
B_b, B_s	base and shaft width (for square piles), respectively
$B_{b,eq}$	equivalent pile base size equal to B_b (for square piles), D_b (for circular piles) or p/π (for other shaped piles)
B_{gi}	smaller plan dimension of a rectangle circumscribing the ground improvement zone, limited to the depth of the zone of influence (in ground improvement)

$B_{s,eq}$	equivalent pile shaft size equal to B_s (for square piles) or D_s (for circular piles)
C	Subgrade reaction modulus
C_a	Cohesive resistance along the slip surface of an active wedge
C_p	Cohesive resistance along the slip surface of a passive wedge
D	bar diameter
D	embedment depth
D_{add}	representative vertical or transverse temporary support force
D_b	base diameter (for circular piles) in pile foundations
D_{ds}	Diameter of depression at the surface
D_{rep}	representative drag force due to moving ground in pile foundations
D_d	design drag force due to moving ground in pile foundations
D_s	shaft diameter (for circular piles) in pile foundations
D_{supp}	representative vertical or transverse temporary support force
D_y	diameter of the void
E_dI	Flexural stiffness of the pile, design value
E_i	initial tangent modulus in at-rest conditions
E_{ur}	unloading-reloading modulus
$F_{ad,SLS}$	design value of the maximum anchor force, including the effect of lock off load, and sufficient to prevent a serviceability limit state in the supported structure
$F_{ak,SLS}$	characteristic value of the maximum anchor force, including the effect of lock-off load, sufficient to prevent a serviceability limit state in the supported structure
$F_{d,SLS}$	design value of an action to prevent a SLS
$F_{d,ULS}$	Design value of an action to prevent an ULS
F_{ax}	axial action applied to the pile
$F_{cd,SLS}$	design axial compression applied to the pile at the serviceability limit state, including potential down drag forces
$F_{d,group}$	design action applied to the pile group or piled raft
F_{td}	design axial tension applied to the pile
$F_{tr,d}$	design transverse force applied to the pile including an allowance for any potential

	transverse force due to moving ground
H	Maximum height of the embankment
H_e	height (depth) of the excavation
H_s	height of material above the geosynthetic layer
H_v	height above the void
I	second moment of area (geometric moment of inertia)
K	earth pressure coefficient averaging the pressure around the whole circumference, $K = (1 + K_0)/2$
K_0	at-rest earth pressure coefficient
K_a	active pressure coefficient
$K_{a\gamma}$, K_{aq}, K_{ac}	normal active earth pressure coefficients
$K_{ac,u}$	normal active earth pressure coefficients for undrained conditions
K_M	consequence factor applied to material properties
$K_{p\gamma}$, K_{pq}, K_{pc}	normal passive earth pressure coefficients
$K_{pc,u}$	normal passive earth pressure coefficients for undrained conditions
K_s	relative stiffness between the foundation and the ground
K_u	corrosion heterogeneity factor for ultimate (in reinforcement elements)
K_y	corrosion heterogeneity factor for yield (in reinforcement elements)
L	foundation length
L'	effective foundation length
L_{bd}	Buckling length, design value
L_{dd}	depth of the neutral plane corresponding to the point where the pile settlement equals the ground settlement
L_{ds}	total length of the reinforcing element along which direct shear stresses are mobilized
L_{int}	mobilized interface length
L_j	Length of the j^{th} layer of reinforcement
L_n	nail length

L_{po} total length of the reinforcing element beyond the failure surface (or line of maximum tension) where pull-out stresses are mobilized (for reinforcement elements)

L_{ps} total length of the length of the reinforcing element beyond the failure surface (or line of maximum tension) where punching shear stresses are mobilized

M1, M2, independent sets of partial factors on material

N component of the total action acting normal to the foundation base

N_a component of the total action acting normal to the slip surface of an active wedge

N_c non-dimensional bearing resistance factor

N_{cu} non-dimensional bearing resistance factor for undrained conditions

N_d design value of N

N'_d design value of the effective action acting normal to the foundation base

N_{rep} representative value of N

N_p component of the total action acting normal to the slip surface of a passive wedge

N_q non-dimensional bearing resistance factor for the influence of the overburden pressure

N_s shape factor depending on the length and the width of the excavation

N_γ non-dimensional bearing resistance factor for the influence of the ground's weight density

$N_{\gamma u}$ non-dimensional bearing resistance factor for the influence of the ground's weight density, for undrained conditions

P percentage of test results passing the required characteristic value (in ground improvement); and

P length of the perimeter of the reinforcing element

P_c critical creep load determined in Test Method 3

P_o lock-off load

P_P proof load

$R_{ad,SLS}$ design value of an anchor's geotechnical resistance at the serviceability limit state

$R_{ad,ULS}$ design value of an anchor's geotechnical resistance at the ultimate limit state

$R_{ak,SLS}$ characteristic value of an anchor's geotechnical resistance at the serviceability limit state

$R_{ak,ULS}$ characteristic value of an anchor's geotechnical resistance at the ultimate limit state

R_{am} measured value of an anchor's geotechnical resistance

$R_{am,SLS}$	measured value of an anchor's geotechnical resistance at the serviceability limit state
$R_{am,ULS}$	measured value of an anchor's geotechnical resistance at the ultimate limit state
$R_{am,\alpha,SLS}$	measured value of an anchor's geotechnical resistance complying with its serviceability limit state criterion α_{SLS}
$R_{am,\alpha,ULS}$	measured value of an anchor's geotechnical resistance complying with its ultimate limit state criterion α_{ULS}
$(R_{am,ULS})_{min}$	minimum value of $R_{am,ULS}$ in a number of tests
$(R_{am,SLS})_{min}$	minimum value of $R_{am,SLS}$ in a number of tests
R_b, R_s, R_{st}	resistance of pile base, shaft, and shaft in tension, respectively
$R_{b,rep}$	pile's representative base resistance in axial compression
$(R_{calc})_{mean}$	mean calculated pile resistance for a set of profiles of field test results
$(R_{calc})_{min}$	minimum calculated pile resistance for a set of profiles of field test results
R_c, R_t, R_{tr}	pile resistance to compression, tension, and transverse actions, respectively
$R_{c,rep}$	pile's representative total resistance in axial compression
$R_{d,group}$	design resistance of the pile group or piled raft
$R_{d,gs,int}$	design tensile strength of the interface with the geosynthetic reinforcing element
$R_{d,st,int}$	design tensile strength of the interface with a steel reinforcing element
$R_{d,sn,int}$	design tensile strength of the interface with a soil nail element
R_g	resistance of the ground supporting the load transfer platform in the net area between the columns mobilized at a settlement that is compatible with the settlement of the ground improvement system
$R_{k,com}$	characteristic resistance to direct shear of the reinforcing element
$R_{k,ds}$	characteristic tensile resistance of the connection (of the reinforcing element)
$R_{m,sn,pul}$	measured pull-out force
R_{pd}	design value of the resisting force caused by earth pressure on the foundation side
R_{Nd}	design bearing resistance normal to the base of a spread foundation
$R_{rep,po}$	representative pull-out resistance of the reinforcing element
$R_{rep,raft}$	representative ultimate vertical compressive resistance of the raft
$R_{ri,i}$	resistance of a rigid inclusion i, depending on its position within the group

$R_{s,rep}$	pile's representative shaft resistance (in axial compression)
$R_{\text{sys,rep}}$	representative value of the total resistance of the ground improvement system with rigid inclusions
R_{td}	design value of pile's design axial tensile resistance; and
R_{td}	design value of the tensile resistance of the structural elements of an anchor
$(R_{\text{test}})_{\text{mean}}$	mean calculated pile resistance measured in a set of load tests
$(R_{\text{test}})_{\text{min}}$	minimum calculated pile resistance measured in a set of load tests
$R_{\text{tr,d}}$	pile's design transverse resistance
$R_{\text{t,rep}}$	pile's representative axial tensile resistance
$R_{\text{t,rep,el}}$	representative tensile resistance of the reinforcing element
$R_{\text{x,d}}$	design resistance of a pile (where x = b, c, s, st, t, or tr, as above)
$R_{\text{x,m}}$	measured resistance of a pile (where x = b, c, s, st, t, or tr, as above)
$R_{\text{x,rep}}$	representative resistance of a pile (where x = b, c, s, st, t, or tr, as above)
S_{t}	sensitivity of fine soil
S_{v}	vertical spacing of the reinforcements
T	component of the total action acting transverse (parallel) to the foundation base; and
T	age of the structure (in reinforcement elements) in years
T_{d}	design value of T;
$T_{\text{f,j}}$	is the tensile force per meter width due to any horizontal loads
$T_{\text{gs,k}}$	characteristic tensile strength of geosynthetic reinforcement
T_{k}	characteristic tensile strength of the reinforcing element
$T_{\text{k,cr}}$	characteristic tensile strength of the reinforcing element allowing for creep and limiting elongation
$T_{\text{p,j}}$	tensile force per metre width due to the vertical loads of self-weight and surcharge
T_{rep}	representative value of T
$T_{\text{s,j}}$	is the tensile force per metre width due to any strip loading
T_{ven}	tensile force to hold the veneer system on the slope without water
V_{norm}	coefficient of variation based on a normal distribution of strength values

W wedge load

W_a wedge load of an active wedge

W_p wedge load of a passive wedge

W_s surcharge load

W_T vertical uniformly distributed (wedge) load on the reinforcement

W_v resultant vertical load excluding external strip loads on the layer of reinforcement

3.2.2 Latin lower-case letters

a adhesion between layers or of ground to a construction

a_d design value of the geometrical property

a_{nom} nominal value of the geometrical property

b base width of the embankment; and

b width of the strip element (in reinforcement elements)

b_c, b_q, b_γ factors accounting for base inclination

b_{cu} factor accounting for base inclination, undrained

b_{gs} width of reinforcement per unit width (b_{gs} = 1 for continuous sheets)

b_{st} width of strip reinforcement per unit width (b_{st} = 1 for grids)

$c_{min,dur}$ minimum concrete cover required for environmental conditions

c_u soil undrained shear strength

$c_{u,d}$ design undrained shear strength of the soft foundation soil (in reinforcing elements)

$c_{u,rep}$ representative undrained shear strength of the soft foundation soil (in reinforcing elements)

d_c, d_q, d_γ factors accounting for the depth of foundation embedment

d_{min} minimum depth of field investigation

d_s rock discontinuity spacing between a pair of immediately adjacent discontinuities

d_{cu} factor accounting for depth, undrained

e eccentricity of the applied or resultant action

e_{0d} maximum transversal deformation of the initial curvature over the buckling length, design value

e_B eccentricity of the applied load in the direction of B

e_d design eccentricity of the resultant action

e_j eccentricity of the resultant vertical load at the level of the j^{th} layer of reinforcement

e_L eccentricity of the applied load in the direction of L

e_z initial zinc thickness of coating (for steel reinforcement elements)

f_{ds} direct shear factor determined from direct shear tests or comparable experience (for reinforcing elements)

$f_{m,d}$ design value of the unconfined compressive strength of the improved ground

f_s reduction factor to allow for extrapolation uncertainty for given design service life

f_{uk} characteristic ultimate tensile strength of steel reinforcement

f_{yk} characteristic yield strength of steel reinforcement

g_c, g_q, g_γ factors accounting for ground inclination

h maximum depth or maximum height of a cutting, excavation or embankment

i load inclination factor; and

i numbering of strata with i from 1 to n

i_c, i_q, i_γ factors accounting for load inclination

i_{cu} load inclination factor, undrained

j index number of layers or strata with j from 1 to n

k subgrade modulus; and

k horizontal subgrade reaction coefficient

$k_{a\gamma}$, k_{aq}, k_{ac} inclined active earth pressure coefficients

$k_{ac,u}$ inclined active earth pressure coefficients for undrained conditions

$k_{p\gamma}$, k_{pq}, k_{pc} inclined passive earth pressure coefficients

$k_{pc,u}$ inclined passive earth pressure coefficients for undrained conditions

k_{cu} reduction factor on c_u

$k_n\{P\}$ acceptance value for the sample distribution in terms of P

k_{po} pull-out factor determined in laboratory pull-out tests in representative conditions, from comparable experience, or from field tests (for reinforcement elements)

k_{sn} soil nail (reinforcement element) pull-out factor determined from field pull-out tests or from comparable experience

$k_{\tan\varphi}$ reduction factor on $\tan\varphi$

k_δ constant depending on the roughness of the ground structure interface and local disturbance during installation: $k_\delta = a/c$

m exponent in bearing resistance formulae for the load inclination factor i

m_y mean of the measured values of $\log(q_{u,field})$ (in ground improvement)

n number of rigid inclusions; and

n exponent (factor covering reduction in corrosion rate in time for reinforcement elements)

p pile perimeter

p_0 total at-rest earth pressure

p'_0 effective at-rest earth pressure

p_a component of the total active earth pressure normal to the retaining wall face

p'_a component of the effective active earth pressure normal to the retaining wall face

$p_{a,min}$ minimum value of p_a to the retaining wall face

p_{fd} design value of the ultimate transversal ground resistance pressure

p_{group} smaller dimension of a rectangle circumscribing a group of piles

$p_{max,d}$ presumed maximum design bearing pressure

p_p component of the total passive earth pressure normal to the retaining wall face

p'_p component of the effective passive earth pressure normal to the retaining wall face

p_{ps} resistance to punching through the ground or fill (of a reinforcing element)

p_{re} perimeter of the reinforcing element

p_{sn} representative perimeter of the failure surface enclosing the soil nail per unit length , where pull-out resistance is mobilized

q overburden or vertical surcharge pressure at given level

q' effective overburden pressure at the level of the foundation base

q_o overburden pressure applied to the ground outside the foundation

q_a vertical surcharge applied at the ground surface (on the active side of the retaining wall)

q_b end bearing or base stress

$q_{m,sn,pul}$ measured interface unit strength

q_p permanent vertical surcharge applied at formation level (on the passive side of the retaining wall)

q_s surface load

$q_{s,i}$ shaft friction in the various strata i

q_{sk} characteristic skin friction along the soil nail (reinforcement element)

$q_{u,field}$ unconfined compressive strength measured in unconfined compressive tests on field samples

$q_{uk,imp}$ characteristic value of the unconfined compressive strength of the improved ground

$q_{u,rep}$ representative value of the unconfined compressive strength of the improved ground

s_0 settlement caused by undrained shear

s_1 settlement caused by consolidation

s_2 settlement caused by creep

s_c, s_q, s_γ factors accounting for the shape of the foundation base

s_{cu} factor accounting for shape, undrained

s_{ground} ground strata settlement profile (at any particular time)

s_p centre to centre spacing of the inclusions

s_{pile} pile settlement with depth

s_y standard deviation of the measured values of $\log(q_{u,field})$ (in ground improvement)

t time in days (since t_0)

t_0 time / date of installation or construction

u groundwater pressure at a point in the ground

u_a groundwater pressure acting at depth z on the active side of the retaining wall

w_s surcharge of the geosynthetic layer

x distance along the length of the reinforcing element

y transversal deflection of the pile

y_f relative deformation between the pile and the supporting soil where p_f is obtained

z depth below ground surface

z_a depth of zone of influence; and

z_a depth at the active side of the retaining wall

z_c depth of the foundation soil when the depth is limited and c_u is constant throughout

$z_{e,e}$ equivalent embedment depth

z_{emb} embedment depth of the foundation

z_f foundation depth (thickness)

z_p depth at the passive side of the retaining wall

z_w groundwater level at a depth

z_{zoi} depth of zone of influence

3.2.3 Greek upper-case letters

Δa deviation in a geometrical property

ΔA_r maximum anticipated loss of steel area during the design service life of the structure

ΔB is a width deviation

Δc_{dev} allowance in design for deviation of the concrete cover

Δe oss of thickness caused by corrosion in the ground

3.2.4 Greek lower-case letters

α angle of inclination of the foundation base to the horizontal; and

α angle of inclination of the surcharge

α_1 limit value of the creep rate in Test Method 1

α_3 limit value of the creep rate in Test Method 3

α_{ds} is a soil/reinforcement interaction coefficient for undrained conditions (for reinforcing elements)

α_{SLS} creep rate defining the geotechnical resistance of an anchor at the serviceability limit state (determined from the displacement per log cycle of time at constant anchor load as defined in EN ISO 22477-5)

α_{ULS} creep rate defining the geotechnical resistance of an anchor at the ultimate limit state (determined from the displacement per log cycle of time at constant anchor load as defined in EN ISO 22477-5)

β inclination of the ground surface

γ unit weight density of the ground

γ_a average weight density of the ground (on active side of the retaining wall) above depth z_a

$\gamma_{a,SLS}$ partial factor on an anchor's geotechnical resistance at the serviceability limit state

$\gamma_{a,SLS,test}$ partial factor on the anchor resistance at the serviceability limit state in acceptance tests

$\gamma_{a,ULS}$ partial factor on an anchor's geotechnical resistance at the ultimate limit state

γ'_d design effective weight density of the ground below the foundation level

γ_E partial factor on effect-of-actions

γ_F partial factor on actions

$\gamma_{F,drag}$ partial factor on a drag force due to moving ground in pile foundations

$\gamma_{F,SLS}$ partial factor on the anchor force at the serviceability limit state

γ_{gs} partial material factor for geosynthetic reinforcement

$\gamma_{gs,int}$ partial resistance factor on interface strength of geosynthetic reinforcement

$\gamma_{gs,d}$ design value of the effective angle of shearing resistance between the ground and geosynthetic reinforcement

γ_M partial material factor, applied to ground properties

γ_{M0}, γ_{M2} partial factors for steel (in reinforcing elements) whose values are specified in prEN 1993-1-1

$\gamma_{M,gs}$ partial factor for geosynthetic reinforcing elements

$\gamma_{M,pwm}$ partial factor for polymer steel woven wire mesh reinforcing elements

γ_p average weight density of the ground (on passive side of the retaining wall) above depth z_p

γ_R partial resistance factor, applied to ground resistance

γ_{Rb}, γ_{Rs} resistance factors in pile foundations

γ_{Rc} resistance factor for an individual pile axial compressive resistance

γ_{Rst} resistance factor

γ_{Rd} partial factor associated with the uncertainty of the resistance model / model factor in pile foundations; and

γ_{Rd} model factor accounting for additional uncertainty owing to extrapolation of measured strengths to the design service life (of reinforcing elements)

$\gamma_{Rd,0}$, $\gamma_{Rd,2}$ model factors that take account of the degree to which the strength of the steel reinforcing element is mobilized in a reinforced ground structure

$\gamma_{Rd,sys}$ model factor on overall system resistance

γ_{Re} passive earth resistance factor (on retaining walls)

γ_{Rst} resistance factor

$\gamma_{Rd,group}$ model factor for the pile group or piled raft

$\gamma_{R,group}$ resistance factor for the pile group axial compressive resistance

γ_{Rh} partial factor for sliding resistance

γ_{RN} partial factor for bearing resistance

$\gamma_{R,ds}$ partial factor to direct shear of the reinforcing element

$\gamma_{R,po}$ partial factor for pull-out resistance of the reinforcing element

$\gamma_{R,raft}$ resistance factor for the raft

γ_{Rst} partial factor of shaft resistance in pile foundations

$\gamma_{R,sys}$ partial resistance factor for the rigid inclusion system

γ_{RT} partial factor for sliding resistance

γ_{Rtr} partial factor of transversal resistance in pile foundations

γ_{SLS} partial factor for pile shaft resistance in the serviceability limit state

$\gamma_{\tan\varphi,cv}$ partial factor on the coefficient of internal friction of the ground under constant-volume conditions

$\gamma_{\tan\varphi,res}$ partial factor on the coefficient of friction of the ground along a residual slip surface

δ ground/structure interface friction angle; and

δ angle of inclination of the earth pressure

δ_d design value of δ

δ_{ep} angle of inclination of the earth pressure

δ_{rep} representative value of δ

ε_l limiting strain in the reinforcement

ε_r reinforcement strain

η_c conversion factor accounting for long term effects (in ground improvement)

η_{ch} conversion factor accounting for the adverse effects of chemical and biological degradation of the element at the design temperature

η_{con} conversion factor accounting for the reduction of resistance (of a reinforcing element) due to the connection

η_{cov} conversion factor allowing for the relationship between the log normal and normal characteristic strength based on field test results

η_{cr} conversion factor accounting for the adverse effect of tensile creep due to sustained static load over the design service life of the structure at the design temperature

η_{dmg} conversion factor accounting for the adverse effects of mechanical damage during execution

η_{dyn} conversion factor accounting for the adverse effects of intense and repeated loading over the design service life of the structure

$\eta_{el,con}$ conversion factor accounting for anticipated loss of strength with time and from other influences at the connection (with reinforcing elements)

η_{gs} conversion factor for geosynthetic reinforcement accounting for potential loss of strength with time and other influences

η_{pwm} conversion factor for reinforcement polymer steel woven wire mesh accounting for potential loss of strength with time and other influences

η_t conversion factor accounting for the difference in time between testing (typically 28 days) and when the improved ground is exposed to the designed stresses

η_w conversion factor accounting for the adverse effects of weathering

θ angle on plan between the L axis and the direction of T

λ inclination of the retaining wall

μ_{norm} mean normal strength of field samples

μ_{po} coefficient of interface friction determined in laboratory pull-out tests in representative conditions or from field tests (for reinforcement elements)

$\xi_{a,SLS,test}$ correlation factor for serviceability limit state verification taking account of the number of suitability tests

$\xi_{a,ULS,test}$ correlation factor for ultimate limit state verification taking account of the number of suitability tests

ξ_{mean} correlation factor for mean values / for the mean of the calculated values

ξ_{min} correlation factor for minimum values/ for the minimum of the calculated values

ξ_n correlation factor based on the number of tests and selected value of measured force

ξ_{sn} correlation factor accounting for the number of field pull-out tests performed or comparable experience (in reinforcement elements)

ξ_{ULS} correlation factor for ultimate limit state verification

σ'_n normal effective stress acting on the reinforcing element at the distance x

σ'_v effective vertical stress acting on the reinforcing element on the anchorage length

τ_{ds} resistance (in units of stress) against direct shear along the ground / grout / reinforcement interface(for reinforcing elements)

τ_n action effect of down drag (negative shaft friction)

$\tau_{n,rep}$ representative action effect of down drag (negative shaft friction)

τ_{po} representative shear resistance (in units of stress) against pull-out along the ground/grout/reinforcement interface (for reinforcing elements)

$\varphi_{cv,k}$ characteristic value of the angle of internal friction of the ground under constant-volume conditions

$\varphi_{res,k}$ characteristic value of the angle of friction of the ground along a residual slip surface

$\varphi_{sn,d}$ design value of the effective angle of shearing resistance between the ground and a soil nail

$\varphi'_{st,d}$ design value of the effective angle of shearing resistance between the ground and steel reinforcement

ω_α intermediate variable on the angle of inclination of the surcharge

ω_δ intermediate variable on the angle of inclination of the earth pressure

3.2.5 Abbreviations

AI, AII diffused ground improvement classes

BI, BII discrete ground improvement classes

CPT Cone Penetration Test

EFA Effects Factoring Approach

EI flexural stiffness product (bending stiffness)

GC Geotechnical Category

MFA Material Factor Approach

NDP	National Determined Parameter
OCR	overconsolidation ratio of the soil
PMT	Pressure meter Test
PWM	Polymer Steel Woven Wire Mesh
RFA	Resistance Factor Approach
SLS	Serviceability Limit State
SPT	Standard Penetration Test
ULS	Ultimate Limit State
VC	Verification Case
XA1 to XA3	Exposure classes for risk of chemical attack

4 Slopes, cuttings, and embankments

4.1 Scope and field of application

(1) This clause shall apply to cuttings, embankments and slopes within the zone of influence of construction works.

NOTE 1 Cuttings cover all type of transient and permanent excavations with an appointed design service life.

NOTE 2 EN 16907 (all parts) applies to the execution of earthworks projects (including cutting and embankments) and their planning.

(2) This clause shall apply to overall stability, local stability, and displacement of nearby structures and infrastructure within the zone of influence.

(3) This clause shall apply to dams and levees but excludes the verification of water retention of those structures.

NOTE The provisions in this clause do not entirely cover design rules needed for dams and levees classified in CC3 and CC4. For these structures additional provisions can be needed.

(4) This clause shall apply to the overall stability of the following geotechnical structures:

- retaining structures;
- ground reinforcing elements and improved ground structures;
- structures, infrastructure and foundation on or near slopes and cuttings; and
- existing slope within the zone of influence of planned construction works.

4.2 Basis of design

4.2.1 Design situations

(1) prEN 1997-1:2022, 4.2.2 shall apply to slopes, cuttings, and embankments.

4.2.2 Geometrical properties

4.2.2.1 General

(1) prEN 1997-1:2022, 4.3.3 shall apply to slopes, cuttings, and embankments.

4.2.3 Zone of influence

(1) prEN 1997-1:2022 4.1.2.1 shall apply to slopes, cuttings, and embankments.

4.2.4 Actions and environmental influences

4.2.4.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to slopes, cuttings, and embankments.

4.2.4.2 Permanent and variable actions

(1) Design situations involving long-term settlement and movement should include permanent and variable actions determined using the quasi-permanent combination of actions specified in prEN 1990:2021, 8.4.3.4.

(2) Design situation for cuttings shall include redistribution of initial in-situ stress due to excavation.

(3) Traffic load shall be included in the verifications of slopes, cuttings and embankments.

NOTE Guidance on traffic loads on geotechnical structures is given in prEN 1991-2:2022, 6.9 and prEN 1992-1-1:2021, 8.10

4.2.4.3 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to slopes, cuttings, and embankments.

4.2.4.4 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.4 shall apply to slopes, cuttings, and embankments.

4.2.5 Limit states

4.2.5.1 Ultimate Limit States

(1) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be verified for all slopes, cuttings, and embankments:

- loss of overall and local stability of the ground and structures within the zone of influence;
- failure due to gradual degradation of ground strength;
- failure along discontinuities;
- failure due to the impact of rock fall;
- loss of bearing resistance of embankments;
- structural failure of the face or surface of the slope, cutting or embankment and parts of it;
- structural failure of stabilizing measures;
- adverse hydraulic effects as a result of failure of drains, filters or seals;
- rapid drawdown of surface water levels causing excess pore water pressure;
- failure in ground caused by surface or internal erosion, or scour;

- structural failure in structures, roads, railway lines, or utilities due to movements in the ground in the zone of influence.

(2) Potential ultimate limit states other than those given in (1) should be verified.

4.2.5.2 Serviceability Limit States

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all slopes, cuttings, and embankments:

- settlement of embankments;
- horizontal ground movements of slopes, cuttings ad embankments;
- creep in soil and fill during the freezing and thawing period;
- loss of serviceability in neighbouring structures, roads or services due to movements in the ground or due to changes in groundwater conditions;
- deformation of the structure, which can cause serviceability limit states of existing nearby structures;
- movements in the ground due to shear deformations, settlement, vibration or heave; and
- accumulated ground movement or settlement due to creep.

NOTE Excavation below groundwater level can cause severe reduction in ground strength, hydraulic heave, groundwater flow, internal erosion, piping or surface erosion.

(2) Potential serviceability limit states other than those given in (1) should be verified.

4.2.6 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply for slope, cuttings, and embankments.

4.2.7 Ground investigation

4.2.7.1 General

(1) prEN 1997-2:2022, 5 shall apply for slope, cuttings, and embankments.

NOTE Specific ground investigations for earthworks are given in EN 16907-1 and EN 16907- 5.

4.2.7.2 Minimum extent of field investigation

(1) The depth and horizontal extent of the field investigation shall be sufficient to determine the ground conditions within the zone of influence in accordance with prEN 1997-1:2022, 4.1.2.1.

(2) The minimum depth of field investigation (d_{min}) should be determined as follows:

- For cuttings: 1.4 h (where h is the maximum depth of excavation);
- For embankments: 1.2 H or 1.0 B, whichever is the larger (where H is the maximum height of the embankment and B is its foundation width i.e. shorter dimension on plan);
- For embankments: at least, down to the bottom of the deepest fine soil layer (or layer of high compressibility) that could undergo consolidation settlement, depending on the depth of influence.

(3) If a layer of high strength is encountered, d_{min}, may be reduced to the depth corresponding to the top of that layer.

(4) Groundwater and piezometric levels shall be determined if they could influence the stability or settlement of the geotechnical structure or any adjacent structures or services.

4.2.8 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to slopes, cuttings, and embankments.

4.3 Materials

4.3.1 Ground properties

(1) prEN 1997-2:2022, 7-12 shall apply to slopes, cuttings, and embankments.

NOTE For fill properties see prEN 1997-1:2022, 5.

(2) Anisotropic properties should be determined if they have the potential to influence ground behaviour.

NOTE For example, anisotropic ground strength is of special importance for cuttings in fine soils due to the unloading and rotation of the principal stresses.

(3) Potential reduction in ground strength properties caused by exposure to weather conditions during or after execution should be considered.

NOTE Examples include desiccation and saturation of the ground and thawing of frozen ground.

(4) Slopes, cuttings, and embankments may be verified using effective stress or total stress ground properties.

(5) The determination of properties of discontinuities shall comply with prEN 1997-2:2022, 6.2.

(6) For unstable, slowly moving slopes, ground properties may be derived from back analyses using prEN 1997-1:2022, 4.3.2 (12) and prEN 1997-2:2022, 5.3.6.

4.3.2 Properties of improved ground

(1) The determination of the representative values of improved ground properties shall comply with Clause 11.

4.4 Groundwater

4.4.1 General

(1) prEN 1997-1:2022, 6 shall apply to slopes, cuttings, and embankments.

(2) Measures shall be taken to prevent the adverse effects of potential scour leading to erosion of soil around an earth-structure or internal erosion of soil within or around an earth structure.

(3) Groundwater pressure at interfaces and in discontinuities shall be determined.

(4) groundwater flow through interfaces and discontinuities shall be determined.

4.4.2 Groundwater control systems

(1) Groundwater control systems may be provided to ensure that design groundwater and piezometric pressures are not exceeded due to unforeseen circumstances.

NOTE 1 Guidance on verification of groundwater control systems is given in Clause 12.

NOTE 2 Examples of drainage for cuttings and embankments are given in EN 16907-1.

(2) If a groundwater control system is not provided, then the design shall be verified to withstand potential increase of groundwater pressures.

(3) It shall be verified that an Accidental Limit State is not exceeded if the groundwater control system fails.

(4) Where the safety and serviceability of the geotechnical structure depend on the successful performance of a groundwater control system, one or more of the following measures should be taken:

- inspection and maintenance of the system, which should be specified in the Maintenance Plan, see prEN 1997-1:2022, 5;
- installing a drainage system that will perform according to specification without maintenance; and
- installing a secondary (“backup”) system.

4.5 Geotechnical analysis

4.5.1 General

(1) prEN 1997-1:2022, 7 shall apply to slopes, cuttings, and embankments.

(2) In addition to prEN 1997-1:2022, 4.3.1, the design of slopes, cuttings, and embankments subject to cyclic and dynamic loading should consider the following:

- degradation of ground strength and stiffness;
- accumulated ground movement or settlement;
- build-up of excess groundwater pressures;
- amplification of loads or displacements owing to resonance; and
- potential liquefaction of the ground.

NOTE For seismic design see EN 1998-5.

(3) The resistance of pre-existing sliding surfaces should be determined using residual strength properties.

(4) If the reliability according to prEN 1990 is not obtained in the design verification, potential necessity of stabilizing measures shall be considered.

(5) When verifying overall stability, all potential failure mechanisms shall be verified.

4.5.2 Analysis of slopes and cuttings

4.5.2.1 Stability in soils and fills

(1) The stability of slopes shall be determined using at least one of the following calculation models:

- limit-equilibrium methods;
- numerical models according to prEN 1997-1:2022, 7.1.4;
- limit analysis.

NOTE 1 Calculation models for overall stability of soil and fill slopes are given in A.3.

NOTE 2 Calculation models for stability of rock slopes are given in Annex A.4.

(2) In layered soils with significant differences in shear strength or subjected to high external loads, the stability of both circular and non-circular failure surfaces intersecting the layers with the lowest shear strength shall be verified.

(3) When it is not obvious which condition (drained or undrained) governs overall stability in any particular geotechnical unit, a calculation using a combination of drained or undrained conditions should be used in which the most unfavourable combination of drainage conditions is chosen.

(4) The weight density of a geotechnical unit should be a superior (upper) value if it has an unfavourable effect on the stability of the slope, or an inferior (lower) value if it has a favourable effect.

(5) The stabilizing effect from capillary action in the unsaturated zone may be used in transient design situations, provided its effect can be verified by comparable experience, groundwater pressure measurements or monitoring.

NOTE The stabilizing effect is also referred to as apparent cohesion and can be significantly reduced with an increase or decrease in moisture content. A common approach is to assume zero groundwater pressure above the piezometric level.

(6) Potential development of tension cracks in cohesive soils shall be considered in the verification of limit state.

(7) Potential instability along soil-rock interfaces shall be considered in verification of limit state.

4.5.2.2 Stability in rock mass

(1) The verification of rock mass stability shall consider, but is not limited to:

- the rock excavation technique and sequence;
- damaging effects of excavation by blasting;
- influence of rock wedges within slopes and cuttings on the local stability;
- effect of possible local instability on the overall stability.

NOTE Calculation models for stability of rock slopes are given in A.4.

(2) The verification of limit states shall be based on geotechnical mapping and documentation of the rock conditions.

(3) Scaling of rock surfaces shall be specified into the design.

4.5.3 Analysis of embankments

(1) For the analysis of the stability of embankments, the rules given in 4.5.2.1 shall apply.

(2) Analysis of embankments should adopt strength and stiffness properties that have been determined at compatible strains for the different materials in the embankment and ground.

(3) Potential uplift due to buoyancy shall be considered as an Ultimate Limit State.

(4) Additional calculation models for bearing resistance and settlement analysis given in Clause 5 may be used to verify that embankments do not exceed limit states.

(5) For embankments on low strength fine soils and organic soils, resistance to punching failure and plastic extrusion failure of the underlying soil should be verified.

NOTE 1 A calculation model for extrusion resistance of reinforces embankments is give in F.4.

NOTE 2 Calculation models for embankments subject to punching shear are given in B.5.

4.5.4 Supporting elements

(1) In cases where a combined failure of supporting elements and the ground could occur, ground-structure interaction shall be considered allowing for the difference in strength and stiffness of the ground and that of the supporting element.

NOTE Cases include failure surfaces intersecting supporting elements such as walls, piles, anchors, discrete ground improvement, and reinforcement elements and walls.

(2) If supporting elements are used to increase overall stability, their structural resistance shall be verified for the combined effects of action from the ground and the structure for all relevant design situations.

(3) Supporting elements used to improve overall or local stability, bearing resistance, or settlement performance shall be verified in accordance with clauses 6-10.

NOTE Actions in the supporting elements can include axial forces, shear forces or bending moments depending on the types of interaction between the ground and the supporting elements.

(4) It shall be verified that the design resistance of the supporting element equals or exceeds the design effect of actions given by Formula (4.1):

$$E_{\mathrm{d}} = \max\left(F_{d,ULS};\ \gamma_F F_{d,SLS}\right) \tag{4.1}$$

where

$F_{\mathrm{d,ULS}}$ is the design value of the action that the supporting element shall provide to prevent an ultimate limit state of the slope, cutting or embankment;

$F_{\mathrm{d,SLS}}$ is the design value of the action that the supporting element shall provide to prevent a serviceability limit state of the slope, cutting or embankment;

γ_{F} is a factor to convert a SLS value into an ULS value (using DC4).

4.5.5 Ground displacement and settlement of embankments

(1) In addition to prEN 1997-1:2022, 4.3.1 potential ground displacement due to the following causes should be considered:

– change of stresses in the ground due self-weight or application and removal of external actions;
– change in groundwater conditions and corresponding groundwater pressures;
– ongoing creep;
– volume loss of soluble strata or due to internal erosion;
– shrinkage and swelling of ground due to change in water content;
– freeze and thaw effects; and
– presence of cavities in the ground.

(2) The following components of settlement should be considered for soils and fill beneath and within the embankment:

– immediate settlement;
– settlement caused by consolidation; and
– settlement caused by creep.

NOTE Consolidation and creep can occur simultaneously, particularly in thick soil layers of low hydraulic conductivity.

(3) Immediate settlement and settlement below an embankment during execution should be included in the calculation of total settlement if it affects the final structure or utilities.

(4) Settlement within and below the embankment after execution due to external actions, self-weight, or delayed compaction effects should be included in the total settlement.

4.6 Ultimate limit states

4.6.1 Verification by the partial factor method

(1) prEN 1997-1:2022, 4.4 shall apply for slopes, cuttings, and embankments.

4.6.2 Verification by prescriptive rules

(1) prEN 1997-1:2022, 4.5 shall apply for slopes, cuttings, and embankments.

4.6.3 Verification by testing

(1) prEN 1997-1:2022, 4.6 shall apply for slopes, cuttings, and embankments.

(1) Staged construction or trial embankments excavations or cuttings may be used to verify limit states.

4.6.4 Verification by the Observational Method

(1) prEN 1997-1:2022, 4.7 shall apply for slopes, cuttings, and embankments.

4.6.5 Partial factors

(1) Partial factors for the verification of slopes, cuttings, and embankments at the ultimate limit states shall be determined according to prEN 1997-1:2022, 4.4.1 using the Material Factor Approach.

NOTE 2 Values of the partial factors are given in Table 4.1 (NDP) for persistent and transient design situations and in Table 4.2 (NDP) for accidental design situations, unless the National Annex gives different values.

Table 4.1 — (NDP) Partial factors for the verification of ground resistance of slopes, cuttings, and embankments for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA)[a,b]
Overall stability	Actions and effects-of-actions	γ_F and γ_E	VC3
	Ground properties[c]	γ_M	M2[b]
Bearing resistance	see Clause 5		

a Values of the partial factors for Verification Case 3, (VC3) are given in prEN 1990 :2021 Annex A.

b Values of the partial factors for Sets M2 are given in prEN 1997-1:2022, 4.4.1.3.

c Also includes ground properties of Class AI ground improvement (Clause 11)

Table 4.2 — (NDP) Partial factors for the verification of ground resistance of slopes, cuttings, and embankments for accidental design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA)[a]
Overall stability	Actions and effects-of-actions	γ_F and γ_E	Not factored
	Ground properties[b]	γ_M	M2
Bearing resistance	see Clause 5		

a Values of the partial factors for Set M2 are given in prEN 1997-1:2022 Annex A.

b Also includes ground properties of Class AI ground improvement (Clause 11).

4.7 Serviceability limit states

4.7.1 General

(1) prEN 1997-1:2022, 9 shall apply to slopes, cuttings, and embankments.

(2) It shall be verified that deformation of the ground within the zone of influence of a slope, cutting, or embankment does not cause a serviceability limit state in nearby structures or civil engineering works.

(3) Serviceability limit states for embankments shall be verified for deformations caused by freezing and thawing.

4.7.2 Displacement of slopes and cuttings

(1) In accordance with prEN 1990:2021, 5.1(2), if there are no explicit serviceability criteria, then the verification of serviceability limit states of slopes may be omitted provided ultimate limit states are verified.

4.7.3 Settlement of embankments

(1) It shall be verified that differential settlement caused by the variability of ground stiffness and thickness does not cause a serviceability limit state to be exceeded.

(2) When verifying the settlement of an embankment, any decrease in effective stress in the ground should be considered.

4.8 Implementation of design

4.8.1 General

(1) prEN 1997-1:2022, 10 shall apply to slopes, cuttings, and embankments.

NOTE For earthworks see EN 16907-3.

4.8.2 Inspection

(1) prEN 1997-1:2022, 10.3 shall apply to slopes, cuttings, and embankments.

(2) Quality control of earthworks should comply with EN 16907-5.

4.8.3 Monitoring

4.8.3.1 General

(1) prEN 1997-1:2022, 10.4 shall apply to slopes, cuttings, and embankments.

(1) In addition to prEN 1997-1:2022, 10.4, a Monitoring Plan should be prepared for slopes, cuttings, and embankments in GC 2 and GC3 for the following situations:

- when existing slopes show permanently or repeatedly ongoing displacement;
- where the stability is sensitive to the groundwater pressure distribution in and beneath the embankment;
- when utilizing the stabilising effect from capillary action; and
- to measure effects on structures.

4.8.3.2 Monitoring of slopes and cuttings

(1) The Monitoring Plan for slopes and cuttings should include, but is not limited to, measurement of the following:

- horizontal and vertical ground displacements with time;
- groundwater levels or groundwater pressures with time as needed;
- location and geometrical properties of the sliding surface in a developed slide, to derive the ground strength parameters from back analysis for the design of remedial works; and
- displacement and visible damage of structures and infrastructures within the zone of influence.

4.8.3.3 Monitoring of embankments

(1) The Monitoring Plan for an embankment should include, but is not limited to, measurement of the following:

– groundwater pressure measurements during execution of embankments on fine soil and fill of high compressibility;
– settlement measurements for the whole or parts of the embankment, different soil layers, and nearby structures, roads, and services;
– measurements of horizontal displacements in the zone of influence;
– checks on strength and stiffness properties of fill during construction;
– chemical analyses before, during and after construction, if pollution control is required;
– if fine grained fill is used: groundwater pressure measurement within the body of the embankment during construction; and
– checks on hydraulic conductivity or grain sized distribution of fill material and of foundation soil during construction.

(2) When an embankment on fine soil of low strength is raised in layers, to avoid potential limit states, groundwater pressures within the zone of influence should be monitored to ensure that they have dissipated to a sufficient degree to prevent a limit state being exceeded, before the next layer is placed.

4.8.4 Maintenance

(1) prEN 1997-1:2022, 10.5 shall apply to slopes, cuttings and embankments.

(2) The Maintenance Plan should include, but is not limited to, the following:

– inspection and maintenance measures of erosion and scour protection, drainage systems and filters;
– allowable dredging or excavation levels;
– procedures for canal or reservoir emptying;
– reconstruction or remedial measures of existing slopes after failure or extensive deformation;
– allowable loads and other restrictions during maintenance work.

4.9 Testing

(1) prEN 1997-1:2022, 11 shall apply to slopes, cuttings, and embankments.

NOTE For earthworks see EN 16907-5.

4.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to slopes, cuttings, and embankments.

5 Spread foundations

5.1 Scope and field of application

(1) This clause shall apply to spread foundations, including pad, strip, raft foundations, unreinforced working platforms and load transfer platforms.

(2) This clause may be applied to deep foundations, including caissons, that behave as spread foundations.

5.2 Basis of design

5.2.1 Design situations

(1) In addition to prEN 1997-1:2022, 4.2., design situations for spread foundations should include the effect of o:

– t soluble, expansive, and collapsible soils;
– the particular features of rock; and
– of scour.

5.2.2 Geometrical properties

(1) prEN 1997-1:2022, 4.3.3 shall apply to spread foundations.

(2) The width of a spread foundation should be chosen considering setting out tolerances, working space requirements, and the dimensions of the structural member supported by the foundation.

(3) When choosing the embedment depth of a spread foundation, influences that could affect the resistance of the bearing stratum and the deformation behaviour of the foundation shall be considered.

NOTE Influences that can affect the resistance of the bearing stratum are given in B.3.

5.2.3 Zone of influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to spread foundations.

5.2.4 Actions and environmental influences

5.2.4.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to spread foundations.

5.2.4.2 Permanent and variable actions

(1) Actions for spread foundation shall include but are not limited:

– imposed actions from the structure;
– the self-weight of the foundation;
– the weight of any backfill placed on the foundation;
– favourable and unfavourable earth pressures acting on the foundation, where significant;
– loading due to lateral or vertical ground displacements;
– actions due to frost, including frost heave, thaw settlement, and thaw weakening of the ground;
– actions due to the swelling in soils with high expansion potential;
– actions due to the collapse of ground;
– actions due to heating of the ground causing a reduction in the groundwater content and ground movements;
– actions due to the swelling of desiccated ground by the restoration of groundwater;
– actions due to seasonal drying and wetting cycles;
– changes in geometrical and geotechnical properties during the structure’s design service life due to anticipated nearby excavations for the replacement of pipes, cables, and drainage;
– actions due to adjacent building; and
– accidental actions.

(2) The adverse effects of actions on a spread foundation due to planned construction of adjacent structures and nearby excavations should be considered.

(3) Hazards due to changes in the volume of the ground shall be identified.

NOTE Examples of risks are active soils, swelling, shrinking and heave.

(4) In grounds with high expansion potential, measures shall be taken to avoid swelling during execution of a spread foundation.

(5) Spread foundations should be designed to accommodate any potential volumetric changes in the ground caused by a change in water content.

NOTE For example, due to the presence or removal of nearby trees or other vegetation or the presence of expansive clays.

(6) For raft and slabs foundation of larger extent, an analysis of the interaction between the supported structure and the ground should be performed to determine the distribution of actions on the spread foundation.

(7) Actions on the foundation may be determined by an analysis of ground structure interaction based on an equivalent spring model of the ground.

NOTE Formula for linear elastic spring stiffnesses are given in B.15.

5.2.4.3 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to spread foundations.

(2) The design of foundations subjected to cyclic and dynamic loading should consider the following:

- occurrence of vibrations that can affect the structure, surrounding structures, people or sensitive machinery;
- degradation of ground strength and potential liquefaction of foundation soil (leading to ultimate limit states being exceeded at loads below those expected from verifications based on static strength);
- changes in the ground hydraulic conductivity;
- large eccentricity leading to smaller effective foundation area and reduced bearing resistance;
- degradation of ground stiffness, leading to an accumulation of permanent foundation displacement;
- damping of vibrations in the ground beneath the structure;
- amplification of loads or movements owing to resonance; and
- potential surface wave issues due to dynamic loading.

5.2.4.4 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.5 shall apply to spread foundations

(2) Measures shall be taken to avoid frost impact on ground during execution.

(3) Testing to determine the frost susceptibility of ground shall comply with prEN 1997-2:2022, 12.1.

(4) Structural damage due to frost in frost susceptible ground may be prevented by adopting one or more of the following measures:

– setting the foundation level beneath the depth of frost penetration; or
– providing insulation to prevent frost.

(5) Insulation to prevent frost should comply with EN ISO 13793.

(6) An alternative to EN ISO 13793 may be used, when specified by the relevant authority or, where not specified, agreed for the specific project by the relevant parties.

(7) The potential of low temperatures due to ground freezing causing deformations of the foundation elements shall be considered in the presence of frost susceptible ground.

NOTE This particularly applies to thin raft foundations, including during execution.

(8) The adverse effects of frost action caused by construction work or by ground freezing should be considered.

(9) Measures shall be taken to avoid structural damage due to drying and wetting cycles of the ground caused by the change of climatic conditions during service life.

(10) Measures shall be provided to prevent the adverse effects of potential scour leading to erosion of soil under and around a spread foundation.

5.2.5 Limit states

5.2.5.1 Ultimate limit states

(1) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be verified for all spread foundations:

– bearing failure;
– sliding failure;
– rotational failure;
– shear and tensile failure of possible ground-foundation reinforcement elements;
– structural failure due to excessive foundation movement; and
– excessive heave due to swelling, frost, or other causes.

(2) Potential ultimate limit states other than those given in (1) should be verified.

5.2.5.2 Serviceability limit states

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all spread foundations:

– settlement;
– heave;
– rotation and tilting; and
– horizontal displacement.

(2) Potential serviceability limit states other than those given in (1) should be verified.

5.2.6 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply to spread foundations.

5.2.7 Ground investigation

5.2.7.1 General

(1) prEN 1997-2:2022, 5 shall apply to spread foundations.

5.2.7.2 Minimum extent of field investigation

(1) The depth and horizontal extent of the field investigation shall be sufficient to determine the ground conditions within the zone of influence of the structure according to prEN 1997-1:2022, 4.2.1.1.

(2) For low-rise structures in Geotechnical Category 1, the minimum depth of investigation below the planned base of an isolated spread foundation should be d_{min} = 2 m.

(3) For low-rise structures in Geotechnical Category 2, the minimum depth of investigation below the planned base of an isolated spread foundation d_{min} should comply with Formula (5.1):

$$d_{\mathrm{min}} \geq \max(3b_{\mathrm{F}}; 3m) \tag{5.1}$$

where

b_{F} is the smaller side length of the foundation (on plan) shown in Figure 5.1a.

(4) For high-rise structures, the minimum depth of investigation below the planned base of a spread foundation d_{min} should comply with Formula (5.2):

$$d_{\mathrm{min}} \geq \max(3b_{\mathrm{B}}; 6m) \tag{5.2}$$

where

b_{B} is the smaller side length of the foundation (on plan) shown in Figure 5.1b.

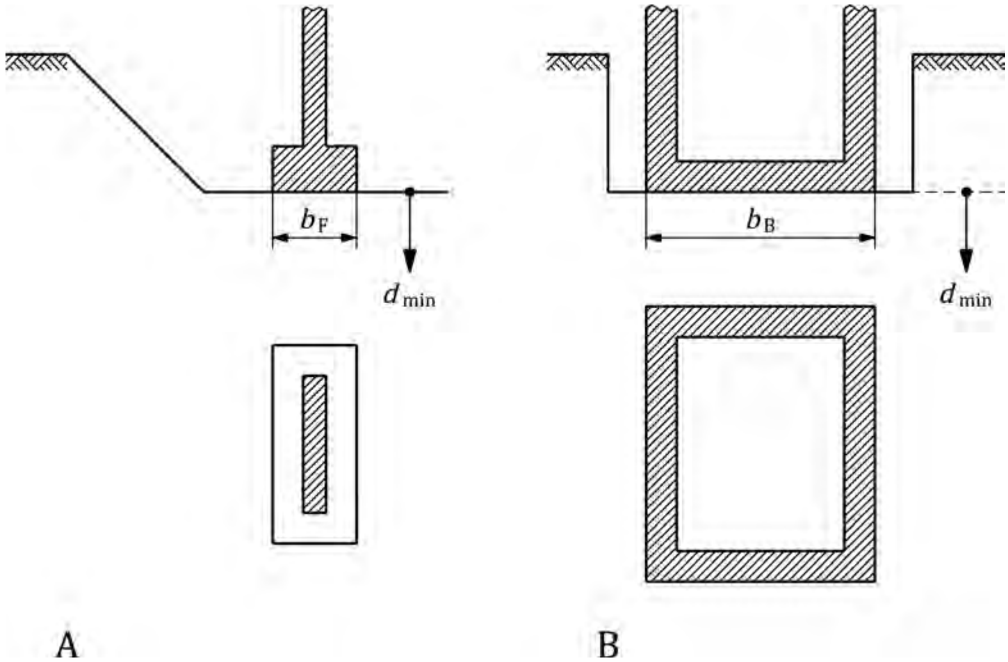
(5) For raft foundations and structures with several foundation elements whose effects in deeper strata are superimposed on each other, the minimum depth of investigation (d_{min}) below the planned base of the foundation should be determined based on the expected zone of influence unless a ground layer of high bearing resistance and sufficient thickness is identified at a shallower depth.

NOTE Minimum depth of investigation is defined in Figure 5.1.

(6) The minimum depth of investigation may be reduced in medium strong rock masses and stiff rock mass, moraine and strongly over consolidated clays provided there is comparable experience to allow the properties of the ground to be predicted up to the depth given by Formula (5.1) and Formula (5.2).

(7) Greater investigation depths should be selected when:

– unfavourable geotechnical conditions, including potential weak or compressible layers below layers with higher bearing resistance or discontinuities;
– unstable ground or groundwater conditions are anticipated; and
– the project involves raising or lowering the ground level.



Key

A foundation

B structure

Figure 5.1 — Definition of d_{min} for spread foundations

5.2.8 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to spread foundations.

5.3 Materials

5.3.1 Ground properties

(1) prEN 1997-2:2022, clause 7 to 12 shall apply to spread foundations.

NOTE For engineered fills see prEN 1997-1:2022, 5.2.

(1) Spread foundations may be verified using effective or total stress properties depending on the permeability of the ground, potential failure mechanisms, and the rate and duration of loading.

5.3.2 Plain and reinforced concrete

(1) prEN 1997-1:2022, 5.5 shall apply to spread foundations.

5.4 Groundwater

5.4.1 General

(1) prEN 1997-1:2022, 6 shall apply to spread foundations.

(2) Groundwater levels and pressures (including potential changes in them) that could affect the bearing resistance, sliding resistance, stability against uplift and loss of equilibrium, and settlement shall be considered in the verification of limit states.

(3) Increased groundwater levels and pressures owing to burst pipes and other failures of engineered systems involving water around a foundation may be classified as accidental actions.

(4) Surface water, groundwater and piezometric levels shall comply with prEN 1997-1:2022, 6.2, and prEN 1997-2:2022, 11.

(5) Where the groundwater level is close to the foundation level, the effects of capillary rise causing deterioration of foundation materials should be considered.

NOTE Capillary rise can be avoided by including waterproofing membranes or a capillary break soil layer.

5.4.2 Groundwater control systems

(1) Clause 12 shall apply to spread foundations.

(2) If ponding of water above a spread foundation reduces its robustness against the occurrence of a limit state below an acceptable level, drainage systems should be provided to remove the surface water or structural measures implemented to prevent ponding.

(3) Where the safety and serviceability of a spread foundation depend on the successful performance of a groundwater control system, one or more of the following measures should be taken:

- a Maintenance Plan should be specified (see prEN 1997-1:2022, 10.5);
- a groundwater control system should be specified that perform according to the specifications without maintenance; and
- a secondary (“backup”) system should be specified that prevent any potential leakage from entering the ground beneath or next to the structure.

NOTE An example of a secondary system is a pipe or channel that encloses the primary system.

5.5 Geotechnical analysis

5.5.1 General

(1) prEN 1997-1:2022, 7 shall apply to spread foundations.

(2) When verifying a spread foundation against ultimate or serviceability limit states, the effect of adjacent foundations on the loading, resistance and movement of the foundation should be considered.

(3) In addition to (2), the effect of the spread foundation on nearby foundations, structures, and services should be considered.

(4) The calculation models given in 5.5.2.1 and 5.5.2.2 may be used to verify limit states for spread foundations on soil or fill.

NOTE Guidance on calculation models is given in B.4 to B.12.

(5) The calculation models given in 5.5.2.3 may be used to verify limit states for spread foundations on rock.

(6) Calculation models used to verify the bearing resistance of a spread foundation should account for the following:

- the failure mechanism (general shear, local shear, punching shear, or squeezing failure);
- the strength of the ground;
- the variability of the ground, especially layering;
- discontinuities and weakness zones in a rock mass or in hard soils;
- the shape, depth, and inclination of the foundation;
- groundwater pressures;
- the inclination of the ground surface;
- the eccentricity and inclination of the loads; and
- the presence of cyclic or dynamic loads.

5.5.2 Bearing resistance

5.5.2.1 Bearing resistance from soil and fill parameters

(1) Provided that the undrained strength of the ground is assumed constant within the zone of influence, the undrained bearing resistance (R_{Nu}) of a spread foundation on soil or fill to a force acting normal to the base may be determined using total stress analysis from Formula (5.3):

$$R_{\mathrm{Nu}} = A'(c_{\mathrm{u}} N_{\mathrm{cu}} b_{\mathrm{cu}} d_{\mathrm{cu}} g_{\mathrm{cu}} i_{\mathrm{cu}} s_{\mathrm{cu}} + q_o) \quad (5.3)$$

where

A'	is the effective plan area of the foundation, see (3) and (4);
c_u	is the soils undrained shear strength;
N_{cu}	is a non-dimensional bearing resistance factor for undrained conditions, see B.4;
q_o	is the overburden pressure applied to the ground outside the foundation;
b_{cu}, d_{cu}, g_{cu}, i_{cu}, and s_{cu},	are non-dimensional factors to account for the effects of base inclination, embedment depth and resistance above the base of the foundation, ground surface inclination, load inclination, and foundation shape.

NOTE 1 Formula for N_{cu}, b_{cu}, d_{cu}, g_{cu}, i_{cu}, s_{cu}, and $N_{\gamma u}$ are given in Annex B.4(1) and (3).

NOTE 2 When the ground surface slopes downwards away from the foundation, it is possible to add a third term ($0.5\ \gamma B' N_{\gamma u}$) in Formula (5.3), being γ the weight density of the ground below the base of the foundation; B' the effective foundation width shown in Figure 5.2; and $N_{\gamma u}$ a non-dimensional bearing resistance factor for the influence of the ground's weight density with negative value in this case.

(2) The effective plan area of a rectangular foundation (A') in Formula (5.3) should be determined from Formula (5.4), assuming an uniform stress distribution:

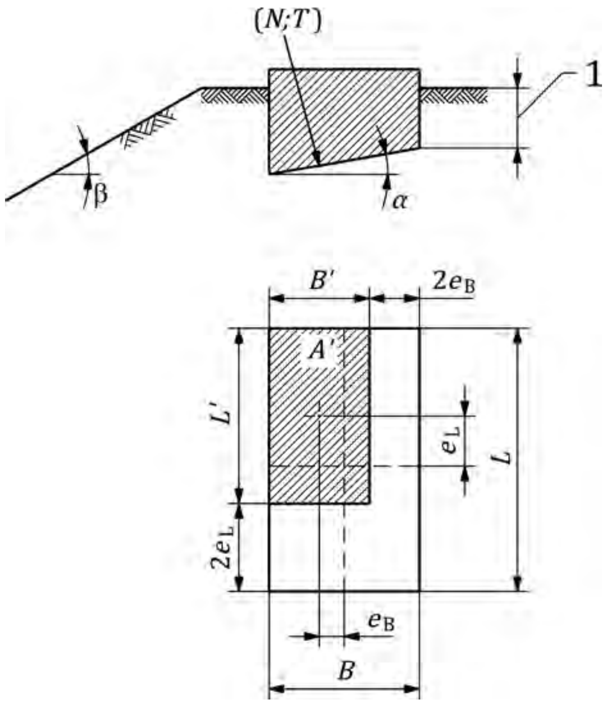
$$A' = B' \times L' = (B - 2e_{\mathrm{B}})(L - 2e_{\mathrm{L}}) \quad (5.4)$$

where

B'	is the effective foundation width;
L'	is the effective foundation length;
B	is the actual foundation width;
L	is the actual foundation length;
e_B	is the eccentricity of the applied load in the direction of B;

e_L is the eccentricity of the applied load in the direction of L.

NOTE The notation used in Formula (5.4)is illustrated in Figure 5.2.



Key

1 Embankment depth

N Component of the total action acting normal to the foundation base

T Component of the total action acting transverse (parallel) to the foundation base

α Angle of foundation base

B Actual foundation width

B' Effective foundation width

L Actual foundation length

L' Effective foundation length

A' Effective plan area of a rectangular foundation

e_B Eccentricity of the applied load in the direction of B

e_L Eccentricity of the applied load in the direction of L

β Sloping down angle of the ground [ω to be adjusted in the Figure]

Figure 5.2 — Notation for a rectangular spread foundation with an inclined base and eccentric load

(3) The effective plan area (A') of a circular foundation for use in Formula (5.3) should be determined from Formulae (5.5) and (5.6):

$$A' = B'_{eq} \times L'_{eq} = \frac{D^2}{2}\left(\cos^{-1}\left(\frac{2e}{D}\right) - \frac{2e}{D}\sqrt{1-\left(\frac{2e}{D}\right)^2}\right) \quad (5.5)$$

$$\frac{B'_{eq}}{L'_{eq}} = \sqrt{\frac{D-2e}{D+2e}} \tag{5.6}$$

where

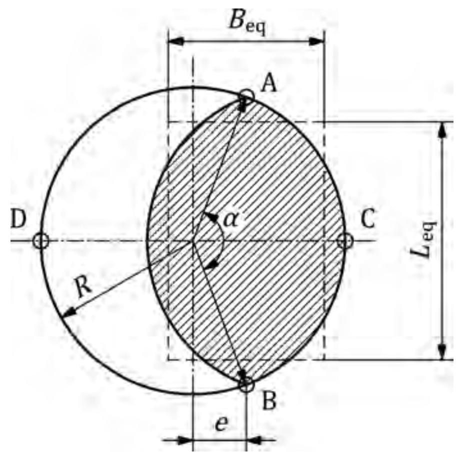
B'_{eq} is the effective width of the equivalent rectangular foundation area;

L'_{eq} is the effective length of the equivalent rectangular foundation area;

D is the diameter of the circular foundation;

e is the eccentricity of the applied action.

NOTE The notation used in Formulae (5.5) and (5.6) is illustrated in Figure 5.3.



Key

B_{eq}	effective width of the equivalent rectangular foundation area
L_{eq}	effective length of the equivalent rectangular foundation area
e	eccentricity of the applied action
α	
R	radius of the circular foundation
A, B, C, D	

Figure 5.3 — Notation for a circular spread foundation with an inclined base and eccentric load

(4) The drained bearing resistance (R_N) of a spread foundation on soil or fill to a force acting normal to the base may be determined using effective stress analysis from Formula (5.7):

$$R_N = A'\left(c'N_c b_c d_c g_c i_c s_c + q'N_q b_q d_q g_q i_q s_q + 0.5\gamma' B' N_\gamma b_\gamma d_\gamma g_\gamma i_\gamma s_\gamma\right) \tag{5.7}$$

where:

A' is the effective plan area of the foundation;

B' is the effective foundation width shown in Figure 5.2;

c' is the soil effective cohesion;

q' is the effective overburden pressure in ground outside the foundation base at the level of the base;

γ' is the buoyant weight density of the ground beneath the foundation;

N_c, N_q, N_γ are non-dimensional bearing resistance factors;

b_c, b_q, b_γ are non-dimensional factors accounting for base inclination;

d_c, d_q, d_γ are non-dimensional factors accounting for the depth of foundation embedment;

g_c, g_q, g_γ are non-dimensional factors accounting for ground surface inclination;

i_c, i_q, i_γ are non-dimensional factors accounting for load inclination;

s_c, s_q, s_γ are non-dimensional factors accounting for foundation base shape.

NOTE 1 Formulae for N_c, N_q, etc. are provided in B.4(4) and B.4(6).

NOTE 2 Guidance is given in B.4(7) to account for the effect of groundwater level on groundwater pressure and buoyant weight density.

(5) Formula (5.7) should only be used in uniform soil or fill or in layered ground where the shear strength properties do not differ by more than 5 % between the layers in the zone of influence for bearing resistance failure.

(6) When calculating the bearing resistance of a foundation on layered ground in which shear strength properties differ by more than 5 % between layers, weighted average values of soil or fill parameters within the zone of influence of the foundation may be used.

NOTE In layered grounds the rupture mechanism can differ from those implied by the adoption of Formula (5.7).

(7) The q term in Formula (5.3) and Formula (5.7) shall be reduced if overburden is potentially removed during the design service life of the foundation.

(8) A value of $d_{cu} > 1.0$ in Formula (5.3) or $d_c > 1.0$ in Formula (5.7) should only be used when the strength of soil or fill above the foundation depth D is equal to or greater than the strength of the soil at foundation level; otherwise $d_{cu} = 1$ or $d_c = 1$.

(9) Where soil or fill beneath a spread foundation has a definite structural pattern of layering or other discontinuities, the assumed rupture mechanism and the selected shear strength and deformation parameters shall consider the characteristics of the layering and discontinuities.

(10) Where a weaker geotechnical unit underlies a stronger unit, including a granular layer forming a working platform foundation, the rupture mechanisms that should be considered depend on the relative thickness of the stronger layer to the foundation width and should include:

- bearing resistance failure in the upper geotechnical unit;
- punching failure through the upper unit and bearing resistance failure in the lower unit; and
- squeezing or extrusion failure in the lower unit.

NOTE Calculation models for punching failure of a spread foundation on a stronger geotechnical unit over a weaker unit are given in B.5.

(11) Soil reinforcement may be placed on a weak geotechnical unit under a spread foundation supporting an inclined force, or under a stronger unit supporting a working platform, to resist the horizontal component of the force.

(12) When soil reinforcement is used to improve the stability of a spread foundation close to sloping ground, verification of overall stability shall comply with Clause 4.

(13) When analytical models cannot accommodate or do not adequately represent the design situations described in (11) and (12), numerical models should be used instead to determine the most unfavourable failure mechanism (see prEN 1997-1:2022, 8.2).

5.5.2.2 Bearing resistance and settlement from empirical models

(1) An empirical calculation model may be used to verify bearing resistance of spread foundations, provided there is comparable experience of its successful use.

(2) The bearing resistance and settlement of a spread foundation on soil may be determined from the results of field investigations and calculation models.

NOTE Empirical calculation models for the bearing resistance and settlement of a spread foundation are given in Annex B.

5.5.2.3 Bearing resistance of rocks

(1) The bearing resistance of a spread foundations on a discontinuous rock mass shall comply with prEN 1997-2:2022, 8.1.

NOTE Mechanisms for bearing resistance of a spread foundation on discontinuous rock can include planer sliding, wedge sliding and toppling.

5.5.2.4 Bearing pressures for structural analysis

(1) The bearing pressure beneath a rigid foundation may be assumed to be distributed linearly when determining bending moments and shear forces in the structural member.

(2) The distribution of bearing pressure beneath a flexible foundation shall consider the stiffness of the foundation and the supported structure.

(3) The distribution of bearing pressure beneath a flexible foundation may be derived by modelling the foundation as a beam or raft resting on a deforming continuum or series of springs, with appropriate stiffness and strength, to determine the bending moments and shear forces.

NOTE 1 Formulae for the relative stiffness of a spread foundation on elastic ground and for subgrade modulus are provided in B.14.

NOTE 2 A method for determining whether a foundation is rigid or flexible on the basis of the relative stiffness value is given in B.14.

NOTE 3 For spread foundations, calculations based on uniform spring stiffness do not provide realistic estimations of deformations due to edge effects.

5.5.3 Sliding resistance

(1) The resistance of a spread foundation to sliding may be determined as the sum of the resistance to sliding on its base plus any resistance to sliding caused by earth pressure on the face of the foundation.

(2) The resistance from earth pressure on the face of the foundation $R_{T,face}$ shall be determined considering the deformation compatibility with the sliding resistances.

(3) Where a spread foundation is constructed on a lean concrete blinding layer or includes a waterproof membrane, failure occurring along a plane weaker than that between the foundation base and the underlying ground shall be considered.

(4) The undrained sliding resistance along the base of a spread foundation ($R_{Tu,base}$) on soil or fill may be determined using total stress analyses from Formula (5.8):

$$R_{\text{Tu,base}} = A_{\text{red}} k_{\text{cu}} c_{\text{u}} \quad (5.8)$$

where

A_{red} is the plan area of the foundation base, not including any area where there is no positive contact pressure between the foundation and the underlying ground as a result of load eccentricity, ground shrinkage, or any other cause;

k_{cu} is a reduction factor depending on the foundation material, execution method, and soil or fill disturbance;

c_u is the soil undrained shear strength.

(5) For spread foundations made of concrete cast directly against soil or fill, the value of k_{cu} should be taken as 1.0 if the base is rough or ridged; or as 2/3 if the base is smooth.

(6) For spread foundations made of pre-cast concrete, the value of k_{cu} should be taken as 2/3.

(7) The drained sliding resistance along the base of a spread foundation ($R_{T,base}$) on soil or fill may be determined using effective stress analysis from Formula (5.9):

$$R_{\text{T,base}} = (N - U) \quad \tan\delta \quad (5.9)$$

where

N' is the normal component of the resulting force acting on the foundation base;

U is the uplift force due to groundwater pressures on the foundation base;

$\tan \delta$ is the coefficient of friction between the foundation and the ground.

(8) The value of the soil structure interface coefficient of friction ($\tan \delta$) shall comply with Formula (5.10):

$$\tan\delta \leq k_{\tan\delta} \tan\varphi' \quad (5.10)$$

where

$\tan \varphi'$ is the value of the soil coefficient of effective friction;

$k_{\tan\delta}$ is a reduction factor depending on the foundation material and execution method.

(9) For spread foundations made of concrete cast directly against soil or fill, the value of $k_{\tan\delta}$ should be taken as 1.0 if the base is rough or ridged; or as 2/3 if the base is smooth.

(10) For spread foundations made of pre-cast concrete, the value of $k_{\tan\delta}$ should be taken as 2/3.

(11) When verifying the sliding resistance of a spread foundation, the representative angle of friction of soil or fill should consider potential disturbance of the soil or fill beneath the foundation.

(12) When designing a spread foundation against sliding using the Mohr-Coulomb model, the value of effective cohesion c' at the base of the foundation should be taken as zero.

(13) The value of the sliding resistance of a spread foundation on its front face ($R_{T,face}$) should be determined considering of the nature of the ground including any backfill within the horizontal zone of influence.

5.5.4 Settlement

(1) The following components shall be considered when calculating the settlement of spread foundations:

- immediate settlement;
- settlement caused by consolidation;
- settlement caused by creep; and
- settlement caused by cyclic and dynamic actions.

NOTE 1 Calculation models for settlements of spread foundations are given in B7 to B13 for situations where comparable experience exists.

NOTE 2 Consolidation and creep can occur simultaneously, particularly in thick layers of soil of low permeability.

NOTE 3 Settlement by consolidation typically occurs in fine soils with a high degree of saturation.

NOTE 4 Cyclic actions can generate settlements due to strain and excess ground water pressure accumulation.

(2) The settlement of a foundation on rock may be determined on the basis of comparable experience related to rock mass classification.

(3) The settlement of a spread foundation may be determined using soil and fill parameters, provided the calculation model used is appropriate for the type of ground and is based on comparable experience.

NOTE Information regarding the use of calculation models for settlement is provided in B.7 to B11.

(4) The depth of the compressible soil layer to be considered when calculating settlement should depend on the load, the size and shape of the foundation, the variation in soil stiffness with depth and the spacing of foundation elements.

(5) The following factors potentially causing additional settlement to the ones due to loading should be considered:

- the effect of a change in the effective stress due to reduction in the groundwater pressure;
- the effect of self-weight compaction of the soil;
- the effects of self-weight, flooding and vibration on fill and collapsible soils; and
- the effects of stress changes on crushable coarse soil.

(6) The settlement of spread foundations should be determined assuming a distribution of bearing pressures resulting from the ground-foundation interaction.

(7) Allowance should be made for differential settlement caused by variability of the ground unless it is prevented by the stiffness of the structure.

(8) The tilting of an eccentrically loaded foundation, which is of limited size and hence assumed to be rigid, may be determined by assuming a linear bearing pressure distribution and then calculating the settlement at the corner points of the foundation, using the vertical stress distribution in the ground beneath each corner point and the settlement calculation models described above.

NOTE Differential settlement calculations that ignore the stiffness of the structure tend to be over-predictions.

5.5.5 Heave

(1) Verification of serviceability limit state shall allow for heave caused by the following potential mechanisms:

- reduction of effective stress;
- volume expansion of partly saturated soil;
- death or removal of vegetation;
- seasonal changes of the water content;
- increase in groundwater as a result of water leaking from damaged pipes;
- constant volume deformations in fully saturated soil, caused by settlement of an adjacent structure; and
- chemical reactions in the ground.

NOTE An example of a chemical reaction in the ground causing heave is the transformation of anhydrite (anhydrous calcium sulphate) to gypsum.

(2) Calculations of heave shall include both immediate and delayed heave.

5.6 Ultimate limit states

5.6.1 General

(1) The ultimate limit states of a spread foundation involving overall stability, bearing, and sliding failure shall be verified using Formula (8.1) of prEN 1990:2021.

(2) The design resistance of soil and fill beneath a spread foundation shall be verified for drained and undrained conditions (or a combination of both), depending on the prevailing drainage conditions.

5.6.2 Verification by the partial factor method

5.6.2.1 Overall stability

(1) It shall be verified, in accordance with Clause 4, that a spread foundation does not exceed an ultimate limit state of overall stability.

NOTE This is particularly relevant when the spread foundation is within the zone of influence of sloping ground; excavations or cuttings; rivers, canals, lakes, reservoirs, or the seashore; mine workings or buried structures; other significant changes in the ground surface profile.

5.6.2.2 Bearing failure and overturning

(1) The design bearing resistance normal to the base of a spread foundation R_{Nd} shall be verified using Formula (5.11):

$$N_{\mathrm{d}} \leq R_{\mathrm{Nd}} \tag{5.11}$$

where

N_d is the design value of the normal component of the resulting force on the foundation base;

(2) The design bearing resistance of a spread foundation subject to a horizontal force should be verified using two separate combinations of actions: one treating the vertical force as a favourable action and the other as an unfavourable action.

(3) Overturning subject to combined n vertical and horizontal forces (including gravity walls, reinforced fill structures, and soil nailed structures) shall be verified for bearing failure according to (1).

(4) The design eccentricity of the load acting on a spread foundation should be determined using design actions.

NOTE 1 The design eccentricity is calculated using the partial factors given in 5.6.6.

NOTE 2 When calculated using partial factors on actions from Verification Case VC1, the design eccentricity of loading e_d is limited to the values given in Table 5.1, unless the National Annex gives different values.

Table 5.1 — (NDP) Limits to the design load eccentricity in the case of ULS design

Strip foundation	**Circular foundation**	**Rectangular foundation**
$e_d \leq \left(\frac{7}{15}\right) B$	$e_d \leq \left(\frac{37}{80}\right) D$	$\left(1 - 2\frac{e_{B,d}}{B}\right)\left(1 - 2\frac{e_{L,d}}{L}\right) \geq \frac{1}{15}$

(5) The following precautions shall be taken where the eccentricity of loading exceeds 1/3 of the width of a rectangular foundation or 0.3 times the diameter of a circular foundation:

— careful review of the design values of the actions; and
— designing the location of the foundation edge by considering the magnitude of construction tolerances.

(6) Unless specific measures or different tolerances are specified to control the dimensions of a cast-in-place concrete foundation where the eccentricity of the loading exceeds 1/3 of the foundation width or 0.3 times the diameter of a circular foundation, the design width of the foundation B_d should be determined from Formula (5.12):

$$B_d = B_{nom} - \Delta B \tag{5.12}$$

where

B_{nom} is the nominal width of the foundation;

ΔB is a deviation.

NOTE The value of ΔB is 0.1 m, unless the National Annex gives a different value.

5.6.2.3 Sliding failure

(1) Where the applied force is not normal to the foundation base, the foundation shall be verified against sliding failure.

(2) The design sliding resistance along the base of a spread foundation shall comply with Formula (5.13):

$$T_{\mathrm{d}} \leq R_{\mathrm{Td,base}} + R_{\mathrm{Td,face}} \tag{5.13}$$

where:

T_{d} is the design value of the applied force acting parallel to the foundation base, including any thrust caused by earth pressure acting on the foundation;

$R_{\mathrm{Td,,base}}$ is the design value of the resistance of the foundation base to sliding;

$R_{\mathrm{Td,,face}}$ is the design value of the resistance force to sliding caused by earth pressure on the front face of the foundation, i.e. the design face resistance.

(3) Thrust caused by earth pressure acting on the foundation (included in T_{d} in Formula (5.13)) and $R_{\mathrm{Td,face}}$ shall be determined according to clause 7.

(4) The values T_{d}, $R_{\mathrm{Td,base}}$, and $R_{\mathrm{Td,face}}$ shall be related to the scale of movement anticipated under the limit state design loading.

NOTE The displacements required to mobilize shear resistance at the base of the foundation are much lower than the displacements required to mobilize earth pressures on the foundation front face.

(5) The value of $R_{\mathrm{Td,face}}$ should allow for potential loss of ground strength caused by large displacements.

(6) For spread foundations on fine soils resting within the zone of seasonal changes of the water content, the possibility that the soil could shrink away from the vertical faces of foundations resulting in face resistance not being available shall be considered.

(7) The possibility that face resistance cannot be available as a result of the soil in front of the foundation being removed by erosion or human activity shall be considered.

(8) When using the material factor approach, the design undrained sliding resistance $R_{\mathrm{Tud,base}}$ of a spread foundation on soil or fill shall be determined using Formula (5.14):

$$R_{\mathrm{Tud}} = A_{\mathrm{red}}\, k_{\mathrm{cu}}\, c_{\mathrm{u,d}} = A_{\mathrm{red}}\, k_{\mathrm{cu}} \frac{c_{\mathrm{u,rep}}}{\gamma_{\mathrm{cu}}} \tag{5.14}$$

where

A_{red} is the plan area of the foundation base, not including any area where there is no positive contact pressure between the foundation and the underlying ground as a result of load eccentricity, ground shrinkage, or any other cause;

k_{cu} is a reduction factor depending on the foundation material, execution method, and soil or fill disturbance;

$c_{\mathrm{u,d}}$ is the design value of the soil or fill undrained shear strength;

$c_{\mathrm{u,rep}}$ is the representative value of the soil or fill undrained shear strength;

γ_{cu} is a partial factor on undrained shear strength.

NOTE Values for the reduction factor k_{cu} are specified in 5.5.3 (5) and (6).

(9) When using the resistance factor approach, the design undrained sliding resistance $R_{\mathrm{Tud,base}}$ of a spread foundation shall be determined using Formula (5.15):

$$R_{\text{Tud,base}} = \frac{A_{red} k_{cu} c_{\text{u,rep}}}{\gamma_{\text{RT}}} \tag{5.15}$$

where, in addition to the parameters defined for Formula (5.14):

γ_{RT} is the partial factor on sliding resistance

(10) In addition to (9) the design sliding resistance $R_{\text{Tud,base}}$ shall comply with Formula (5.16) if:

- it is possible for water or air to reach the interface between the foundation and the surrounding soil or fill; or
- the formation of a gap between the foundation and the surrounding soil or fill is not prevented by suction in areas where there is no positive bearing pressure.

$$R_{\text{Tud}} \leq 0.4\, N_{\text{rep,fav}} \tag{5.16}$$

where

$N_{\text{rep,fav}}$ is the design value of the force acting normal to the foundation base, considered as a favourable action

(11)When using the material factor approach, the design drained sliding resistance R_{Td} in of a spread foundation on ground shall be determined from Formula (5.17):

$$R_{\text{Td}} = (N_{G,d,fav} - U_d)\ \tan\delta_{\text{d}} \tag{5.17}$$

where:

N_{d} is the design value of the permanent force acting normal to the foundation base, considered as a favourable action;

U_{d} Is the design value of any uplift force from groundwater pressures acting normal to the foundation base;

$\tan\delta_{\text{d}}$ is the design value of interface friction between the foundation and the ground.

NOTE 1 Design values of groundwater pressures are specified in prEN 1997-1:2022, 6.

NOTE 2 Values of partial factors $\gamma_{\tan\delta}$ are given in prEN 1997-1:2022, 4.4.1.3.

(12)When using the resistance factor approach, the design drained sliding resistance $R_{\text{Td,base}}$ of a spread foundation on ground shall be determined using Formula (5.18) for VC1 or Formula (5.19) for VC4:

$$R_{Td} = \frac{(N_{G,d,fav} - U_d)\, tan\,\delta_{rep}}{\gamma_{RT}} \tag{5.18}$$

$$R_{Td} = \frac{(N_{G,rep,fav} - U_{rep})\, tan\,\delta_{rep}}{\gamma_{RT}} \tag{5.19}$$

where:

$N_{\text{G,d,fav}}$ is the design value of the favourable permanent force acting normal to the foundation base;

$N_{\text{G,rep,fav}}$ is the representative value of the favourable permanent force acting normal to the foundation base;

U_d is the design value of any uplift force from groundwater pressures normal to the foundation base;

U_{rep} is the representative value of the any uplift force from groundwater pressures normal to the foundation base;

δ_{rep} is the representative value of interface friction between the foundation and the ground;

γ_{RT} is a partial factor on sliding resistance.

NOTE 1 Representative values of groundwater pressures are specified in prEN 1997-1:2022, 6.

NOTE 2 Values of partial factors γ_{RT} are given in 5.6.6.

(13) The determination of $N_{G,d,fav}$ and $N_{G,rep,fav}$, shall consider whether T and N are independent or interdependent actions.

5.6.2.4 Toppling

(1) The stability against toppling of a spread foundation shall be verified in accordance with prEN 1990.

NOTE Toppling is rotational failure that does not involve failure of the ground.

5.6.3 Verification by prescriptive rules

(1) prEN 1997-1:2022, 4.5 shall apply to spread foundations

NOTE Guidance on the use of the presumed bearing pressures can be given in the National Annexes

5.6.4 Verification by testing

(1) prEN 1997-1:2022, 4.6 shall apply to spread foundations

(2) The results of large-scale tests may be used to verify limit states for a spread foundation directly.

(3) The location of the test shall be chosen in accordance with the ground investigation results to be representative of the most unfavourable ground conditions likely to be found under the structure.

(4) When evaluating the results of large-scale foundation tests to verify limit states, any excess groundwater pressures beneath the foundation shall be measured and considered.

(5) When using a test to verify limit states for a spread foundation, any differences in scale and response between the test foundation and the real foundation shall be considered, including the adverse influence of weak layers within the zone of influence of the test or real foundation.

5.6.5 Verification by the Observational Method

(1) prEN 1997-1:2022, 4.7 shall apply to spread foundations

5.6.6 Partial factors

(1) Partial factors for the verification of spread foundations at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using either the Material Factor Approach or the Resistance Factor Approach.

NOTE 1 The National Annex can specify which Factor Approach to use.

NOTE 2 Values of partial factors are given in Table 5.2 (NDP) for persistent and transient design situations, and Table 5.3 (NDP), for accidental design situations, unless the National Annex gives different values.

NOTE 3 If the Material Factor Approach is used, the National Annex can specify whether to use both combinations (a) and (b) or the single combination (c) in Table 5.2 (NDP) and Table 5.3 (NDP).

Table 5.2 — (NDP) Partial factors for the verification of ground resistance of spread foundations for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA), either both combinations (a) and (b) or the single combination (c)			Resistance factor approach (RFA), either combination (d) or ©[c]	
			(a)	(©(c)	(d)	(e)	
Overall stability	See Clause 4						
Bearing and sliding resistance	Actions and effects-of-actions	γ_F and γ_E	VC1[a]	VC3[a]	VC1[a]	VC1[a]	VC4
	Ground properties	γ_M	M1[b]	M2[b]	M2[b]	Not factored	
	Bearing resistance	γ_{RN}	Not factored			1,4	
	Sliding resistance	γ_{RT}	Not factored			1,1	

[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in prEN 1990:2021 Annex A.

[b] Values of the partial factors for Sets M1 and M2 are given in prEN 1997-1:2022, Table 4.7.

[c] Use combination (d) except where specified otherwise in 5.6.6 (2) and (3)

Table 5.3 — (NDP) Partial factors for the verification of ground resistance of spread foundations for accidental design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA), either both combinations (a) and (b) or the single combination (c)			Resistance factor approach (RFA)
			(a)	(b)	©	
Overall stability	See Clause 4					
Bearing and sliding resistance	Actions and effects-of-actions	γ_F and γ_E	Not factored			
	Ground properties	γ_M	M1[a]	M2[a]	M2[a]	Not factored
	Bearing resistance	γ_{RN}	Not factored			1,20
	Sliding resistance	γ_{RT}	Not factored			1,05

[a] Values of the partial factors for Sets M1 and M2 are given in prEN 1997-1:2022, Table 4.7.

(2) If the resistance factor approach is used to determine the bearing resistance of spread foundations under inclined loading, Verification Case 4 may be used instead of Verification Case 1, provided the condition in Formula (5.20) is satisfied:

$$T_{\text{rep}} \leq 0{,}2N_{\text{rep}} \tag{5.20}$$

where

T_{rep} is the representative value of the force acting tangential to the foundation base;

N_{rep} is the representative value of the force acting normal to the foundation base, considered as a favourable action.

(3) If the resistance factor approach is used to determine bearing resistance of gravity retaining structures, Verification Case 4 may be used instead of Verification Case 1.

(4) Provided the conditions specified in prEN 1997-1:2022 4.4.3(10) are satisfied, the value of γ_{RN} and γ_{RT} for transient design situations may be multiplied by a factor $K_{\text{R,tr}} \leq 1{,}0$ provided that the products $K_{\text{R,tr}}\ \gamma_{\text{RN}}$ and $K_{\text{R,tr}}\ \gamma_{\text{RT}}$ are not less than 1,0.

NOTE For spread foundations, the value of $K_{\text{R,tr}}$ is 1,0 unless the National Annex gives a different value.

5.7 Serviceability limit states

5.7.1 General

(1) prEN 1997-1:2022, 9 shall apply to spread foundations.

(2) The adverse effects of foundation displacements shall be considered both in terms of displacement of the entire foundation and differential displacements of parts of the foundation.

(3) Displacements caused by actions on the foundation shall be considered, including the actions given in prEN 1997-1:2022, 4.3.1.2(1).

(4) In determining the magnitude of foundation displacements, comparable experience shall be considered, as given in prEN 1997-1:2022, 3.1.2.3.

(5) The effect of existing adjacent foundations, fills, and excavations shall be considered, including the stress increase in the ground and its influence on ground compressibility and displacement.

5.7.2 Settlement

(1) To ensure the avoidance of a serviceability limit state, determination of differential settlements and relative rotations shall consider both the distribution of loads and the variability of the ground.

(2) Upper and lower bound values of settlement should be determined using inferior and superior representative values of stiffness and hydraulic conductivity.

5.7.3 Tilting

(1) For spread foundations subject to eccentric loading, it shall be verified that differential settlement of the foundation will not result in the occurrence of a serviceability limit state due to unacceptable tilting of the supported structure.

5.7.4 Vibration

(1) Foundations for structures subjected to vibrating loads shall be designed to ensure that vibrations will not cause excessive settlements or a loss of serviceability of supported or adjacent structures.

(2) Precautions should be taken to ensure that resonance will not occur between the frequency of the dynamic load and a critical frequency in the foundation-ground system, and to ensure that liquefaction will not occur in the ground.

5.8 Implementation of design

5.8.1 General

(1) prEN 1997-1:2022, 10 shall apply to spread foundation.

(2) The execution of concrete spread foundations should comply with EN 13670.

5.8.2 Inspection

(1) prEN 1997-1:2022, 10.3 shall apply to spread foundation

5.8.3 Monitoring

(1) prEN 1997-1:2022, 10.4 shall apply to spread foundation

5.8.4 Maintenance

(1) prEN 1997-1:2022, 10.5 shall apply to spread foundation

(2) Groundwater control systems around spread foundations should be designed for ease of maintenance and renewal during the design life of the structure.

5.9 Testing

(1) prEN 1997-1:2022, 11 shall apply to spread foundation

(2) The results of Plate Loading Tests should only be used for verification of limit state if:

- the size of the plate has been chosen considering the width of the planned spread foundation; and
- a homogeneous layer up to two times the width of the planned spread foundation exists.

NOTE The depth of the zone tested by the Plate Loading Test is limited to approximately twice the diameter of the plate. Therefore, no inference concerning the soil quality below that depth can be made unless additional investigation, e.g. sounding, is carried out.

(3) Based on established experience, the results of a Plate Loading Test may be used with an adjusted elasticity method to determine Young's modulus and evaluate the settlement of a spread foundation on soil and fill and on rock.

NOTE An adjusted elasticity method is given in B.7.

(4) When a Plate Loading Test is used to determine the Young's modulus and evaluate the settlement of a spread foundation on soil and fill, the effects of any groundwater pressures generated on loading should be considered.

(5) Dummy footing tests, skip tests, zone tests, and small-scale prototype tests may also be used to verify the design of a spread foundation on soil or fill, provided the size of the loaded area and the depth of a homogeneous layer beneath the planned foundation comply with (3).

5.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to spread foundation.

6 Piled foundations

6.1 Scope and field of application

(1) This Clause shall apply to single piles, pile groups and piled rafts.

(2) In addition to Clause 11, part of this clause shall apply to rigid inclusions.

(3) Piles should be classified according to their method of execution.

NOTE 1 The classification is given in Table 6.1 (NDP) unless the National Annex gives a different classification.

NOTE 2 The pile type is used to determine resistance factors, see 6.6.3.

NOTE 3 Examples of different pile types are given in Annex C.3.

Table 6.1 — (NDP) Classification of piles

Pile type	Description	Class
Displacement pile	Pile installed in the ground without excavation of material	Full displacement
		Partial displacement
Replacement pile	Pile installed in the ground after the excavation of material	Replacement
Pile not listed above	---	Unclassified

6.2 Basis of design

6.2.1 Design situations

(1) prEN 1997-1:2022, 4.2.2 shall apply to piled foundations.

6.2.2 Geometrical properties

6.2.2.1 General

(1) prEN 1997-1:2022, 4.3.3 shall apply to piled foundations.

6.2.2.2 Single Pile

(1) Pile dimensions shall be selected according to the pile type and method of execution, the stability of the ground, and the potential adverse changes that can occur due to pile installation.

NOTE Nominal dimensions are given in the execution standards given in 6.8.1

(2) The adverse effects of pile geometrical imperfections shall be considered in the verification of limit states.

NOTE 1 The execution standards given in 6.8.1 give positional and verticality tolerances. Other geometrical imperfections can include curvature of the pile shaft, bulging or necking of the pile, and oversized or undersized bores.

NOTE 2 Annex C.13 provides calculation models to consider second order effects induced by some geometrical imperfections.

6.2.2.3 Pile groups

(1) The spacing of piles in groups should be selected according to the pile type, method of execution, proposed sequence of execution, pile length, ground conditions, and anticipated pile group behaviour.

(2) Pile spacing should be sufficient to avoid damage to previously constructed piles, considering positional and verticality tolerances.

6.2.3 Zone of influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to piled foundations.

(2) The adverse effects of nearby construction activity on the piled foundation shall be considered.

(3) The adverse effects of pile execution resulting in ground movement and vibrations that could impact on nearby structures should be considered.

6.2.4 Actions and environmental influences

6.2.4.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to piled foundations.

6.2.4.2 Permanent and variables actions

(1) Actions for piled foundations shall include, but are not limited to:

- applied axial, transverse, and shear forces in any combination;
- applied bending and torsional moments in any combination;
- static, cyclic, dynamic, or impact actions in any combination;
- loading due to lateral or vertical ground displacements;
- pile imperfections that result in additional bending moment or shear loads;
- loading due to thermal deformations of the pile or surrounding ground.

NOTE Seismic actions are defined in EN 1998 (all parts).

6.2.4.3 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to piled foundations.

(2) The adverse effects of cyclic and dynamic action on the long-term bearing and transverse resistance of piled foundations, shall be considered.

NOTE 1 Cyclic and dynamic actions can result in reduced ground strength and stiffness leading to additional pile displacements and loss of resistance.

NOTE 2 In coarse fills and soils, cyclic and dynamic actions can result in densification of the ground leading to increased stiffness, particularly in the horizontal direction.

(3) For axially loaded piles, the stability diagram may be used to assess whether the effects of cyclic loads can significantly affect the response of the pile or can be neglected.

NOTE 1 The concept of a pile stability diagram is presented in Annex C.14.

NOTE 2 The effect of cyclic actions on the axial pile resistance depends on the pile properties, load characteristics and ground properties.

6.2.4.4 Actions due to ground displacement

(1) The adverse effects on the piled foundation of vertical and horizontal ground movements shall be considered.

NOTE 1 See 6.5.2.2 for a method of calculating downdrag action on piles.

NOTE 2 Ground mass displacement are assessed according to Clause 4.

6.2.4.5 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.5 shall apply to piled foundations.

6.2.5 Limit states

6.2.5.1 Ultimate Limit States

(1) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be verified for all piled foundations:

- failure of the ground surrounding the piled foundation;
- failure of the ground between individual piles;
- buckling of the pile element;
- structural failure of the pile element (see EN 1992 (all parts), prEN 1993 (all parts) or EN 1995 (all parts) respectively based on pile material);
- combined failure of the ground and the structural pile element;
- failure of the supported structure caused by excessive pile movement.

(2) Potential ultimate limit states other than those given in (1) should be verified.

6.2.5.2 Serviceability Limit States

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all piled foundations:

- pile settlement;
- differential settlements;
- settlement caused by downdrag;
- heave;
- transverse movement;
- unacceptable movements or distortions of the structure caused by pile movements.

(2) Potential serviceability limit states other than those given in (1) should be verified.

6.2.6 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply to piled foundations.

6.2.7 Ground investigation

6.2.7.1 General

(1) prEN 1997-2:2022, 5 shall apply to piled foundations.

(2) The ground investigation should include one or more of the following:

- field tests to allow direct correlation with the pile shaft and base resistance;
- field tests to determine the shear strength and stiffness of ground;
- laboratory tests to determine ground shear strength and stiffness;
- description of the geological and geotechnical ground conditions.

(3) In addition to (1) for piled foundations on or in very weak to weak rock mass or weakness zones at the anticipated pile base level, the ground investigation should include one or more of the following:

- rotary core drill holes to provide undisturbed core samples;
- assessment of any core loss, fracturing and joint spacing;
- a full core description complying with EN ISO 14689, including estimates of rock strength;
- laboratory testing to determine the compressive strength of the rock.

(4) In addition to (1) for piled foundations on or in medium to strong rock mass at the anticipated pile base level, the ground investigation should include one or more of the following:

- measurement while drilling;
- borehole video logging;
- comprehensive comparable experience.

(5) The aggressiveness of the ground and groundwater shall be determined during the ground investigation.

(6) In addition to (1) – (3), the ground investigation may include:

- visual inspection of rock surfaces;
- site trials and prototype pile installation;
- installation of piles for load testing;
- observation of spoil from drilled or bored replacement piles;
- measurement of drive blows for driven displacement piles;
- drive energy analysis;
- static load testing;
- dynamic impact load testing;
- rapid load testing.

6.2.7.2 Minimum extent of field investigation

(1) The depth and horizontal extent of field investigation shall be sufficient to determine ground conditions within the zone of influence of the structure according to prEN 1997-1:2022, 4.2.1.1.

(2) The field investigation shall determine ground conditions over the full depth of the piled foundation including any overlying fills or low strength soils, and should extend beyond the anticipated founding stratum at or pile base.

(3) The minimum depth of field investigation below the anticipated base of a piled foundation d_{min} in soils and in very weak and weak rock masses should be determined from Formula 6.1:

$$d_{min} = max(5\text{ m}; 3B_{\text{b,eq}}; p_{\text{group}}) \quad (6.1)$$

where

$B_{b,eq}$ is the equivalent size of the pile base, equal to B_b (for square piles), D_b (for circular piles), or p_b/π (for other piles);

B_b is the base width of the pile with the largest base (for square piles);

D_b is the base diameter of the pile with the largest base (for circular piles);

p_b is the base perimeter of the pile with the largest base (for other piles);

p_{group} is the smaller dimension of a rectangle circumscribing the group of piles forming the foundation, limited to the depth of the zone of influence.

(4) The value of d_{min} in strong rock masses should be determined from Formula (6.2):

$$d_{min} = max(3\ m; 3B_{b,eq}) \quad (6.2)$$

(5) The value of d_{min} should be increased for rock masses that are susceptible to dissolution features or cavities, or where closely spaced discontinuities may reduce the mass strength and stiffness.

(6) The value of d_{min} in medium strong and strong rock mass or dense moraine may be reduced provided there is comparable experience to allow the properties of the rock mass or moraine to be predicted.

6.2.8 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to piled foundations.

(2) Piled foundations shall be classified as GC 2 or GC 3.

6.3 Materials

6.3.1 Ground properties

(1) prEN 1997-2:2022, Clauses 7 to 12 shall apply to piled foundations.

(2) The following non-exhaustive list of field tests and ground parameters may be used to calculate axial or transverse pile resistance:

- cone resistance from Cone Penetration Tests;
- corrected blow counts from Standard Penetration Tests;
- limit pressure from Pressuremeter Tests;
- effective shear strength parameters of fill, soil, or weak rock;
- constant volume effective stress parameter of fill or soil;
- undrained shear strength of fill or soil;
- unconfined compressive strength of rock;

– compressive strength of rock mass and mechanical properties of discontinuities.

(3) The effect of subsequent excavation, placement of overburden, or changes in groundwater pressure on the values of ground properties should be considered.

(4) Verification of limit states should be based on ground parameters that represent the strength and stiffness of the ground after pile execution, unless the selected design method implicitly allows for execution effects.

6.3.2 Plain and reinforced concrete

(1) prEN 1997-1:2022, 5.5 shall apply to piled foundations.

(2) Exposure classes for concrete should comply with EN 206.

(3) Concrete cover requirements shall comply with prEN 1992-1-1.

NOTE For many reinforced concrete piles or piled foundations constructed in natural ground, the exposure class will be XA1, XA2 or XA3. Currently prEN 1992-1-1 does not provide guidance for the cover allowance for durability for these exposure classes.

(4) In the absence of alternative guidance, the minimum cover for environmental conditions $c_{\text{min,dur}}$ should be 25 mm for reinforced concrete used for both precast and cast-in-place piles.

(5) In the absence of alternative guidance, the allowance for deviation Δc_{dev} should be 50 mm for concrete cast against the ground and 10 mm for precast piles.

NOTE EN 12794 and EN 13369 give additional recommendations.

(6) The value for Δc_{dev} for precast piles may be reduced in accordance with prEN 1992-1-1:2021, 4.4.1.3 (3) when fabrication is subject to a quality assurance system with measurement of concrete cover.

6.3.3 Plain and reinforced grout and mortar

(1) prEN 1997-1:2022, 5.4 shall apply to piled foundations.

(2) Exposure classes for grout and mortar should comply with:

– 6.3.2(2) for durability;
– EN 14199 for corrosion protection.

(3) In the absence of guidance, exposure classes for grout and mortar, and rules for durability may be determined from comparable experience or testing.

6.3.4 Steel

(1) prEN 1997-1:2022, 5.6 shall apply to piled foundations.

6.3.5 Steel reinforcement

(1) prEN 1997-1:2022, 5.5 shall apply to piled foundations.

6.3.6 Ductile cast iron

(1) Cast iron for piles or piled foundation and the values of cast iron properties should comply with EN 1563.

6.3.7 Timber

(1) prEN 1997-1:2022, 5.7 shall apply for pile design.

(2) Timber grading for pile foundations should comply with the general requirements of EN 14081-1.

(3) Timber piles without preservative treatment may be used provided the piles are installed below the groundwater table and remain fully submerged throughout their design service life.

6.4 Groundwater

(1) prEN 1997-1:2022, 6 shall apply to piled foundations.

6.5 Geotechnical analysis

6.5.1 General

(1) prEN 1997-1:2022, 7 shall apply to piled foundations.

(2) The interaction between the structure, pile foundation and ground shall be considered when verifying limit states.

(3) Combined axial and lateral loading may be analysed by separating each load component and applying the principle of superposition, provided pile internal behaviour remains substantially elastic.

(4) The non-linearity of the load-displacement curve of axially and transversally loaded piles should be considered for the verification of both geotechnical and structural limit states.

6.5.2 Effect of ground displacement

6.5.2.1 General

(1) Actions due to ground displacement shall be modelled either by treating the displacement as an action or as an equivalent design force.

(2) Evaluation of an equivalent design force should take account of the strength and stiffness of the ground, together with the source, magnitude and direction of the ground displacement by assuming the most unfavourable values of the strength and stiffness of the moving ground.

6.5.2.2 Downdrag

(1) The adverse effects of the drag force caused by moving ground shall be included in the verification of serviceability and ultimate limit states.

(2) The effects of the downdrag should be modelled by carrying out a ground-pile interaction analysis, to determine the depth of the neutral plane L_{dd} corresponding to the point where the pile settlement spile equals the ground settlement..

NOTE 1 The neutral plane marks the boundary between downwards shaft friction (occurring above the neutral plane), and upwards shaft friction (occurring below the neutral plane).

NOTE 2 The depth of the neutral plane L_{dd} is usually different for serviceability and ultimate limit state conditions.

(3) The ground-pile interaction analysis should provide force, displacement, and strain profiles for the full depth of the pile to enable the representative drag force D_{rep} acting on the pile shaft above the neutral plane to be determined.

NOTE See C.9 for detailed models and combinations of actions for downdrag.

(4) In addition to prEN 1990-1:2021, 6.1.1(4) and prEN 1990-1:2021, 8.3.3.1(3)-(4), when carrying out an interaction analysis, if the drag force and shaft resistance originate in a single geotechnical unit, with no significant change in strength or stiffness across the neutral plane, then both the drag force and the resistance may be considered as coming from a single-source.

(5) The equivalent drag force D_{rep} should be determined from Formula 6.3:

$$D_{\mathrm{rep}} = p\int_0^{L_{\mathrm{dd}}} \tau_{\mathrm{s}} \cdot \mathrm{dz} \tag{6.3}$$

where

p is the perimeter of the pile;

τ_s is the unit shaft friction causing downdrag at depth z;

L_{dd} is the depth to the neutral plane.

(6) In order to provide a cautious estimate of the downdrag force, the shaft friction causing downdrag should be determined from upper (superior) ground parameters.

6.5.2.3 Heave

(1) Verification of the pile compression or tensile resistance shall take account of ground heave (including swelling) which could take place during execution before piles are fully loaded by the structure.

(2) The adverse effects of heave caused by moving ground shall be included in the verification of serviceability and ultimate limit states, especially to avoid tensile failure of the pile.

(3) Verification of serviceability limit states should consider short- or long-term ground heave sufficient to cause unacceptable uplift to the pile element or to result in a serviceability limit state in the overall structure.

(4) Long-term heave may be disregarded where the imposed permanent actions exceed the heave load.

6.5.2.4 Transverse loading

(1) Verification of the pile transverse resistance and displacement shall take account of actions on piles originating from the adverse effect of ground movements or asymmetric loads around a pile.

6.5.3 Axially loaded single piles

6.5.3.1 Calculation

(1) The axial resistance of a single pile shall be determined based on comparable experience from the results of field investigation and laboratory testing or load tests.

(2) The axial resistance of a single pile designed by calculation shall be determined by one of the following methods:

– using ground properties determined from field and laboratory tests (the Ground Model Method); or
– using individual pile resistance profiles determined from correlations with field test results or ground properties from field or laboratory tests (the Model Pile Method).

NOTE The method (Ground Model or Model Pile) to be used can be given in the National Annex.

(3) The validity of the method used to assess the base and shaft resistance of a pile shall be proved by documented load testing of comparable piled foundations and case histories that confirm that the method provides reliable pile resistance and performance.

NOTE Methods of calculating base and shaft resistance are included in C.4 and C.5 for ground parameters, C.6 for cone penetration test methods, and C.7 for pressuremeter methods.

(4) The axial compressive resistance R_c of a single pile should be determined from Formula 6.4

$$R_c = R_b + R_s \tag{6.4}$$

where

R_b is the pile base resistance;

R_s is the pile shaft resistance.

NOTE 1 The use of Formula (6.4 assumes the compatibility of the displacements to mobilise both base resistance and shaft resistance considering the pile geometry and the difference of stiffness between the ground and the pile. In case of layered ground with layers of significant different stiffness, shaft resistance may not be fully mobilized in layers of lower stiffness.

NOTE 2 For piled foundation on rock the proportion of base resistance and shaft resistance to be taken into account depends on the ratio of E_c (concrete Young's modulus) to E_{rm} (rock mass Young's modulus) and on the pile slenderness. The shaft resistance of soil layers tends to reduce to 0, when a pile is socketed in competent rock.

(5) The weight of the pile should be included as an action in the calculation model, in which case the beneficial contribution of overburden should be included in the axial compressive resistance at the pile base.

(6) The weight of the pile and the additional resistance at the pile base due to overburden pressure may both be disregarded provided that:

– the pile weight and the contribution to resistance due to overburden pressure are approximately equal;
– downdrag is not significant;
– the soil or fill does not have a very low weight density;
– the pile does not extend above the surface of the ground.

(7) The weight of the pile element may be included as a resistance for piles loaded by tension.

(8) The pile base resistance in compression R_b should be determined from Formula(6.5):

$$R_{\mathrm{b}} = A_{\mathrm{b}} \cdot q_{\mathrm{b}} \tag{6.5}$$

where

q_{b} is the unit base resistance;

A_{b} is the area of the pile base.

(9) The pile shaft resistance Rs in compression should be determined from Formula(6.6):

$$R_{\mathrm{s}} = \sum_{i=1}^{n} A_{\mathrm{s,i}} q_{\mathrm{s,i}} \tag{6.6}$$

where

$q_{\mathrm{s,i}}$ is the unit shaft resistance in the *i*-th geotechnical unit;

$A_{\mathrm{s,i}}$ is the area of the pile shaft in the *i*-th geotechnical unit;

i is an index that varies from 1 to *n*;

n is the number of geotechnical units providing resistance.

(10)The pile shaft resistance in tension Rst should be determined from Formula(6.7):

$$R_{\mathrm{st}} = \sum_{i=1}^{n} A_{\mathrm{s,i}} q_{\mathrm{st,i}} \tag{6.7}$$

where

$q_{\mathrm{st,i}}$ is the unit shaft resistance in tension in the *i*-th geotechnical unit.

6.5.3.2 Prescriptive rules

(1) The axial compressive resistance of a single pile may be determined using prescriptive rules where specified by a relevant authority.

6.5.3.3 Testing

(1) The axial compressive resistance of a single pile at the ultimate limit state may be determined from the results of static load, dynamic impact, or rapid load tests.

(2) The axial tensile resistance of a single pile at the ultimate limit state may be determined from the results of static load tests.

(3) Determination of the axial resistance of a single pile from static load tests should account for potential temporary support.

(4) The compressive resistance of a single pile may be determined from the results of dynamic impact or rapid load tests provided adjustments are made to account for temporary support.

(5) The compressive resistance of a friction pile from a dynamic impact test should be determined from the maximum applied test load determined by signal matching.

(6) In the absence of site-specific correlations, the validity of dynamic impact or rapid load tests shall have been established using static load test previously carried out in documented comparable

situation on the same pile type, with similar geometry, in comparable ground conditions, and tested to similar load levels.

(7) Results of dynamic impact or rapid load tests where more than 30 % of the pile resistance is provided by shaft friction or end bearing in fine soils should only be used to determine R_c if there is site-specific calibration against static load test.

(8) The validity of the interpreted results from dynamic impact or rapid load tests should be demonstrated by static load tests carried out in parallel to allow direct site-specific correlation.

(9) Allowance for any potential pile set-up may be included provided this has been either verified by load tests on piles of different ages or established by comparable experience.

(10) The compressive resistance of a pile may be determined from the results of wave equation analysis based on the registered energy transfer to the pile during driving, provided the analysis has previously been calibrated against the results of static load tests on the same pile type, with similar geometry and installation method and in comparable ground conditions.

(11) The compressive resistance of an end-bearing pile in coarse soil or rock may be based on a pile driving formula provided the formula has previously been calibrated against the results of static load tests on the same pile type, with similar geometry, of similar installation method and in comparable ground conditions.

(12) Analysis of the results of dynamic impact tests may be carried out using wave equation analysis for confirmation of design or for interpolation between test locations when it is necessary to modify the design to consider different design situations.

(13) Wave equation analysis may also be used to determine the effect of significant changes in dimensions, length, impact energy, and final set of piles that are not load tested.

(14) Wave equation analysis or driving formulae may be used to determine driving criteria for control purposes.

6.5.4 Transversely loaded single piles

(1) The transverse resistance of a single pile may be determined by calculation or by testing.

(2) The transverse resistance of a single pile may be determined assuming rotation or translation of the pile as a rigid body (for short piles with a ratio (length to diameter ratio $L/D < 6$) or bending failure and local yielding of the pile for longer piles ($L/D \geq 6$).

NOTE Verification of piles for transverse loading is often controlled by the serviceability limit state rather than ultimate limit state.

(3) Temporary support from moving ground that will reduce or reverse during the design service life of the piled foundation shall not be included in the computation of transverse resistance.

(4) The transverse resistance of a single pile shall take account of the fixity of the pile head to the pile cap or sub-structure and the fixity of the pile base.

(5) The transverse resistance of a single pile should take account of potential variations of ground stiffness with depth.

(6) For piles in multi-layered soils, superior (upper) and inferior (lower) values of soil stiffness in different layers should be combined in the most adverse manner.

NOTE For example, upper bound stiffness for stiff soil layers and lower bound for less stiff layers.

(7) The transverse geotechnical and structural resistance of a socketed pile should include specific analyses of the pile base, especially when shear forces are present owing to a large difference in stiffness between the rock mass and any overlying soil.

(8) If piles are additionally loaded transversally, they should be verified using second order theory.

NOTE For example, additionally load can be induced by settlement of the ground, displacement of sloping ground or by structural actions.

6.5.5 Pile groups

(1) Verification of limit states for pile groups may be carried out by numerical, analytical, or empirical calculation methods, or determined from the observed performance of comparable pile groups.

(2) Pile group design shall consider that the resistance and load-displacement behaviour of individual piles in a group might show significant variation compared to the behaviour of single piles.

(3) Calculation of pile group effects should consider the potential changes in stress and density of the ground resulting from pile installation together with the effects of group behaviour due to the structural loads.

(4) Pile group design may be based on the results of load tests on individual piles provided the interaction between individual piles and pile group effects are considered.

(5) The ultimate vertical resistance of a pile group R_{group} should be determined from Formula 6.8:

$$R_{\text{group}} = min\left\{\sum_{i}^{n} R_{\text{i}} ; R_{\text{block}}\right\} \tag{6.8}$$

where

- R_{i} is the ultimate axial resistance of the i-th pile in the pile group, taking full account of the effects of pile interaction;
- i is an index that varies from 1 to n;
- n is the number of piles within the piled foundation;
- R_{block} is the ultimate vertical resistance of the block of ground bounded by the perimeter of the pile group.

(6) In the case of tension loading, the reduction in effective vertical stresses in the ground should be considered when deriving the shaft resistance of individual piles in the group.

NOTE For the evaluation of the block failure of pile groups subject to axial tension see C.10.

(7) The effects of pile interaction, the shadow effect of closely spaced piles, and head fixity of piles should be accounted for when deriving the transverse resistance of a pile group from the results of calculations or load tests on individual test piles.

(8) Where interaction effects between piles are expected to be significant, the verification of limit states should be based on numerical models that consider non-linear ground-pile response and can cater for combined axial, lateral, and moment actions.

(9) If the piles in a group are connected by a pile cap that is unable to redistribute loads, verification of limit states shall be based on the pile in the most unfavourable condition.

(10) The verification of geotechnical ultimate and serviceability limit states for individual piles may be omitted provided is verified that the pile cap is able to redistribute loads without itself exceeding an ultimate or serviceability limit state.

6.5.6 Piled rafts

(1) The ultimate compressive resistance of a piled raft $R_{\text{piled-raft}}$ should be determined from Formula 6.9 considering the compatibility of the displacements of the piles and the rafts:

$$R_{\text{piled}-\text{raft}} = \left(\sum_{i}^{n} R_{\text{c,i}} + R_{\text{raft}} \right) \tag{6.9}$$

where

R_{raft} is the ultimate compressive resistance of the raft alone;

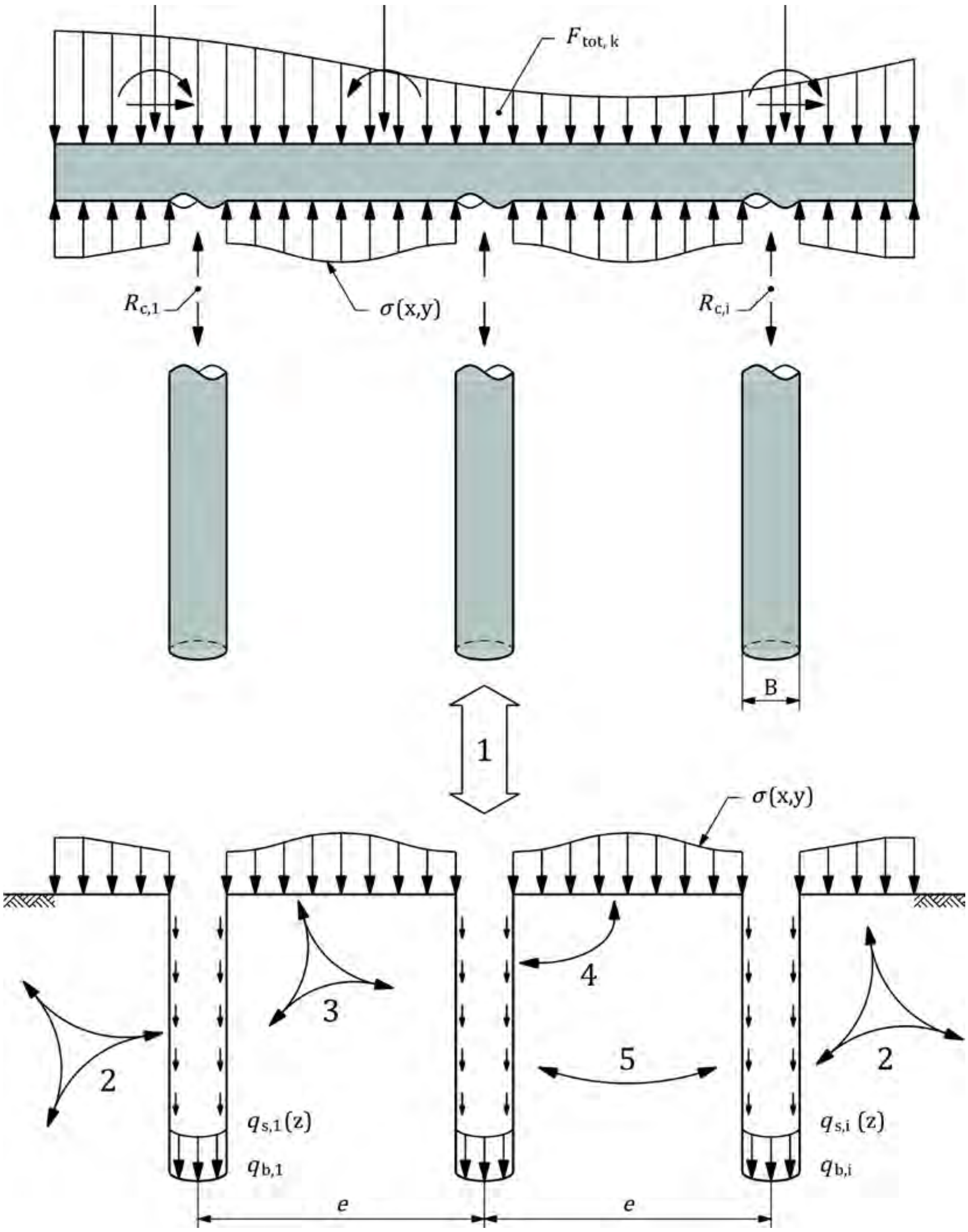
$R_{\text{c,i}}$ is the compressive resistance of the i-th pile;

i is an index that varies from 1 to n;

n is the number of piles supporting the piled-raft.

(2) The design of piled rafts should consider the interaction effects shown in Figure 6.1:

- pile-soil interaction;
- pile-pile interaction;
- raft-soil interaction;
- pile-raft interaction.



Key

1 interaction between piled raft and ground

2 Piled-Ground-Interaction

3 Raft-Ground-Interaction

4 Piled-Raft-Interaction

5 Pile-Pile-Interaction

e distance between piles

B pile diameter

Figure 6.1 — Interaction effects of a piled raft

(3) Analysis of a piled raft may be based on numerical modelling including nonlinear stress–strain models for the ground, the structural flexural stiffness of the raft and the interactions between ground, raft and piles.

(4) Verification of the ultimate limit state of individual piles within a piled raft may be omitted provided an ultimate limit state of the combined structure is not exceeded.

(5) The ultimate compressive resistance of a piled raft may be determined in a simplified manner by neglecting pile resistances and considering the ultimate compressive resistance of the raft alone R_{raft} according to 5.5.2.2 and 5.6.3.

(6) Provided that an ultimate limit state in the combined structure is not exceeded, the shaft and base resistances of individual piles used for settlement reduction of a raft foundation may be allowed to reach their limiting value.

NOTE 1 This is particularly beneficial when piles are used for the purpose of settlement or raft bending moment reduction.

NOTE 2 The limiting value here is not necessarily the same as that of a single pile, since it includes pile-raft interaction effects, especially the surcharge effect and the restrain provided by the raft in contact with the ground.

6.5.7 Displacement of piled foundations

6.5.7.1 General

(1) The settlement and transverse displacement of a piled foundation shall be determined from the results of load tests; analytical, numerical or empirical calculations, or prescriptive rules based on the observed performance of comparable single piles or pile groups.

NOTE Load testing of pile groups is seldom feasible, and so the performance of pile groups is normally verified by other methods.

(2) The validity of analytical, numerical and empirical calculation methods should be demonstrated using documented load tests on and case histories of comparable pile foundations to confirm that the methods provide reliable parameter values and predictions of pile settlement and transverse displacement.

(3) Potential downdrag shall be considered for both serviceability and ultimate conditions and shall take account of the relevant pile foundation loading and the strain mechanisms between the piles and the surrounding fill or soil in accordance with 6.5.2.

6.5.7.2 Single piles

(1) The settlement and transverse displacement of a single pile may be determined from load tests or calculated using empirical or analytical methods or numerical modelling.

NOTE Owing to rapid degradation of mobilized ground stiffness with pile head movement, calculation models based on nonlinear stiffness are particularly appropriate for calculating the transverse response of a pile foundation.

(2) Elastic shortening of the pile shaft under axial compression should be included in the calculation of pile head settlement taking into account the effects of creep.

6.5.7.3 Pile groups and piled rafts

(1) The settlement and transverse displacement of pile groups and piled rafts may be determined using empirical or analytical methods or numerical modelling.

(2) Calculation methods for pile group design should take account of:

– the load-displacement behaviour of individual piles as well as behaviour of pile group;
– the movement and loading effects caused by pile to pile interaction through the ground;
– the interaction with the supported structure.

NOTE Examples of appropriate methods include finite element/difference, boundary element, and interaction factor approaches.

(3) Load transfer functions should not be used to determine groups effects unless the they account for interaction between the piles.

(4) Interactions between piles should consider the non-linear behaviour of the ground.

NOTE Methods based on purely linear behaviour tend to overestimate pile displacement at working load.

6.5.8 Confirmation of pile design by site-specific load testing or comparable experience

(1) Pile design should be validated using site-specific static load testing to confirm design parameter values, verify compressive or tensile resistance, and establish behaviour under serviceability limit state conditions.

NOTE Unlike static load tests, rapid load and dynamic impact tests do not provide direct information about the pile behaviour under serviceability limit state conditions.

(2) Pile resistance to axial compression may be confirmed using dynamic impact or rapid load tests provided that these tests have been validated by static pile load tests.

(3) Site-specific ultimate control test may be omitted where there is comparable experience or evidence of previous successful use for the same pile type, with similar geometry, installed in similar ground conditions.

(4) The number and type of site-specific pile loads tests n_{test} needed to confirm pile design by calculation may be selected based on the type and purpose of the load test.

NOTE Values of n_{test} are given in Table 6.2 (NDP) unless the National Annex gives different values.

Table 6.2 — (NDP) Minimum quantity of load testing for confirmation of pile design by calculation

Type of load test	Confirmation of design by Ultimate Control Tests	Confirmation of design by Serviceability Control Tests
Static load test	max (1, 0.5 % N)	max (2, 1 % N)
Rapid load test	max (3, 1.0 % N)	max (6, 5 % N)
Dynamic impact load test	max (3, 1.0 % N)	max (6, 5 % N)
NOTE N = total number of piles in similar ground conditions		

(5) When selecting the value of ntest, piles with different geometries may be considered as a single set of tests, provided they are anticipated to exhibit a similar response to loading.

(6) The value of n_{test} may be adjusted proportionately when carrying out both Ultimate and Serviceability Control Tests or when carrying out a mix of static, rapid, or dynamic impact load tests.

(7) All pile load test should be carried out in accordance with 6.9.

(8) The design of piles shall consider any adverse effect of Control Tests on the load-settlement behaviour of the test pile during its design service life.

6.6 Ultimate limit states

6.6.1 Single piles

6.6.1.1 Verification of axial compressive resistance

(1) The axial compressive resistance of a single pile shall be verified using Formula 6.10):

$$F_{\mathrm{cd}} \leq R_{\mathrm{cd}} \tag{6.10}$$

where

F_{cd} is the design axial compression applied to the pile including an allowance for any potential drag force (see 6.6.1.4);

R_{cd} is the pile's design axial compressive resistance.

NOTE R_{cd} includes cyclic degradation effects where applicable.

(2) The design axial compressive resistance R_{cd} shall be determined from Formula (6.11):

$$R_{\mathrm{cd}} = \frac{R_{\mathrm{c,rep}}}{\gamma_{\mathrm{Rc}} \cdot \gamma_{\mathrm{Rd}}} \ or \ \left(\frac{R_{\mathrm{b,rep}}}{\gamma_{\mathrm{Rb}} \cdot \gamma_{\mathrm{Rd}}} + \frac{R_{\mathrm{s,rep}}}{\gamma_{\mathrm{Rs}} \cdot \gamma_{\mathrm{Rd}}}\right) \tag{6.11}$$

where

$R_{\mathrm{c,rep}}$ is the pile's representative total resistance in axial compression;

$R_{b,rep}$ is the pile's representative base resistance in axial compression;

$R_{s,rep}$ is the pile's representative shaft resistance in axial compression;

γ_{Rd} is a model factor;

γ_{Rc}, γ_{Rb}, γ_{Rs} are resistance factors given in 6.6.3.

NOTE 1 Values of γ_{Rd} are given in Table 6.3 (NDP) for verification by calculation for compressive and tensile actions unless the National Annex gives different values.

NOTE 2 Value of γ_{Rd} are given in Table 6.4 (NDP) for verification by testing for compressive and tensile action, unless the National Annex gives different values.

Table 6.3 — (NDP) Model factor γRd for verification of axial pile resistance by calculation

Verification by		**Model factor γ_{Rd}**	
Ground Model Method	Ultimate Control Tests	1.2	
	Extensive comparable[a,b] experience without site-specific Control Tests	1.3	
	Serviceability Control Tests	1.4	
	No pile load tests and limited comparable experience[a,c]	1.6	
	Pile on competent rock using properties determined from field and laboratory tests	1.1	
		Compressive resistance	**Tensile resistance**
Model Pile Method	Pressuremeter test[d]	1.15	1.4
	Cone penetration test[d]	1.1	1.1
	Profiles of ground properties based on field or laboratory tests[d,e]	1.2	1.2

[a] Comparable experience assumes documented records (or database) of static pile load test results conducted on similar piles, in similar ground conditions, under similar loading conditions from a certain number of sites n,

[b] Extensive comparable experience assumes $n \geq 10$

[c] Limited comparable experience assumes $n < 10$

[d] Value can be multiplied by 0.9 when accompanied by Ultimate Control Tests

[e] Ground strength properties determined at maximum vertical spacings of 1.5 m

Table 6.4 — (NDP) Model factor γ_{Rd} for verification of axial pile resistance by testing

Verification by		Model factor γ_{Rd}			
		Fine soils	Coarse soils	Rock	Competent rock
Static load tests		1.0	1.0	1.0	1.0
Rapid load tests (multiple load cycles)[a]		1.4	1.1	1.2	1.1
Rapid load tests (single load cycle)[a]		1.4	1.1	1.2	1.1
Dynamic impact tests (signal matching)[b]	Shaft bearing	1.5	1.1	1.2	1.1
	End bearing	1.4	1.25	1.25	1.15
Dynamic impact tests (multiple blow)[b]	Shaft bearing	1.5	1.1	1.2	1.1
	End bearing	1.4	1.2	1.2	1.1
Dynamic impact tests (closed form solutions)[b]	Shaft bearing	Not permitted	Not permitted	Not permitted	1.3
	End bearing	Not permitted	1.3	1.3	1.3
Wave equation analysis		Not permitted	1.6	1.5	1.4
Pile driving formulae		Not permitted	1.8	1.7	1.5

[a] When dynamic impact tests are not calibrated by site-specific static load testing, but by comparable experience only (see Table 6.3 (NDP)), the values for γ_{Rd} are increased by::

+0.1 when calibration is based on extensive comparable experience;

+0.25 when calibration is based on limited comparable experience.

[b] When dynamic impact tests are carried out on cast-in-place piles, the values for γ_{Rd} are increased by 0.2

6.6.1.2 Verification of axial tensile resistance

(1) The axial tensile resistance of a single pile shall be verified using Formula (6.12):

$$F_{\mathrm{td}} \leq R_{\mathrm{td}} \tag{6.12}$$

where

F_{td} is the design axial tension applied to the pile;

R_{td} is the pile's design axial tensile resistance.

(2) The design axial tensile resistance R_{td} shall be determined from Formula (6.13):

$$R_{\mathrm{td}} = \frac{R_{\mathrm{t,rep}}}{\gamma_{\mathrm{Rst}} \cdot \gamma_{\mathrm{Rd}}} \tag{6.13}$$

where

$R_{\mathrm{t,rep}}$ is the pile's representative axial tensile resistance;

γ_{Rd} is a model factor;

γ_{Rst} is a resistance factor, specified in 6.6.3.

NOTE 1 Values of γ_{Rd} are given in 6.6.1.1

NOTE 2 R_{td} include potential cyclic degradation effects.

6.6.1.3 Verification of transverse resistance

(1) The transverse resistance of a single pile shall be verified using Formula (6.14):

$$F_{\mathrm{tr,d}} \leq R_{\mathrm{tr,d}} \tag{6.14}$$

where:

$F_{\mathrm{tr,d}}$ is the design transverse force applied to the pile including an allowance for any potential transverse force due to moving ground (see 6.6.1.5);

$R_{\mathrm{tr,d}}$ is the pile's design transverse resistance.

(2) If using the material factor approach, the design transverse resistance $R_{\mathrm{tr,d}}$ shall be determined according to prEN 1990:2021, Formula (8.12), by applying material factors γ_{M} to the representative values of the material properties X_{rep}.

NOTE The values of γM is given in prEN 1997-1:2022, 4.4.1

(3) If using the resistance factor approach, the design transverse resistance $R_{\mathrm{tr,d}}$ shall be determined according to prEN 1990:2021, Formula (8.13), by applying resistance factors $\gamma_{\mathrm{R,tr}}$ to the representative transverse resistance of the single pile $R_{\mathrm{tr,rep}}$.

NOTE The value of $\gamma_{\mathrm{R,tr}}$ is given in 6.6.3

6.6.1.4 Downdrag

(1) Downdrag should be classified as a permanent action arising from the relative axial movement when ground settlement exceeds pile settlement.

(2) The design drag force due to settling ground shall be determined from Formula (6.15):

$$D_{\mathrm{d}} = \gamma_{\mathrm{F,drag}} D_{\mathrm{rep}} \tag{6.15}$$

where:

D_{d} is the design drag force due to moving ground;

D_{rep} is the representative drag force due to moving ground;

$\gamma_{\mathrm{F,drag}}$ is a partial action factor given in 6.6.3.

6.6.1.5 Transverse ground loading

(1) Transverse forces on the pile due to moving ground should be classified as permanent actions arising from relative transverse movement between the ground and the pile.

6.6.1.6 Representative values of resistance

(1) For design by calculation using the Ground Model Method, the representative value of resistance of a single pile R_{rep} shall be determined from Formula (6.16):

$$R_{\mathrm{rep}} = R_{\mathrm{calc}} \tag{6.16}$$

where:

R_{rep} is $R_{c,rep}$ for compression, $R_{t,rep}$ for tension, or $R_{tr,rep}$ for transverse resistance, as appropriate;

R_{calc} is the calculated pile resistance based on ground parameters.

(2) For design by calculation using the Model Pile Method, the representative value of resistance of a single pile R_{rep} shall be determined from Formula (6.17):

$$R_{\text{rep}} = min\left\{\frac{(R_{\text{calc}})_{\text{mean}}}{\xi_{\text{mean}}};\frac{(R_{\text{calc}})_{\text{min}}}{\xi_{\text{min}}}\right\} \tag{6.17}$$

where:

$(R_{calc})_{mean}$ is the mean calculated pile resistance for a set of profiles of field test results;

$(R_{calc})_{min}$ is the minimum calculated pile resistance for a set of profiles of field test results;

ξ_{mean} is a correlation factor for the mean of the (calculated) values;

ξ_{min} is a correlation factor for the minimum of the (calculated) values.

NOTE Values of ξ_{mean} and ξ_{min} are given in Table 6.5 (NDP) unless the National Annex gives different values.

Table 6.5 — (NDP) Correlation factors

Correlation Factor[a,b]	Coefficient of variation (CoV)	Number of tests or profiles								
		1	2	3	4	5	7	10	20	≥ 50
ξ_{min}	n/a	1.4	1.27	1.23	Use ξ_{mean} alone					
ξ_{mean}	≤ 12 %	Use ξ_{min} alone		1.30	1.28	1.28	1.27	1.26	1.25	1.25
	15 %			1.40	1.39	1.38	1.37	1.36	1.36	1.35
	20 %			1.67	1.64	1.63	1.61	1.60	1.59	1.58
	25 %			1.98	1.95	1.93	1.90	1.89	1.87	1.85
	≥ 25 %	Sub-divide the Geotechnical Design Model to reduce the CoV								

[a] If all piles in a group are tested, use ξ_{mean} = 1.0 provided load can be transferred through the pile cap. For individually tested piles, use $\xi_{mean} = \xi_{min}$ = 1.0.

[b] The correlation factors given here assume field test profiles arranged on a grid with reference spacing d_{ref} of 30 m

(3) Profiles of field test results shall only be considered as a single data set if they are obtained in an area of the site with similar ground conditions and over similar depths as the installed piles.

(4) For each single data set defined in (3), the coefficient of variation (CoV) of the computed pile resistance for each profile should be determined.

(5) The values of the correlation factors ξmean and ξmin for the Model Pile Method shall be determined based on the number of profiles in the single data set and the coefficient of variation determined in (4).

(6) For design by testing, the representative value of resistance of a single pile R_{rep} shall be determined from Formula (6.18):

$$R_{\text{rep}} = min\left\{\frac{(R_{\text{test}})_{\text{mean}}}{\xi_{\text{mean}}};\frac{(R_{\text{test}})_{\text{min}}}{\xi_{\text{min}}}\right\} \tag{6.18}$$

where:

$(R_{test})_{mean}$ is the mean pile resistance measured in a set of load tests;

$(R_{test})_{min}$ is the minimum pile resistance measured in a set of load tests;

ξ_{mean} is a correlation factor for the mean of the (measured) values;

ξ_{min} is a correlation factor for the minimum of the (measured) values.

(7) Results of pile load tests shall only be considered as a single data set if they relate to similar pile types, pile geometry, loading conditions, and ground conditions.

(8) The values of ξ_{mean} and ξ_{min} may be reduced by 10 % for pile groups or piled rafts that are able to redistribute load from a single pile to other piles in the group without any significant additional settlement of the foundation provided the value of the final correlation factor is not less than 1.0.

(9) If ξ_{mean} and ξ_{min} are reduced according to (8), then the verification of limit states in the pile cap shall consider the load redistribution.

(10)The values of ξ_{mean} and ξ_{min} may be calculated by considering corresponding to the number of test profiles N in the area S:

$$\xi_{\mathrm{mean}}(S) = 1 + \frac{\mathrm{d}}{d_{ref}}(\xi_{\mathrm{mean}} - 1) \text{ or } \xi_{\mathrm{min}}(S) = 1 + \frac{d_{ave}}{d_{ref}}(\xi_{\mathrm{min}} - 1) \tag{6.19}$$

where:

$\xi_{mean}(S)$ is the value of ξ_{mean} by considering the area S corresponding to the number of test profiles N;

$\xi_{min}(S)$ is the value of ξ_{min} by considering the area S corresponding to the number of test profiles N;

d_{ave} is the average distance between the N test profiles located in the area S;

d_{ref} is the reference spacing of 30 m for the Model Pile Method.

NOTE Formula (6.19) is applied unless the National Annex provides different formula.

6.6.2 Pile groups and piled rafts

(1) The design resistance of a pile group or piled raft $R_{d,group}$ shall be verified using Formula (6.20):

$$F_{\mathrm{d,group}} \leq R_{\mathrm{d,group}} \tag{6.20}$$

where:

$F_{d,group}$ is the design action applied to the pile group or piled raft;

$R_{d,group}$ is the design resistance of the pile group or piled raft.

(2) If using the material factor approach, the design resistance $R_{d,group}$ shall be determined according to prEN 1990:2021, Formula (8.12), by applying material factors γ_M to the representative values of the material properties X_{rep}.

NOTE The valued of γM is given in prEN 1997-1:2022, 4.4.1

(3) If using the resistance factor approach the design resistance $R_{\mathrm{d,group}}$ for vertical resistance may be determined from Formula (6.21):

$$R_{\mathrm{d,group}} = \frac{R_{\mathrm{rep,group}}}{\gamma_{\mathrm{R,group}}\gamma_{\mathrm{Rd,group}}}\ or\ R_{d,piled-raft} = \left(\frac{\sum_i^n R_{\mathrm{c,rep,i}}}{\gamma_{\mathrm{Rd}}\gamma_{\mathrm{Rc}}} + \frac{R_{\mathrm{rep,raft}}}{\gamma_{\mathrm{R,raft}}}\right) \quad (6.21)$$

where:

$\gamma_{\mathrm{R,group}}$ is a resistance factor for the pile group axial compressive resistance;

γ_{Rc} is a resistance factor for individual pile axial compressive resistance;

$\gamma_{\mathrm{R,raft}}$ is a resistance factor for the raft, given in 6.6.3;

$\gamma_{\mathrm{Rd,group}}$ is a model factor for the pile group or piled raft.

$\gamma_{\mathrm{R,d}}$ Is a model factor for a single pile, given in 6.6.1.1

NOTE The value of $\gamma_{\mathrm{Rd,group}}$ is 1.0, unless the National Annex gives different values.

6.6.3 Partial factors

6.6.3.1 Single piles

(1) Partial factors for the verification of the axial resistance of single piles at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using either the Resistance Factor Approach in combination with either the Ground Model Method or the Model Pile Method.

NOTE 1 Values of the partial factors for single piles are given in Table 6.6 (NDP) for persistent and transient design situations and for accidental design situations unless the National Annex gives different values.

NOTE 2 Either the Model Pile Method or the Ground Model Method can be used, unless the National Annex specifies otherwise.

Table 6.6 — (NDP) Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations and accidental situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA) – both combinations		Resistance factor approach (RFA)				
			(a)	(b)	Pile class	model pile		ground model	
Axial compressive resistance	Actions and effects-of-actions[1]	γ_F and γ_E	Not Used		All	VC1			
	Drag force due to settling ground	$\gamma_{F,drag}$				1.35 (1.0)[d]			
	Ground properties[2]	γ_M				Not factored			
	Base and shaft resistance in compression	γ_{Rb} | γ_{Rs}				Base	Shaft	Base	Shaft
					Full displacement	1.2 (1.1)[d]		1.2 (1.1)[d]	1.05 (1.0)[d]
					Partial displacement	1.2 (1.1)[d]	1.0 (1.0)[d]	1.3 (1.15)[d]	1.05 (1.0)[d]
					Replacement	1.2 (1.1)[d]		1.4 (1.2)[d]	1.15 (1.05)[d]
					Unclassified	1.35 (1.15)[d]	1.25 (1.1)[d]	1.5 (1.25)[d]	1.25 (1.1)[d]
	Total resistance in compression	γ_{Rc}			Full displacement			1.1 (1.05)[d]	
					Partial displacement			1.2 (1.1)[d]	
					Replacement			1.3 (1.15)[d]	
					Unclassified	1.3 (1.15)[d]		1.4 (1.2)[d]	
Axial tensile resistance	Actions and effects-of-actions[a]	γ_F and γ_E	Not Used		All	DC1			
	Ground properties[b]	γ_M				Not factored			
	Shaft resistance in tension	γ_{Rst}			Full displacement	1.15 (1.05)[d]		1.2 (1.1)[d]	
					Partial displacement			1.2 (1.1)[d]	
					Replacement			1.3(1.15)[d]	
					Unclassified	1.4 (1.2)[d]		1.5 (1.25)[d]	
Transverse resistance	Actions and effects-of-actions[a,c]	γ_F, and γ_E	vC4 (EFA[e]	vC3	Not used				
	Ground properties[b]	γ_M	M1	M2	Not factored				

Verification of	Partial factor on	Symbol	Material factor approach (MFA) – both combinations		Resistance factor approach (RFA)		
			(a)	(b)	Pile class	model pile	ground model
	Transverse resistance	γ_{Rtr}	Not factored		Not used		

a Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in prEN 1990:2021 Annex A. For transverse resistance, DC1 may be used as alternative to VC4.

B Values of the partial factors for Sets M1 and M2 are given in prEN 1997-1:2022, Table 4.7

c Including drag force due to moving ground.

D Values in brackets are given for accidental design situations.

E See prEN 1997-1 :2022, 8.2

6.6.3.2 Pile groups and piled rafts

(1) Partial factors for the verification of pile groups and piled rafts at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1 using either the Material Factor Approach or the Resistance Factor Approach.

NOTE 1 Values of the partial factors for pile groups and piled rafts are given in Table 6.7 (NDP) for persistent, transient, and accidental design situations unless the National Annex gives different values.

NOTE 2 The National Annex can specify which Factor Approach to use.

Table 6.7 — (NDP) Partial factors for the verification of ultimate resistance of pile groups and piled rafts for fundamental (persistent and transient) design situations and accidental situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA) – both combinations		Resistance factor approach (RFA)
			(a)	(b)	
Vertical resistance	Actions and effects-of-actions[a]	γ_F and γ_E	VC4	VC3	VC1
	Ground properties[b]	γ_M	M1	M2	Not factored
	Vertical resistance	$\gamma_{R,group}$	Not factored		1.4 (1.1)[c]
		γ_{Rc}			See Table 6.6 (NDP)
		$\gamma_{R,raft}$			1.4 (1.1)[c]
Combined axial and transverse resistance (see prEN 1997-1:2022, 8.2)	Actions and effects-of-actions[a]	γ_F and γ_E	DC4 (EFA)[d]	DC3	Not used
	Ground properties[b]	γ_M	M1	M2	
	Compressive and transverse resistance	$\gamma_{R,group}$	Not factored		

A Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in prEN 1990;2021 Annex A.

b Values of the partial factors for Sets M1 and M2 are given in prEN 1997-1:2022, Table 4.7.

Verification of	Partial factor on	Symbol	Material factor approach (MFA) – both combinations		Resistance factor approach (RFA)
			(a)	(b)	
[c] Values in brackets are given for accidental situations. [D] See prEN 1997-1:2022, 8.2					

6.6.4 Structural design and verification

(1) The structural resistance of single piles should be verified in accordance with:

- prEN 1992-1-1 for reinforced and plain concrete, grout or mortar piles;
- prEN 1993-1-1 and EN 1993-5 for steel piles;
- EN 1994-1-1 for composite steel and concrete piles;
- EN 1995-1-1 for timber piles.

(2) Ground stiffness should be determined considering the magnitude of any axial or transverse displacement of the pile.

(3) The representative value of stiffness should be selected as either an upper or lower value, depending on which is more critical.

NOTE Upper values are sometimes critical when transversal loads are present (e.g. from settling soil).

(4) Bending stresses due to initial curvature, eccentricities and induced deflection should be considered together with stresses due to transverse load.

(5) Buckling and torsional stability should be verified considering second order effects, particularly for long slender piles.

NOTE Annex C.13 provides calculation models to take into account buckling and second order effects.

(6) For piles and rigid inclusions subjected to compression, the structural resistance and buckling should be verified by theory of second order when the following conditions are met:

- Pile diameter $B<B_{\mathrm{ref}}$;
- Pile length embedded in soil layers with a thickness of h>href and with a shear strength in total stress analyses $c_u<c_{u,\mathrm{ref}}$.

NOTE 1 B_{ref} = 0.3 m, h_{ref} = 1.0 m and $c_{u,\mathrm{ref}}$ = 15 kPa unless the National Annex gives other values.

NOTE 2 Example of second order theory is given in Annex C.13

6.7 Serviceability limit states

(1) prEN 1997-1:2022, 9 shall apply to piled foundations.

(2) Serviceability behaviour of piled foundations shall be determined in accordance with 6.5.7.

(3) Explicit verification of the serviceability of a piled foundation may be omitted provided serviceability performance of the piled foundation can be demonstrated by comparable experience.

(4) Explicit verification of settlement may be omitted for single piles loaded in compression when founded in medium to dense coarse soils, medium to high strength fine soils, or rock, provided the inequality given in Formula (6.22) is verified:

$$F_{\mathrm{cd,SLS}} \leq \kappa_{\mathrm{b,SLS}} R_{\mathrm{b,rep}} + \kappa_{\mathrm{s,SLS}} R_{\mathrm{s,rep}} \tag{6.22}$$

where:

$F_{\mathrm{cd,SLS}}$ is the design axial compression applied to the pile with the quasi-permanent and characteristic serviceability limit state combinations, including potential downdrag forces;

$R_{\mathrm{b,rep}}$ is the representative value of base resistance;

$R_{\mathrm{s,rep}}$ is the representative value of shaft resistance;

$\kappa_{\mathrm{b,SLS}}$ is a mobilization factor for base resistance in the serviceability limit state;

$\kappa_{\mathrm{s,SLS}}$ is a mobilization factor for shaft resistance in the serviceability limit state.

NOTE The values of $\kappa_{\mathrm{b,SLS}}$ and $\kappa_{\mathrm{s,SLS}}$ are respectively 0.1 and 0.85 unless the National Annex gives different values.

(5) Verification of the serviceability limit state for pile groups and piled rafts should be based on modelling that accounts for non-linear stiffness of the ground, flexural stiffness of the structure, and interaction between the ground, structures, and piles.

6.8 Implementation of design

6.8.1 General

(1) prEN 1997-1:2022, 10 shall apply to piled foundations.

(2) The execution of piled foundations should comply with the following execution standards:

- EN 1536 for bored piles;
- EN 12699 for displacement piles;
- EN 14199 for micropiles;
- EN 12063 for sheet piles used for bearing resistance;
- EN 1538 for diaphragm walls for bearing resistance;
- EN 12716 for jet grouting;
- EN 14679 for deep mixing.

6.8.2 Inspection

6.8.2.1 General

(1) In addition to prEN 1997-1:2022, 10.3, the Inspection Plan should include:

- the location and general layout of the piled foundations;
- the sequence of works;
- the working level and working platform;
- rig monitoring and instrumentation;
- non-destructive integrity tests.

6.8.2.2 Rig monitoring and instrumentation

(1) For continuous flight auger and continuous helical displacement piles, the piling rig should be fitted with a suitable automated instrumentation and monitoring system capable of measuring the execution metrics throughout the boring and concreting of the pile.

(2) Piling rigs used to install driven displacement piles should be fitted with a suitable automated instrumentation and monitoring system capable of measuring the execution metrics throughout the pile driving process.

(3) Installation and monitoring records should be inspected after pile execution to verify conformance of the pile to its design criteria.

6.8.2.3 Non-destructive integrity tests

(1) Cast-in-place or precast concrete piles may be subject to non-destructive integrity testing to verify the pile does not include any defects within the shaft and has not been damaged during installation.

(2) The method for integrity testing may be chosen from the following:

– low strain Pile Integrity Test;
– thermal integrity profiling;
– cross-hole sonic logging method;
– distributed fibre optic sensing method.

(3) Results of dynamic impact load testing may also be used to verify pile shaft integrity.

(4) For driven precast concrete piles, the need of integrity tests may be based on evaluation of the driving based on observations and discontinuities in the drive blow record.

6.8.3 Monitoring

(1) In addition to prEN 1997-1:2022, 10.4, the Monitoring Plan for piled foundations should comply with the execution standards.

(2) In addition to prEN 1997-1:2022, 10.4, the Monitoring Plan should include, but is not limited to, the following:

– settlement, lateral and distortion measurements of the supported structure;
– vibration measurements;
– settlement, lateral and distortion measurements of nearby sensitive structures.

(3) Monitoring of pile execution should be carried out for all piles over the full depth of each pile and should include, but is not limited to:

– piling rig monitoring and instrumentation records;
– drive blow and hammer energy records for driven piles;
– visual inspection of spoil and observations of ground conditions for auger bored and drilled piles.

NOTE Piling rig monitoring and instrumentation records can include pull-down force, duration per depth, penetration per revolution, torque.

(4) Installation and monitoring records should be inspected after pile execution to verify conformance of the pile to its design criteria.

6.8.4 Maintenance

(2) In addition to prEN 1997-1:2022, 10.5, the Maintenance Plan of piled foundations should comply with the execution standards.

6.9 Testing

6.9.1 General

(1) prEN 1997-1:2022, 11 shall apply for piles.

(2) Pile load tests should conform to the following standards:

- EN ISO 22477-1 for static compression load testing;
- prEN ISO 22477-2 for static tension load testing;
- prEN ISO 22477-3 for transverse load testing;
- EN ISO 22477-4 for dynamic load testing;
- EN ISO 22477-10 for rapid load testing.

(3) Ultimate Control Tests shall be carried out when verification of limit states is to be based on the results of pile load testing.

(4) Ultimate Control Tests should be performed when using a pile type or installation method for which there is no comparable experience or when piles have not previously been tested under comparable ground or loading conditions.

(5) Serviceability Control Tests should be carried out on working piles during the main piling works for the purpose of verifying acceptable pile movement.

(6) Control Tests should also be carried out when observations during pile execution indicates conditions that deviate from the anticipated Ground Model.

(7) Inspection Tests should be carried out to verify the integrity of all piles susceptible to installation damage or other piles when execution procedures cannot be monitored in a reliable way.

6.9.2 Trial piles

(1) Trial piles should be installed and tested before commencement of the piling works to confirm the chosen pile type, its design, dimensions, resistance, and performance.

(2) If only one trial pile is installed, it should be located in the most adverse ground conditions identified on the project site.

(3) Execution of the trial pile shall be performed in an identical manner to that proposed for the working piles and shall comply with the execution standards.

(4) In cases where it is impractical to install or construct full-size large diameter trial piles, a smaller diameter trial pile may be installed provided that:

- the ratio of the trial pile to working pile diameter is not less than 0.5;
- the trial pile is constructed or installed in an identical manner to the proposed working piles;
- the trial pile is instrumented to allow separation of the base and shaft resistance during any test.

6.9.3 Test proof load

(1) The test proof load shall be determined allowing for potential drag force, transverse ground force, and temporary support load.

(2) The proof load P_P for Ultimate Control Tests shall be determined from Formula (6.23):

$$P_P \geq R_{rep} + D_{sup} \tag{6.23}$$

where:

R_{rep} is the representative value of the pile's ultimate resistance, estimated from previous load testing, calculation, or comparable experience;

D_{sup} is the vertical temporary support force provided by the ground.

(3) The value of D_{sup} should be estimated using superior (upper) ground strength and stiffness properties.

(4) In presence of a significant vertical temporary support force provided by the ground, the pile should be instrumented.

(5) When the pile ultimate resistance is unknown at the time of test, the proof load P_p may be determined from Formula (6.24):

$$P_P \geq \gamma_{Rd} \cdot \xi \cdot \gamma_R \cdot F_{d,ULS} + D_{add} + D_{sup} \tag{6.24}$$

where:

γ_{Rd} is the model factor used in the verification of ultimate resistance;

ξ is the correlation factor (if any) used in the verification of ultimate resistance;

γ_R is the resistance factor to be used in the verification of ultimate resistance;

$F_{d,ULS}$ is the design action at the ultimate limit state excluding any drag force or transverse force as appropriate to the type of load test.

(6) The test proof load P_P for Serviceability Control Tests shall be determined from Formula (6.25):

$$P_P = \gamma_{test} \cdot F_{d,SLS} + D_{add} + D_{sup} \tag{6.25}$$

where:

γ_{test} is a partial factor;

$F_{d,SLS}$ is the design action at the serviceability limit state of the quasi-permanent combination excluding any drag force or transverse force as appropriate to the type of load test.

NOTE The value of γ_{test} is 1.35, unless the National Annex gives a different value.

(7) Determination of the proof load for transverse load testing should take account of the level at which the applied load or transverse force from moving ground is to be applied and any differences in geometry and head fixity of the test pile compared to the pile under service conditions.

6.9.4 Static load tests

(1) Static load tests in compression should comply with EN ISO 22477-1.

(2) The interpretation of load testing should take account of the systematic and random variations that exist in the ground and the variability of the test pile installation and its influence when deriving the pile's resistance.

(3) Separation of the base and shaft resistance components from a static compression load test may be performed using instrumented test piles or specialist testing procedures.

(4) In an Ultimate Control Test, the ultimate compressive resistance shall be determined as the load corresponding to a downward plunging failure of the pile, with adjustments for temporary support resistance.

(5) The ultimate compressive resistance should be mathematically defined as the resistance corresponding to infinite settlement.

(6) Provided the Ultimate Control Test has been taken to a sufficiently high load level to mobilise a large proportion of the base resistance, an extrapolated asymptotic value of pile compressive resistance at infinite movement may be adopted.

(7) As an alternative to (5) and (6), the ultimate compressive resistance may be determined as:

- the maximum applied test load; or
- the test load at a pile head settlement equal to 10 % of the pile's base diameter.

(8) For a tension load test, the ultimate tension resistance R_t shall be determined as the load corresponding to pull-out failure of the pile corresponding to infinite vertical displacement.

NOTE The limiting criteria to be used is as specified by the relevant authority or where not specified, as agreed for a specific project by the relevant parties.

(9) Interpretation of horizontal load test results shall take account of the different deformation mechanism between a load test carried out on a free-headed pile and the in-service behaviour where the pile caps and sub-structure can result in significant head fixity to the pile.

NOTE 1 It is unlikely that a horizontal load test can achieve sufficient displacement to fully mobilize the resistance of the ground to any appreciable depth.

NOTE 2 Under test conditions, the behaviour of the pile will be dominated by the strength, stiffness and variability of the ground over the top few metres of the pile. The pile diameter due to oversized or undersized ores and the concrete rate stiffness dependency will also affect the results.

6.9.5 Rapid load tests

(1) Rapid load tests should comply with EN ISO 22477-10.

(2) The compressive pile resistance R_c determined from the results of a rapid load test should be set equal to the maximum frictional resistance, with allowance for temporary support resistance.

(3) For rapid load tests carried out on piles installed in fine fills and soils, an additional allowance for potential consolidation and creep should be applied.

6.9.6 Dynamic impact tests

(1) Dynamic impact load tests should comply with EN ISO 22477-4.

(2) The compressive pile resistance R_c determined from the results of a rapid load test should be set equal to the maximum frictional resistance, with allowance for any drag force or temporary support resistance.

(3) Where Ultimate Control Tests using dynamic load test are used to confirm design by calculation or testing, the pile's total resistance and an estimate of its shaft and base resistances may be determined from an analysis of test measurements using signal matching.

6.10 Reporting

(1) In addition to prEN 1997-1:2022, 12, pile test reports shall include full details of the pile execution including type of pile, method of installation, size, length, material properties, and other observations made during installation.

(2) Pile load test reports shall comply with 6.9.4-6.9.6 and the test standards given in 6.9.1.

(3) In addition to (2), pile load test reports shall include applied load and displacement measurements at all stages of the test, together with results of any instrumentation or external measurements.

7 Retaining structures

7.1 Scope and field of application

(1) This Clause shall apply to structures that retain ground, groundwater, engineered fill, and surface water.

7.2 Basis of design

7.2.1 Design situations

(1) prEN 1997-1:2022, 4.2.2 shall apply to retaining structures.

7.2.2 Geometrical properties

7.2.2.1 General

(1) prEN 1997-1:2022, 4.3.3 shall apply to retaining structures.

7.2.2.2 Ground surfaces

(1) Values for the geometry of the retained material shall take account of any variation in actual field values and anticipated excavation or possible scour or erosion in front of the retaining structure.

NOTE Anticipated excavation includes post-construction excavation in front of the structure, e.g. due to buried services maintenance.

(2) The design level of the resisting ground should be lowered below the nominal level by an amount Δa given by:

– for a cantilever wall, $\Delta a = \min(0.1\ H; 0.5\ \text{m})$, where H is wall height above excavation level;

– for a supported wall, Δa = min(0.1 h_s; 0.5 m), where h_s is the distance between the lowest support and excavation level at each construction stage.

(3) Values of Δa smaller than those given in (2), including $\Delta a = 0$, may be used when the surface level is specified to be controlled reliably throughout the relevant execution period.

(4) Values of Δa larger than those given in (2) should be used when the surface level is particularly uncertain.

NOTE This can be relevant for marine structures during dredging operations or for erosion conditions.

7.2.3 Zone of influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to retaining structures.

7.2.4 Actions and environmental influences

7.2.4.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to retaining structures.

7.2.4.2 Permanent and variables actions

(1) Actions for retaining structures shall include, but are not limited to:

– stages of excavation, construction, operation, and maintenance;
– anticipated future structures or any anticipated future loading or unloading within the zone of influence of the geotechnical structure;
– effects on waterfront structures, ice, and wave force;
– potential adverse effects of repeated surcharge loading;
– potential actions arising from temperature changes in struts or integral bridges.

NOTE Seismic actions are defined in EN 1998 (all parts)

(2) Loads that act within the zone of influence may be considered as concentrated or uniform depending on their nature and proximity to the retaining structure.

7.2.4.3 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to retaining structures.

7.2.4.4 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.5 shall apply to retaining structures.

(2) The adverse effects of temperature changes shall be considered, especially when determining the loads in struts and props due to wall movements.

NOTE Direct sunlight effects can often be reduced by specific measures, such as coating or painting.

(3) Measures should be taken to prevent frost heave and potential ice lenses forming in the ground behind a retaining structure.

NOTE 1 Frost heave can occur in frost susceptible soil, especially in silt.

NOTE 2 Formation of ice lenses can occur in silt with access to free water leading to a significant volume expansion of the soil.

NOTE 3 Possible measures include selection of suitable backfill material, drainage, or insulation.

7.2.5 Limit states

7.2.5.1 Ultimate Limit States

(1) In addition to the limit states specified in prEN 1997-1:2022, 8.1, the following ultimate limit states shall be verified for all retaining structures:

– failure of a structural element, including the wall, anchor, rock bolt, corbel, or strut;
– failure of the connection or interface between structural elements;
– combined failure in the ground and in the structural element;
– excessive movement of the retaining structure, which may cause collapse of the structure or nearby structures or services that rely on it (see prEN 1997-1:2022, 8.1.2 (1)).

(2) Potential ultimate limit states other than those given in (1) should be verified.

(3) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be considered for gravity walls and for composite retaining structures:

— bearing resistance failure of the ground below the base, taking into account eccentricity and inclination of loads;
— failure by sliding along the base;
— failure by overturning or by toppling (see 5).

(4) In addition to this Clause 7, ultimate limit states for gravity walls shall be verified according to Clause 5.

(5) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be considered for embedded retaining walls:

– failure by rotation or translation of the wall or parts thereof;
– failure by lack of vertical equilibrium.

(6) Ultimate limit states for embedded retaining walls shall be verified according to this Clause 7.

7.2.5.2 Serviceability Limit States

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all retaining structures:

– movements of the retaining structure that cause damage or affect the appearance or the use of the structure or nearby structures or services;
– unacceptable leakage through or beneath the structure;
– unacceptable change in the groundwater conditions induced by retaining structure itself.

(2) Potential serviceability limit states other than those given in (1) should be verified.

7.2.6 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply to retaining structures.

7.2.7 Ground investigation

7.2.7.1 General

(1) prEN 1997-2:2022, 5 shall apply to retaining structures.

(2) Investigations should include the installation of sufficient piezometers to measure groundwater variations within each relevant geotechnical unit considering seasonal, tide and fluvial changes.

7.2.7.2 Minimum extent of field investigation

(1) The depth and horizontal extent of the investigation shall be sufficient to determine the ground conditions within the zone of influence of the structure according to prEN 1997-1:2022, 4.2.1.2.

(2) The depth of field investigation shall comply with prEN 1997-3:2022, 5.2.7.2 for gravity retaining structures and with prEN 1997-3:2022, 6.2.7.2 for embedded retaining structures with particular attention paid to hydraulic conditions at the bottom of the wall.

(3) The field investigation shall determine ground conditions over the full height of the retaining wall including any overlying fills or low strength soils.

7.2.8 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to retaining structures.

7.3 Materials

7.3.1 Ground properties

(1) prEN 1997-2:2022, Clause 7 to Clause 12 shall apply to retaining structures.

7.3.2 Plain and reinforced concrete

(1) prEN 1997-1:2022, 5.5 shall apply to retaining structures.

7.3.3 Steel

(1) prEN 1997-1:2022, 5.6 shall apply to retaining structures.

7.3.4 Sprayed concrete

(1) Clause10 shall apply to retaining structures.

7.3.5 Timber

(1) prEN 1997-1:2022, 5.7 shall apply to retaining structures.

7.3.6 Masonry

(1) prEN 1997-1:2022, 5.8 shall apply to retaining structures.

7.3.7 Other structural materials

(1) Materials other than concrete, steel, timber or masonry may be used provided they comply with a material standard specified by the relevant authority or, where not specified, as agreed for a specific project by appropriate parties.

7.3.8 Improved ground properties

(1) In case ground improvement techniques are used, either to form the retaining structure itself, or to improve the adjacent ground, material properties shall comply with Clause 11.

7.4 Groundwater

7.4.1 General

(1) prEN 1997-1:2022, 6 shall apply to retaining structures.

(2) Potential obstruction of natural groundwater flow caused by l embedded retaining walls shall be considered.

(3) Retaining walls should be designed for an accidental design situation corresponding to a water table at the surface of the retained material unless the three following conditions are met:

– a persistent groundwater control system is installed (see Clause 12); or
– infiltration is prevented; or
– efficient piezometric control is ensured.

(4) Unfavourable potential effects of hydraulic gradients due to dewatering shall be considered when calculating groundwater pressures and resulting effective stresses (see 7.6.5).

7.4.2 Groundwater control systems

(1) Clause 12 shall apply to retaining structures.

(2) When the safety and the serviceability of the structure depends on the successful performance of a drainage system, a Maintenance Plan shall be specified.

7.5 Geotechnical analysis

7.5.1 General

(1) prEN 1997-1:2022, 7 shall apply to retaining structures.

(2) The limit states specified in 7.6 and 7.7 should be verified using one or more of the following calculation models:

– an analytical model (including limit equilibrium model and limit analysis);
– a semi-empirical model (including earth pressure envelopes);
– a numerical model (including beam-on-spring models or continuum model).

NOTE Further details of these models are given in Annex D.

(3) Prestressing forces exerted on the retaining structure by anchors or struts should be included in the calculation model.

7.5.2 Determination of earth pressures

(1) Determination of earth pressures shall take account of the expected failure mechanisms and deformations at the limit state under consideration.

NOTE 1 The magnitudes of earth pressures and directions of resultant forces are strongly influenced by horizontal and vertical movements of the retaining structure in relation to the ground block, which may vary with time, successive design situations, and limit states being considered.

NOTE 2 The term “earth pressure” includes ground pressure from rock.

(2) Total stress analysis may only be adopted if comparable experience exists.

(3) Calculations of earth pressure and the forces resulting from them shall consider, but are not limited to, the following:

– shear strength and weight density of the ground;
– amount and direction of the movement of the wall relative to the ground;
– surcharge on the ground surface;
– inclination of the ground surface;
– inclination of the wall to the vertical;
– wall roughness;
– rigidity of the structure and its supporting system relative to the stiffness of the ground;
– water levels and the seepage forces in the ground;
– strain and stiffness time-dependence for low-permeability fine soils;
– effect of compaction;
– horizontal and vertical equilibrium for the entire retaining structure;
– effect of initial stresses and stiffness of the ground;
– inclination of the ground strata and potential discontinuities;
– the swelling potential of the ground;
– anisotropy of the ground for mechanical and hydraulic properties;
– potential for strain ratcheting due to imposed cyclic actions.

(4) The shear stress mobilized at the interface between the ground and the structure shall be determined by the ground-structure interface coefficient (tan δ), where δ is the inclination of stresses applied to the interface.

(5) The value of the ground-structure interface coefficient (tan δ) shall comply with Formula (7.1):

$$\delta \leq k_{\delta}\varphi \quad (7.1)$$

where:

φ is the value of the ground’s angle of friction;

k_{δ} is a constant depending on the roughness of the ground structure interface and local disturbance during execution.

NOTE 1 The value of the interface coefficient depends on the relative displacement of the retaining structure in relation to the ground block that might, in specific circumstances, reduce the inclination of earth pressure.

NOTE 2 This reduction in inclination is automatically considered when using continuum numerical models. Explicitly introducing a value lower than the maximum is only relevant for analytical models that do not automatically take the relative displacement into account.

NOTE 3 The assessment of reduced values of the interface coefficient in the presence of structural forces is considered in 7.6.4.2 and more guidance is given in Annex D.

(6) In fine soils, it may be assumed that k_δ = a/c, where a is the adhesion to the wall and c the soil's cohesion.

(7) The value of k_δ shall not exceed 1.0.

(8) A value of k_δ = 1,0 may be assumed for concrete cast directly against soil and for stone infill or backfill used for crib walls and gabions.

(9) The value of k_δ should not exceed 2/3 for retaining structures formed with smooth surfaces.

NOTE This limit can also be applied conservatively to retaining structures with rough surfaces.

(10) A value of k_δ = 0 should be used for steel sheet piles walls immediately after installation into clay or peat.

(11) In the case of structures retaining rock masses, calculations of the earth pressures shall take account of the effects of discontinuities in the rock mass, with particular attention to their orientation, spacing, aperture, roughness and the mechanical characteristics of any joint filling material.

NOTE The mechanical resistance of the matrix itself can be a limiting parameter in specific materials, such as schist.

7.5.3 Limiting values of earth pressure

(1) Limiting values of earth pressures shall be determined considering the relative movement of the ground and the wall at failure and the corresponding shape of the failure surface.

(2) When using tabulated values of earth pressure coefficients or computer software based on limit equilibrium analysis, the consistency between limiting values of earth pressure assuming straight failure surfaces and interface parameters δ should be considered in order to avoid unsafe results (see 7.5.5).

(3) In cases where struts, anchors, or similar structural elements impose restraints on movement of the retaining structure, the possibility of more adverse earth pressures than limiting active and passive values should be considered.

7.5.4 Values of active earth pressure

(1) For ground in an active state, the component of the total earth pressure normal to the wall face (p_a) at a depth (z_a) below ground surface may be determined from Formula (7.2):

$$p_\mathrm{a} = p'_\mathrm{a} + u_\mathrm{a} \geq p_\mathrm{a,min} \tag{7.2}$$

where:

p'_a is the component at depth z of the effective active earth pressure normal to the wall face, defined in (7.3);

u_a is the groundwater pressure acting at depth z on the active side of the wall;

$p_\mathrm{a,min}$ is the minimum value of p_a.

(2) A minimum value of $p_{a,min} > 0$ should be used when very large cohesion values result in no effective pressure being applied over a significant height of the wall.

(3) The component of the effective active earth pressure normal to the wall face (p'_a) at a depth (z_a) below ground surface may be determined from Formula (7.3):

$$p'_{\mathrm{a}} = K_{\mathrm{a}\gamma}\left(\bar{\gamma}_{a,av} z_{\mathrm{a}} - u_{\mathrm{a}}\right) - K_{\mathrm{ac}} c' + K_{\mathrm{aq}} q_{\mathrm{a}} \tag{7.3}$$

where, in addition to the symbols defined for Formula (7.2):

$\gamma_{a,av}$	is the average weight density of the ground above depth z_a;
c'	is the soil's effective cohesion;
q_a	is the vertical surcharge applied at the ground surface; and
$K_{a\gamma}$, K_{ac}, and K_{aq}	are active earth pressure coefficients.

NOTE Values of $K_{a\gamma}$, K_{ac}, and K_{aq} are given in Annex D.

(4) When using a total stress calculation of undrained behaviour (see 7.5.2), Formula (7.4) may be used instead of (7.2) and (7.3):

$$p_{\mathrm{a}} = (\bar{\gamma}_{\mathrm{a}} z_{\mathrm{a}}) - K_{\mathrm{ac,u}} c_{\mathrm{u}} + q_{\mathrm{a}} \geq p_{\mathrm{a,min}} \tag{7.4}$$

where, in addition to the symbols defined for Formula (7.2):

c_u	is the soil's undrained shear strength;
$K_{ac,u}$	is an active earth pressure coefficient for undrained conditions.

NOTE Values of $K_{ac,u}$ are given in Annex D.

(5) The value of $p_{a,min}$ shall be ≥ 0.

NOTE The value of $p_{a,min}$ is 10 % of the total vertical stress unless the National Annex gives different values.

(6) A value of $p_{a,min} > u_a$ should be used when very large cohesion values result in no pressure being applied over a significant height of the wall.

7.5.5 Values of passive earth pressure

(1) For ground in a passive state, the component of the total earth pressure normal to the wall face (p_p) at a depth (z) below formation level may be determined from Formula (7.5):

$$p_{\mathrm{p}} = p'_{\mathrm{p}} + u_{\mathrm{p}} \tag{7.5}$$

where, in addition to the symbols defined for Formula (7.2):

p'_p	is the component at depth z of the effective passive earth pressure normal to the wall face, defined in (7.6);
u_p	is the groundwater pressure acting at depth z on the passive side of the wall.

(2) The component of the effective passive earth pressure normal to the wall face (p'_{p}) at a depth (z_{p}) below formation level may be determined from Formula (7.6):

$$p'_{\mathrm{p}} = K_{\mathrm{p\gamma}}\left(\gamma_{p,av} z_{\mathrm{p}} - u_{\mathrm{p}}\right) + K_{\mathrm{pc}} c' + K_{\mathrm{pq}} q_{\mathrm{p}} \tag{7.6}$$

where, in addition to the symbols defined for Formula (7.5):

$\gamma_{\mathrm{p,av}}$	is the average weight density of the ground above depth z_{p};
q_{p}	is any permanent vertical load applied at formation level; and
$K_{\mathrm{p\gamma}}$, K_{pc}, and K_{pq}	are passive earth pressure coefficients.

NOTE Values of $K_{\mathrm{p\gamma}}$, K_{pc}, and K_{pq} are given in Annex D.

(3) Coefficients of passive earth pressure should be cautiously assessed for high values of the friction angle (> 40°).

(4) When using a total stress analysis for calculation of undrained behaviour, Formula (7.7) may be used instead of Formula (7.5):

$$p_{\mathrm{p}} = \left(\overline{\gamma_{p,av}} z_{\mathrm{p}}\right) + K_{\mathrm{pc,u}} c_{\mathrm{u}} + q_{\mathrm{p}} \tag{7.7}$$

where, in addition to the symbols defined for Formula (7.5):

$K_{\mathrm{pc,u}}$ is a passive earth pressure coefficient for undrained conditions.

NOTE Values of $K_{\mathrm{pc,u}}$ are given in Annex D.

(5) If limiting values of passive earth pressure are determined by assuming planar failure surfaces, the ground-structure interface coefficient in Formula (7.1) should be reduced to tan δ = 0.

(6) Only permanent loads shall be considered on the passive side of the retaining structure.

7.5.6 At-rest values of earth pressure

(1) The earth pressure coefficient at rest K_0 should be determined according prEN 1997-2:2022, 7.1.7 taking into account in addition the type of retaining structures and the conditions of installation.

NOTE Some examples of conditions that affect the earth pressure coefficient at rest include the ratio of overconsolidation in clay, a cylindrical wall layout on plan, and the wall's installation method.

(2) For ground in an at-rest state, the total earth pressure (p_0) at a depth (z_0) below ground surface may be determined from Formula (7.8):

$$p_0 = p'_0 + u = K_0\left(\gamma_{o,av} z_o - u + q\right) + u \tag{7.8}$$

where:

p'_0	is the effective at-rest earth pressure at depth z;
u	is the groundwater pressure;
K_0	is the at-rest earth pressure coefficient.

$\gamma_{o,av}$ is the average weight density of the ground above depth z_0; and

q is the vertical load applied at the surface of the ground.

NOTE Calculation models to determine K_0 are given in Annex D.

7.5.7 Intermediate values of earth pressure

(1) Intermediate values of earth pressure, between active and passive limits, shall be determined considering the amount of wall movement and its direction relative to the ground.

(2) The intermediate values of earth pressures acting on the wall may be determined using empirical rules, beam on springs models, or continuum numerical models.

NOTE Guidance on suitable calculation models and determination of ground stiffness, which plays an important part in soil structure interaction, is given in Annex D.

7.5.8 Compaction pressures

(1) The determination of earth pressures acting behind the wall shall consider any additional pressures generated by compacting backfill, in relation with the procedures adopted for its compaction.

NOTE Guidance for determining these additional pressures is given in Annex D.

(2) For integral bridges, enhanced values of earth pressure shall be determined considering the total movement of the abutment from its maximum expansion position to its maximum contraction position, and the direction of movement being considered in conjunction with the position of the abutment.

NOTE For a given position of the abutment, there will be a maximum and minimum potential pressure depending on whether the abutment is moving in or out of the backfill.

7.5.9 Groundwater pressures

(1) prEN 1997-1:2022, 6 shall apply to retaining structures.

7.6 Ultimate limit states

7.6.1 General

(1) Effects of actions derived from ultimate limit state verifications shall be considered when checking the structural resistance of the retaining structure and associated supports, as well as the pull-out resistance of anchors.

7.6.1.1 Verification by the Observational Method

(1) For all retaining structures, when verification of limit states by the Observational Method is performed, prEN 1997-1:2022, 4.7 shall apply.

7.6.2 Overall stability

(1) The overall stability of a retaining structure shall be verified in accordance with Clause 4.

NOTE Figure 7.1 gives examples of limit modes for overall stability of retaining structures.

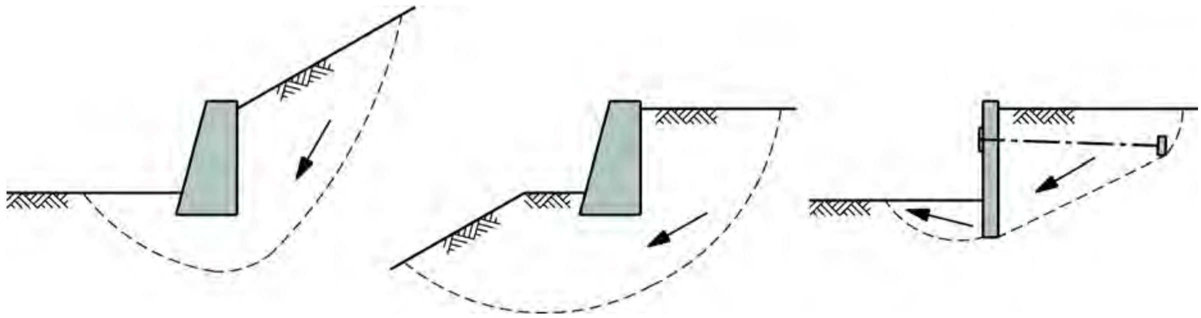


Figure 7.1 — Examples of limit modes for overall stability of retaining structures

(2) If measures are necessary to ensure the overall stability of the site and the retaining structure plays a part in those measures, then the stability of failure surfaces that intersect the retaining structure shall be verified.

(3) If a continuum numerical model is used for overall stability calculations, it should also be used to verify the ultimate limit states given in 7.6.4.1 (rotational resistance), 7.6.5 (stability of excavations), and 7.6.7 (structural failure).

NOTE This does not exclude that other calculation models are additionally used when checking local failure mechanisms.

(4) When a numerical model is used for overall stability calculations with elastic properties for structural elements, forces into these structural elements shall be checked according to prEN 1992 (all parts), prEN 1993 (all parts), prEN 1995 (all parts) or prEN 1996 (all parts) depending on the nature of structural elements (concrete, steel, timber, masonry).

(5) When a numerical model is used for overall stability calculations with elasto-plastic properties for structural elements shall be verified according to prEN 1997-1:2022, 8.2 with the ultimate resistance of structural elements defined according to prEN 1992 (all parts), prEN 1993(all parts), prEN 1995(all parts) or prEN 1996 (all parts) depending on the nature of structural elements (concrete, steel, timber, masonry).

(6) If the rotational resistance of a retaining structure is verified using the resistance factor approach, with partial factors only applied to passive earth pressure (see 7.6.8), one of the following approaches should be used for overall stability calculations:

– the effects of actions into the retaining wall are checked using a continuum numerical model;
– failure surfaces intercepting the retaining structure are checked using a limit equilibrium method;
– the overall stability is checked by considering an additional model factor γ_{Rd}.

NOTE Unless the National Annex gives different values, the value of γ_{Rd} is 1.2 for persistent design situations and sensitive structures, 1.05 for transient design situations, and 1.0 for deep failure mechanisms that have no possibility of interfering with the retaining structure.

7.6.3 Gravity walls

(1) Overall stability of a gravity retaining structure shall be verified according to Clause 4 and 7.6.2.

(2) The resistance of a gravity retaining structure to bearing, sliding, overturning resistance and toppling shall be verified according to Clause 5.

7.6.4 Embedded walls

7.6.4.1 Rotational resistance

(1) Resistance to loss of rotational equilibrium may be verified using analytical calculation models or continuum numerical models.

NOTE 1 Figure 7.2 gives examples of mechanisms involving failure of embedded walls.

NOTE 2 Further information about calculation models is given in Annex D.

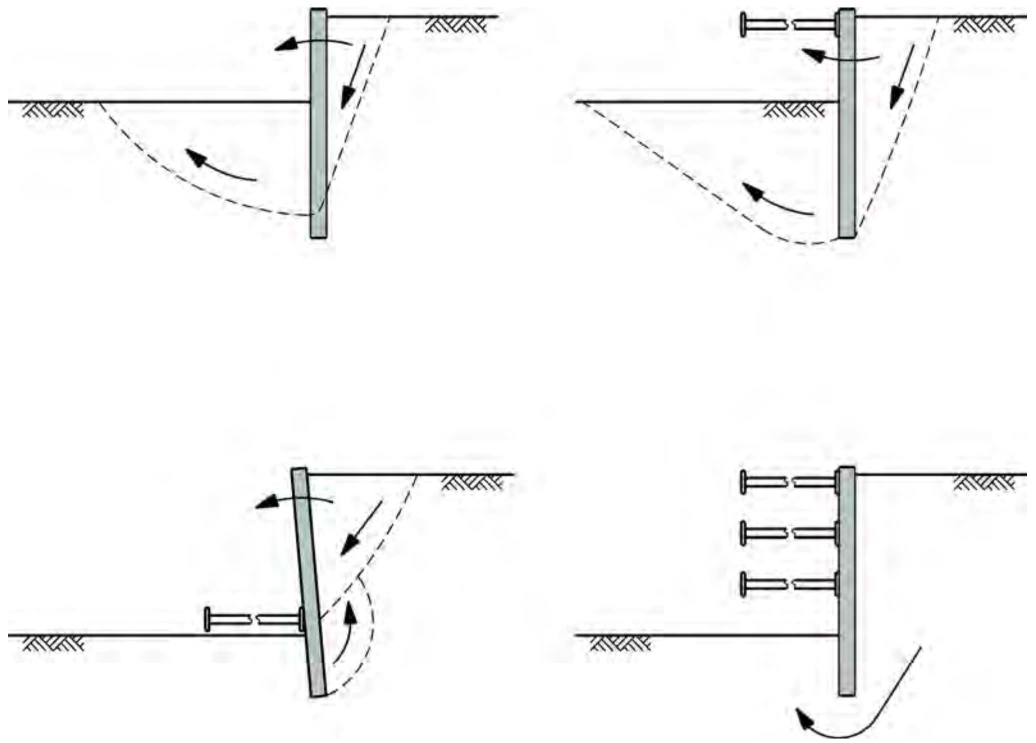


Figure 7.2 — Examples of failure mechanisms for embedded walls

7.6.4.2 Bearing resistance

(1) The bearing resistance of an embedded wall that is subject to significant imposed vertical forces, shall be verified according to either Clause 5 or Clause 6, depending on its embedded length.

NOTE Significant vertical forces can be imposed on an embedded wall by inclined anchors.

(2) It shall be verified that the shaft friction mobilized to ensure the vertical equilibrium is compatible with the horizontal equilibrium in terms of stress inclination.

NOTE 1 Shaft friction acting downwards on the active side of the wall or upwards on the passive side considerably change the coefficients of earth pressure in an adverse way.

NOTE 2 Guidance is provided in 7.5.1(6) and Annex D.

7.6.5 Stability of excavations

(1) Resistance to failure by heave of the bottom of excavations due to unloading of the ground shall be verified.

NOTE Guidance about suitable models is provided in Annex D.

(2) Resistance to basal heave during excavation in fine soils should be verified assuming undrained ground conditions.

(3) Resistance to basal heave should be verified assuming drained conditions when undrained conditions are likely to be less critical, particularly in layered soils.

(4) Resistance to basal heave in coarse soils should be verified considering hydraulic gradients in the soil.

(5) In the presence of hydraulic gradients, it shall be verified that limit states due uplift (see prEN 1997-1:2022, 8.2.3.2), hydraulic heave (see prEN 1997-1:2022, 8.2.4.2), and internal erosion or piping (see prEN 1997-1:2022, 8.2.4.3) or bottom failure mechanisms, i.e. basal heave, are not exceeded.

NOTE See Annex D for basal heave.

(6) Measures should be taken to avoid the adverse effects of upward hydraulic gradients.

NOTE Examples of preventive measures include: deep relief wells to protect the passive zone close to embedded walls; increased embedment; embedment down to impervious layers and grouting,.

(7) If upward hydraulic gradients cannot be avoided in the passive zone close to the retaining structure, passive earth resistance shall be reduced accordingly and potential failure due to soil erodibility shall be checked.

7.6.6 Supporting elements

(1) It shall be verified that the supporting element can resist a design force effect given by Formula (7.9):

$$E_d = max(\gamma_{Sd} F_{d,ULS}; \gamma_{F;} F_{d,SLS}) \tag{7.9}$$

where:

$F_{d,ULS}$ is the design value of the action that the supporting element shall provide to prevent an ultimate limit state;

$F_{d,SLS}$ is the design value of the action that the supporting element shall provide to prevent a serviceability limit state;

γ_{Sd} is a model factor to address the concentration of lad in the supporting element and depending on the stiffness of the retained wall and the arching effects;

γ_F is used to convert a SLS value to an ULS value (using DC4).

NOTE 1 The value of the model factor, γ_{Sd}, is 1.0 unless the National Annex gives another value.

NOTE 2 The value of the partial factor, γ_F is 1.35 according to VC4 unless the National Annex gives another value.

7.6.7 Structural failure

(1) The structural resistance of retaining structures and their component members shall be verified in accordance with:

- prEN 1992 (all parts) for reinforced or plain concrete retaining walls;
- FprEN 1993 (all parts) for steel retaining walls;
- EN 1994 (all parts) for composite steel and concrete retaining walls;

- EN 1995 (all parts) for timber members in retaining walls;
- prEN 1996 (all parts) for masonry retaining walls.

(2) Structural resistance shall be verified considering all geotechnical failure mechanisms that interfere with the retaining structure.

7.6.8 Partial factors

(1) Partial factors for the verification of retaining structures at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using either the Material Factor Approach or the Resistance Factor Approach

NOTE 1 The National Annex can specify which Factor Approach to use

NOTE 2 Values of the partial factors are given in Table 7.1 (NDP) for persistent and transient design situations unless the National Annex gives different values.

NOTE 3 Additional guidelines for use of partial factors for numerical models, is given in prEN 1997-1:2022, 8.2.

(2) If the resistance factor approach is used, the partial factor γ_{Re} should be applied to the resultant passive earth resistance.

NOTE When using the resistance factor approach, the partial factors γ_R and γ_E can be combined into a single factor applied to passive soil resistance.

(3) When using the resistance factor approach, explicit verification of rotational resistance may be omitted if the upper part of the retaining structure is supported by anchors, struts, or slabs and the ratio between the passive earth resistance and the mobilized earth pressure in front of the wall is greater or equal to $\gamma_{Re}\,\gamma_E$.

Table 7.1 — (NDP) Partial factors for the verification of ground resistance against retaining structures for fundamental (persistent and transient) design situations and accidental design situations

Verification of	Partial factor on	Symbol	Material factor approach (MFA) – both combinations (a) and (b)		Resistance factor approach (RFA)
			(a)	(b)	
Overall stability	See Clause 4				
Bearing resistance of gravity walls	See Clause 5				
Bearing resistance of embedded walls	See Clause 6				
Rotational resistance	Actions and effects-of-actions	γ_F and γ_E	VC4[a]	VC3[a]	VC4[a] (EFA)[d]
	Ground properties	γ_M	M1[b]	M2[b]	Not factored
	Passive earth resistance	γ_{Re} γ_E	Not factored		1.4 γ_E (1.12 γ_E)[c]
Basal heave	See Annex D and Clause 5				

a Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in prEN 1990.

B Values of the partial factors for Sets M1 and M2 are given in prEN 1997-1:2022, Table 4.7.

c Values in brackets are for accidental situations.

D See prEN 1997-1:2022, 8.2.

7.7 Serviceability limit states

7.7.1 General

(1) Where relevant, the assessment of design values of earth pressures should consider initial stresses in and the stiffness and strength of the ground and the stiffness of the structural elements.

7.7.2 Displacements

(1) Limiting values of ground movement around retaining structures shall comply with prEN 1997-1:2022, 4.2.5 and 9.3, considering the tolerance to displacements of supported structures and utilities within the zone of influence.

(2) Ground movement around retaining structures, and their effects on supported structures and services, shall always be checked against comparable experience.

(3) Determination of ground movement around retaining structures shall consider the sequence of work.

(4) Vibrations caused by traffic loads or construction machinery close to the retaining wall should be considered when estimating ground movements around retaining structures.

NOTE Guidance on traffic loads is given in prEN 1991-2.

(5) When linear ground behaviour is assumed, the stiffness adopted for the ground and structural materials should be defined according to the potential range of deformation and the potential stress paths.

NOTE When linear behaviour is assumed differential movements in the zone of influence of the retaining structure are usually under-estimated, as well as the effects of ground movements of adjacent structures.

7.8 Implementation of design

7.8.1 General

(1) prEN 1997-1:2022, 10 shall apply to retaining structures.

(2) The execution, and control of concrete gravity walls should comply with EN 13670.

(3) The execution, and control of steel sheet pile walls should comply with EN 12063.

(4) The execution, and control of diaphragm walls should comply with EN 1538.

(5) The execution, and control of pile walls should comply with EN 1536, EN 14199, or EN 12699 depending on type of piles.

(6) The execution, and control of steel combined walls and high modulus walls should comply with EN 12063.

(7) The execution, and control of deep mixing and jet grouting walls should comply with EN 14679 and EN 12716 respectively.

7.8.2 Inspection

7.8.2.1 General

(1) In addition to prEN 1997-1:2022, 10.3, the Inspection Plan should include, but is not limited to:

– verification of ground and groundwater conditions, and of the location and general layout of the retaining structure and any adjacent settlement sensitive structure (above and below ground);
– verification of the sequence of works, and control of ground excavation levels, as well as temporarily applied loads behind the retaining structure;
– for gravity retaining structures, verification of the quality of foundation ground, including as necessary placement of a concrete screed or a drainage layer properly compacted.

7.8.2.2 Water flow and groundwater pressures

(1) In addition to prEN 1997-1:2022, 10.3, the Inspection Plan should include, but is not limited to, measures to check:

- adequacy of systems to ensure control of groundwater pressures in all aquifers where excess pressure could affect stability of slopes or base of excavation, including artesian pressures in an aquifer beneath the excavation;
- disposal of water from dewatering systems;
- depression of groundwater table throughout entire excavation to prevent boiling or quick conditions, piping and disturbance of formation by construction equipment;
- diversion and removal or rainfall or other surface water;
- efficient and effective operation of dewatering systems throughout the entire construction period, considering encrusting of well screens, silting of wells or sumps;
- wear in pumps;
- clogging of pumps
- control of dewatering to avoid disturbance of adjoining structures or areas;
- observations of piezometric levels;
- effectiveness, operation and maintenance of water recharge systems, if installed; and
- effectiveness of sub-horizontal borehole drains.

(2) In addition to (1), the Inspection Plan should include, but is not limited to, measures to check:

- groundwater flow and pressure regime;
- effects of dewatering operations on groundwater table;
- effectiveness of measures taken to control seepage inflow;
- internal erosion processes and piping;
- chemical composition of groundwater; and
- corrosion potential.

7.8.3 Monitoring

(1) In addition to prEN 1997-1:2022, 10.4, the Monitoring Plan should include, but is not limited to:

- settlements at established time intervals of adjoining structures or areas, more especially in the case of compressible or weak quality soil layers;
- evolution of existing cracks in adjacent structures;
- piezometric or groundwater levels under buildings or behind the structure, or in adjoining areas, especially if permanent dewatering systems are installed;
- deflection or displacement of retaining structures;
- behaviour of temporary or permanent support systems, such as anchors or struts; and
- the required degree of water tightness.

7.8.4 Maintenance

(1) prEN 1997-1:2022, 10.5 shall apply to retaining structures.

(2) In addition to prEN 1997-1:2022, 10.5, for permanent retaining structures, the Maintenance Plan should include specifications relative to maintenance of sensitive devices, including anchors, drains and pumping wells.

7.9 Testing

(1) prEN 1997:2022, 11 shall apply to retaining structures.

(2) The efficiency of any dewatering system should be tested before the beginning of excavation, in accordance with EN ISO 22282-4.

7.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to retaining structures.

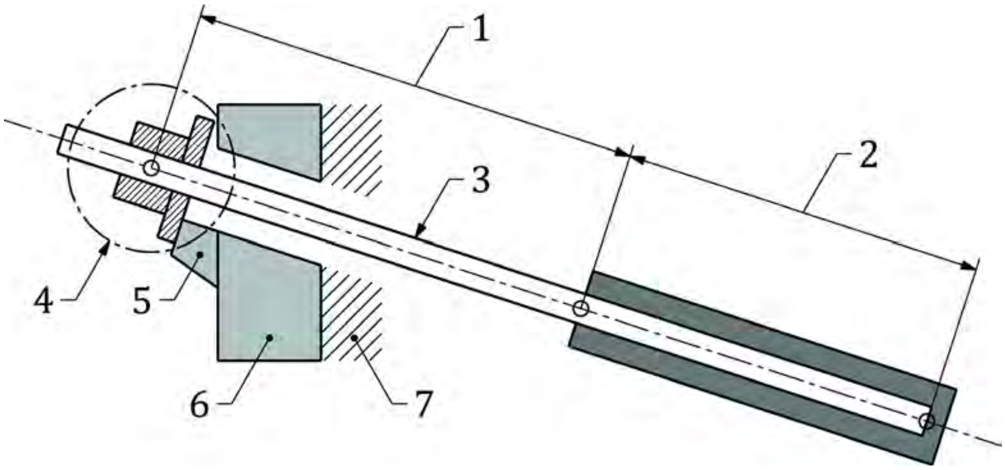
8 Anchors

8.1 Scope and field of application

(1) This Clause shall apply to temporary and permanent anchors that transmit a tensile force from the anchor head through a free anchor length over a resisting element to a load resisting formation of soil or rock.

NOTE 1 This includes anchors within the scope of EN 1537 and mechanical anchors with a free anchor length (such as screw, harpoon, and expander anchors).

NOTE 2 Figure 8.1 shows an anchor within the scope of this clause.



Key

1 free anchor length

2 fixed anchored length(e.g. the grout body)

3 tendon

4 anchor head

5 load transfer block

6 anchored structure

7 soil/rock

Figure 8.1 — Grouted anchor within the scope of Clause 8

(2) Tension elements without a free length shall be designed according to Clause 6 or Clause 10.

NOTE 1 For tension elements without a free length such as piles and micropiles see Clause 6

NOTE 2 For tension elements without a free length such as soil nails and rock bolts see Clause 10.

(3) Anchor walls providing fixity for dead-man anchors shall be designed according to Clause 7.

8.2 Basis of design

8.2.1 Design situations

(1) prEN 1997-1:2022, 4.2.2 shall apply to anchors.

8.2.2 Geometrical properties

8.2.2.1 General

(1) prEN 1997-1:2022, 4.3.3 shall apply to anchors.

(2) The required free anchor length shall be determined in the design of the anchored structure.

(2) The anchor head shall be designed to tolerate angular deviations complying with EN 1537.

(3) The anchor head shall be designed to allow the tendon to be stressed, proof-loaded, and locked-off and (if required) released, de-stressed, and re-stressed.

(4) The anchor head shall be designed to accommodate deformations and load variation that can occur during the design service life of the structure.

(5) Measures shall be taken to avoid adverse interactions between anchors that are located close to each other.

NOTE Details are given in Annex E.

(6) The resisting ground should be sufficiently distant from the anchored structure to avoid any adverse interaction between the two.

(7) The orientation of the anchor should be chosen to enable self-stressing under deformation.

(8) If self-stressing under deformation is not possible, the adverse effects of potential failure mechanisms shall be considered.

(9) The orientation of the anchor should be chosen to optimize the transfer of load into the resisting ground.

8.2.2.2 Zone of influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to anchors.

8.2.3 Actions and environmental influences

8.2.3.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to anchors.

8.2.3.2 Permanent and variable actions

(1) Design values of the anchor force and lock off load shall be obtained from the verification of limit states for the anchored structure.

(2) Anchor forces required to support slopes, cuttings, and embankments shall comply with Clause 4.

(3) Anchor forces required to support retaining structures shall comply with Clause 7.

(4) For uplift design values of the anchor forces shall exceed the resistance required by prEN 1997-1:2022, 8.1.3.2.

(5) The lock-off load shall not give rise to a limit state in the ground, in the anchored or in the supported structures.

(6) It shall be verified that the lock-off load is sufficient to ensure that the anchor resistance can be restrictions without exceeding the serviceability limit state of both the anchored and adjacent structures.

8.2.3.3 Cyclic and Dynamic actions

(1) prEN 1997-1:2022 4.3.1.3 shall apply to anchors.

8.2.3.4 Environmental influences

(1) prEN 1997-1:2022 4.3.1.5 shall apply to anchors.

(2) The potential adverse effect of chemical components of ground or groundwater according to EN 1537 shall be taken into account for design for durability.

8.2.4 Limit states

8.2.4.1 Ultimate Limit States

(1) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be verified for all anchors:

- structural failure of the tendon or anchor head;
- rupture at the interface between the tendon and the grout body;
- rupture at the interface between the grout body or the resisting element and the resisting ground;
- loss of anchor force by displacement of the resisting element due to creep, deformations or fall-out of ground behind;
- limit states in anchored or adjacent structures, including those consequence of testing and pre-stressing;
- excessive deformation of the anchored structure.

(2) Potential ultimate limit states other than those given in (1) should be verified.

(3) For a group of anchors, verification shall be based on the most critical failure surface.

NOTE Depending on spacing and the profile of ground strength, this can involve displacement of part of or the whole anchored ground body, often combined with pull-out of the distant ends of the anchors.

8.2.4.2 Serviceability Limit States

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all anchors:

- deformation of the anchored structure;
- increase of anchor load during the design service life;

– loss of anchor force by displacement of the resisting element due to creep, deformations or fall-out of ground behind.

(2) Potential serviceability limit states other than those given in (1) should be verified.

8.2.5 Robustness

(1) prEN 1997-1:2022 Clause 4.1.4 shall apply to anchors.

8.2.6 Ground investigation

8.2.6.1 General

(1) prEN 1997-2:2022, 5 shall apply to anchors.

(2) The zone of ground into which tensile forces are transferred should be included in ground investigations.

(3) The ground investigation should determine the potential influence of difficulties caused by, but not limited to:

- potential obstructions to drilling;
- the process of borehole drilling (drillability);
- abrasivity;
- anchor borehole instability;
- flow of groundwater in or out of the borehole;
- geometrical properties of discontinuities and weakness zones in ground;
- borehole axis deviations; and
- loss of grout from the borehole.

8.2.6.2 Minimum extent of field investigation

(1) The depth and horizontal extent of the field investigation shall be sufficient to determine the ground conditions within the zone of influence of the structure according to prEN 1997-1:2022, 4.2.1.1.

(2) The depth and horizontal extent of the field investigation should be sufficient to ensure that:

- the Ground Model within the zone of influence of the anchors is confirmed;
- no underlying stratum will affect the anchor design;
- groundwater conditions are well defined; and
- the geometry of discontinuities and of the weak zones in the zone of influence of the anchors are well defined.

8.2.7 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to anchors.

(2) Anchors shall be classified in GC2 or GC3.

8.3 Materials

8.3.1 Ground Properties

(1) prEN 1997-2:2022, Clause 7 to 12 shall apply to anchors.

8.3.2 Steel

(1) prEN 1997-1:2022, 5.6 shall apply to anchors.

8.3.3 Grout

(1) prEN 1997-1:2022, 5.4 shall apply to anchors.

8.3.4 Other materials

(1) If a material other than steel is used for the anchor tendon, it shall be checked independently as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

8.4 Groundwater

(1) prEN 1997-1:2022, 6 shall apply to anchors.

8.5 Geotechnical analysis

(1) prEN 1997-1:2022, 7 shall apply to anchors.

(2) In addition to prEN 1997-1:2022, 7, the geotechnical analysis shall address all limit state verifications listed in 8.2.4.

8.6 Ultimate limit states

8.6.1 General

(1) The design value of the ultimate limit state resistance of an anchor shall satisfy Formula (8.1).

$$E_{\mathrm{d,ULS}} \leq min\left(R_{\mathrm{ad,ULS}}; R_{\mathrm{td}}\right) \tag{8.1}$$

where

$E_{\mathrm{d,ULS}}$ is the design value of the effects of actions at the ultimate limit state;

$R_{\mathrm{ad,ULS}}$ is the design value of an anchor's geotechnical resistance at the ultimate limit state;

R_{td} is the design value of the tensile resistance of the structural element.

(2) $E_{\mathrm{d,ULS}}$ shall be evaluated according to 4.5.4 and 7.6.6 and prEN 1997-1:2022, 8.1.3.

(3) $E_{\mathrm{d,ULS}}$ shall include the effect of anchor lock-off load.

8.6.2 Geotechnical resistance

(1) Anchors shall only be used if their geotechnical design and construction have been verified by:

- investigation or suitability tests; or
- comparable experience.

NOTE 1 Anchors are verified by investigation and suitability tests unless the National Annex states otherwise.

NOTE 2 Comparable experience is defined in prEN 1997-1:2022, 3.1.2.3

(2) Acceptance tests shall be carried out on all anchors.

(3) Investigation, suitability and acceptance tests on grouted anchors should comply with EN ISO 22477-5.

(4) In addition to (2), the measured value of the geotechnical resistance of a grouted anchor at the ultimate limit state shall be determined for each distinct geotechnical unit from a minimum of:

- three investigation or suitability tests, when using Test Method 1 specified in EN ISO 22477-5;
- two investigation tests and three suitability tests, when using Test Method 3 specified in EN ISO 22477-5.

(5) For non-grouted anchor types, the minimum number of tests shall comply with (4) unless otherwise specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(6) The measured value of the geotechnical resistance of a grouted anchor at the ultimate limit state ($R_{\mathrm{am,ULS}}$) shall be obtained from the results of an anchor test using Formula (8.2):

$$R_{\mathrm{am,ULS}} = \min(R_{\mathrm{am}}(\alpha_{\mathrm{ULS}}); P_{\mathrm{P}}) \tag{8.2}$$

where:

$R_{\mathrm{am}}(\alpha_{\mathrm{ULS}})$ is the measured value of the anchor's geotechnical resistance complying with the ultimate limit state criterion, α_{ULS};

P_{P} is the proof load.

(7) For grouted anchors, the ultimate limit state criterion α_{ULS} in Formula (8.2) shall be the creep rate:

- α_1 for Test Method 1;
- α_3 for Test Method 3.

NOTE 1 The values of α_1 and α_3 are given in Table 8.3 (NDP), unless the National Annex gives different values.

NOTE 2 The load relating to the physical pull-out resistance can be higher than the value of the load corresponding to the creep rates given above.

(8) The measured value of the geotechnical resistance of a non-grouted anchor at the ultimate limit state ($R_{\mathrm{am,ULS}}$) shall be obtained from the results of anchor test using Formula (8.3):

$$R_{\mathrm{am,ULS}} = \min\left(R_{am}\left(C_{\mathrm{ad,ULS}}\right); P_{\mathrm{P}}\right) \tag{8.3}$$

where:

$R_{\mathrm{am}}(C_{\mathrm{ad,ULS}})$ is the measured value of the anchor's geotechnical resistance complying with the ultimate limit state criterion, $C_{\mathrm{ad,ULS}}$;

P_{P} is the proof load.

(9) For non-grouted anchors, $C_{\mathrm{ad,ULS}}$ should be specified by the relevant authority or, where not specified, be agreed for a specific project by the relevant parties.

NOTE For non-grouted anchors, $C_{\mathrm{ad,ULS}}$ can be given in the National Annex.

(10) If the ultimate limit state criterion is not reached during a test, P_{p} shall be taken as $R_{\mathrm{am,ULS}}$.

(11) The characteristic value of an anchor's geotechnical resistance at the ultimate limit state $R_{ak,ULS}$ shall be determined from Formula (8.4):

$$R_{ak,ULS} = \frac{(R_{am,ULS})_{min}}{\xi_{ULS}} \tag{8.4}$$

where:

$(R_{am,ULS})_{min}$ is the minimum value of $R_{am,ULS}$ measured in a number of tests;

ξ_{ULS} is a correlation factor taking into account the number of tests.

NOTE The value of ξ_{ULS} is 1.0 unless the National Annex gives a different value.

(12) The design value of an anchor's geotechnical ultimate limit state resistance $R_{ad,ULS}$ shall be determined from Formula (8.5):

$$R_{ad,ULS} = \frac{R_{ak,ULS}}{\gamma_{Ra,ULS}} \tag{8.5}$$

where:

$R_{ak,ULS}$ is the characteristic value of the anchor's geotechnical resistance at the ultimate limit state;

$\gamma_{Ra,ULS}$ is a partial factor on the anchor's geotechnical resistance at the ultimate limit state, given in 8.6.4.

8.6.3 Structural resistance

(1) The design value of the ultimate limit state resistance of the structural elements of an anchor shall comply with EN 1993-5 and with Formula (8.6):

$$E_{d,ULS} \leq R_{td} \tag{8.6}$$

where:

$E_{d,ULS}$ is the design value of the effects of actions at ultimate limit state (see formula 8.2);

R_{td} is the design value of the tensile resistance of the structural element.

(2) The structural design of steel tendons under a proof load should comply with EN ISO 22477-5.

8.6.4 Partial factors

(1) Partial factors for the verification of anchors at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach in combination with either Text Method 1 or Test Method 2.

NOTE 1 The National Annex can specify which Test Method to use.

NOTE 2 Values of $\gamma_{Ra,ULS}$ are given in Table 8.1 (NDP) for persistent, transient, and accidental design situation unless the National Annex gives different values.

Table 8.1 — (NDP) Partial factors for the verification of geotechnical resistance of anchors for fundamental (persistent and transient) and addicental design situations at the ultimate limit state

Verification of	Partial factor on	Symbol	Resistance factor approach (RFA)	
			Test Method 1	Test Method 3
Geotechnical resistance of an anchor	Geotechnical resistance at the ultimate limit state	$\gamma_{a,\text{ULS}}$	1,1[a,b] (1,05)[c]	1,1 [a] (1,05)[c]

[a] See Formula (8.5)

[b] See Formulae (8.13) and (8.15)

[c] Values in brackets are for accidental design situations

8.7 Serviceability limit states

8.7.1 General

(1) If Test Method 3 is used to determine the ultimate limit state resistance of a grouted anchor, then its geotechnical resistance at the serviceability limit state should be verified in Suitability and Acceptance Tests against the critical creep load P_c determined in a previous Investigation Test.

NOTE In Test Method 1, the serviceability limit state of a grouted anchor is implicitly verified by verification of the ultimate limit state.

(2) If Test Method 3 is used, the anchor's design resistance ($R_{\text{ad,SLS}}$) shall comply with Formula (8.7):

$$E_{d,\text{SLS}} \leq R_{\text{ad,SLS}} \tag{8.7}$$

where:

$E_{d,\text{SLS}}$ is the design value of the maximum anchor force, including the lock-off load, and sufficient to prevent the serviceability limit state in the anchored structure;

$R_{\text{ad,SLS}}$ is the design value of the anchor's geotechnical resistance at the serviceability limit state.

8.7.2 Geotechnical resistance

(1) If Test Method 3 is used, the measured serviceability limit state resistance $R_{\text{am,SLS}}$ of an anchor shall be determined from a minimum of two investigation tests in each geotechnical unit.

(2) The measured geotechnical resistance of a grouted anchor at the serviceability limit state ($R_{\text{am,SLS}}$) shall be determined from Formula (8.8):

$$R_{\text{am,SLS}} = \min(R_{am}(\alpha_{\text{SLS}}); P_{\text{C}}; P_{\text{P}}) \tag{8.8}$$

where:

$R_{am}(\alpha_{\text{SLS}})$ is the measured value of the anchor's geotechnical resistance complying with α_{SLS};

α_{SLS} Is the serviceability limit state criterion for grouted anchors, given in 8.9.2;

P_C is the critical creep load P_c evaluated in Test Method 3 of EN ISO 22477-5;

P_P is the proof load.

(3) The measured geotechnical resistance of a non-grouted anchor at the serviceability limit state ($R_{am,SLS}$) shall be determined from Formula (8.9):

$$R_{am,SLS} = \min\left(R_{am}(C_{ad,SLS}); P_C; P_P\right) \tag{8.9}$$

where:

$R_{am}(C_{ad,SLS})$ is the measured value of the anchor's geotechnical resistance at complying with $C_{ad,SLS}$;

$C_{ad,SLS}$ is the serviceability limit state criterion for non-grouted anchors;

P_C is the critical creep load P_c evaluated in Test Method 3 of EN ISO 22477-5;

P_P is the proof load.

(4) For non-grouted anchors, $C_{ad,SLS}$ should be specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE $C_{ad,SLS}$ can be given in the National Annex.

(5) The characteristic value of the geotechnical resistance of an anchor at the serviceability limit state ($R_{ak,SLS}$) shall be determined from Formula (8.10):

$$R_{ak,SLS} = \left(R_{am,SLS}\right)_{min} \tag{8.10}$$

where:

$(R_{am,SLS})_{min}$ is the minimum value of $R_{am,SLS}$ measured in a number of tests.

(6) The design value of the geotechnical resistance of an anchor at the serviceability limit state ($R_{ad,SLS}$) shall be determined from Formula (8.11):

$$R_{ad,SLS} = \frac{R_{ak,SLS}}{\gamma_{Ra,SLS}} \tag{8.11}$$

where:

$R_{ak,SLS}$ is the characteristic value of the anchor's geotechnical resistance at the serviceability limit state;

$\gamma_{Ra,SLS}$ is a partial factor on the anchor's geotechnical resistance at the serviceability limit state, given in 8.9.

8.7.3 Partial factors

(1) Partial factors for the verification of anchors at the serviceability limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach in combination with either Text Method 1 or Test Method 3

NOTE Value of partial factors is given in Table 8.2 (NDP) unless the National Annex give different values.

Table 8.2 — (NDP) Partial factors for the verification of geotechnical resistance of anchors at the serviceability limit state

Verification of	Partial factor on	Symbol	Resistance factor approach (RFA)	
			Test Method 1	Test Method 3
Geotechnical resistance of an anchor	Resistance of a permanent anchor at the serviceability limit state	$\gamma_{Ra,SLS}$	Not used	1.2[a]
	Resistance of a temporary anchor at the serviceability limit state			1.1[a]
Suitability and Acceptance Tests	Resistance of a permanent anchor at the serviceability limit state	$\gamma_{Ra,SLS,test}$		1.25[b]
	Resistance of a temporary anchor at the serviceability limit state			1.15[b]

[a] See Formula (8.11)

[b] See Formulae (8.13) and (8.15)

8.8 Implementation of design

8.8.1 General

(1) prEN 1997-1:2022, 10 shall apply to anchors.

(2) Execution of grouted anchors should comply with EN 1537.

(3) Execution of non-grouted anchors should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(4) In addition to (2) the specifications shall be given in the Geotechnical Design Report and in the execution specification.

(5) Prior to their usage, it should be demonstrated that the anchor components have the required performance and durability as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

8.8.2 Supervision

(1) prEN 1997-1:2022, 10.2 shall apply to anchors.

(2) In addition to prEN 1997-1:2022, 10.2, supervision of the installation and testing of anchors should comply with EN 1537.

8.8.3 Inspection

(1) prEN 1997-1:2022, 10.3 shall apply to anchors.

(2) In addition to prEN 1997-1:2022, 10.3, inspection of the installation and testing of anchors should comply with EN 1537.

8.8.4 Monitoring

(1) prEN 1997-1:2022, 10.4 shall apply to anchors.

(2) In addition to prEN 1997-1:2022, 10.4, monitoring of grouted anchors should comply with EN 1537.

8.8.5 Maintenance

(1) prEN 1997-1:2022, 10.5 shall apply to anchors.

8.9 Testing

8.9.1 General

(1) prEN 1997-1:2022, 11 shall apply to anchors.

(2) Testing of grouted anchors should comply with one of the test methods given in EN ISO 22477-5.

NOTE 1 Test Method to be used can be specified in the National Annex.

NOTE 2 Limiting values for creep in investigation, suitability and acceptance tests are given in Table 8.3 (NDP) unless the National Annex gives different values.

Table 8.3 — (NDP) Limiting criteria for investigation, suitability and acceptance tests at the ultimate and serviceability states

<table>
<tr><th rowspan="3">Test method</th><th rowspan="3">Parameter[a]</th><th rowspan="3">Anchor type</th><th rowspan="2">Investigation test</th><th colspan="2">Suitability test</th><th colspan="2">Acceptance test</th></tr>
<tr><th>α_{ULS}</th><th>α_{SLS}</th><th>α_{ULS}</th><th>α_{SLS}</th></tr>
<tr><th>α_{ULS}</th><th>(8.12)</th><th>(8.13)</th><th>(8.14)</th><th>(8.15)</th></tr>
<tr><td>1</td><td>α_1</td><td>All</td><td>2 mm</td><td>2 mm</td><td>Not used</td><td>2 mm</td><td>Not used</td></tr>
<tr><td rowspan="2">3</td><td rowspan="2">α_3</td><td>Temporary</td><td rowspan="2">5 mm</td><td rowspan="2">Not used</td><td>1,2 mm</td><td rowspan="2">Not used</td><td>2,5 mm</td></tr>
<tr><td>Permanent</td><td>1,0 mm</td><td>1,5 mm</td></tr>
<tr><td colspan="8">[a] Creep rate per log cycle of time</td></tr>
</table>

(3) Testing of non-grouted anchors should be carried out in accordance with EN ISO 22477-5, unless specified otherwise by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

8.9.2 Grout

(1) The compressive strength of grout used for load transfer shall be verified by testing prior to the use of grout for anchor installation.

(2) The testing of compressive strength of grout used for load transfer shall be conducted by two series of tests for every 20 m³ of mixed grout.

(3) Each series of tests shall comprise 3 samples.

8.9.3 Investigation tests

(1) The proof load in investigation tests should be estimated from the expected geotechnical resistance of the anchor at the ultimate limit state.

NOTE Limit values for creep at the proof load in investigation tests are given in 8.9.1

(2) Grouted anchors with tendon bond lengths spaced less than 1,5 m centre to centre should be tested in groups of three anchors unless comparable experience has shown that the interaction has no quantifiable adverse effects.

(3) Anchors for investigation tests should comply with EN ISO 22477-5.

8.9.4 Suitability tests

(1) Suitability tests shall be used to verify that specified criteria are not exceeded at a proof load, P_P, determined from Formula (8.12) for Test Method 1 or (8.13) for Test Method 3:

$$P_P \geq \xi_{a,ULS,test} \cdot \gamma_{Ra,ULS} \cdot E_{d,ULS} \tag{8.12}$$

$$P_P \geq \xi_{a,SLS,test} \cdot \gamma_{Ra,SLS,test} \cdot E_{d,SLS} \tag{8.13}$$

where:

$E_{d,ULS}$	is the design value of the effects of actions at the ultimate limit state (see formula 8.2);
$E_{d,SLS}$	is the design value of the maximum anchor force, including the lock-off load, and sufficient to prevent the serviceability limit state in the anchored structure;
$\gamma_{Ra,ULS}$	is a partial factor on the anchor's geotechnical resistance at the ultimate limit state, given in 8.6.4;
$\gamma_{Ra,SLS,test}$	is a partial factor on the anchor's geotechnical resistance in suitability and acceptance tests at the serviceability limit state, given in 8.7.3;
$\xi_{a,ULS,test}$, $\xi_{a,SLS,test}$	are correlation factors, taking account of the number of suitability tests.

NOTE 1 The values of $\xi_{a,ULS,test}$ and $\xi_{a,SLS,test}$ are 1,0 unless the National Annex gives different values.

NOTE 2 Limit values for creep in suitability tests are given in 8.9.1

(2) Unless comparable experience has shown that the interaction has no quantifiable adverse effects, grouted anchors with tendon bond lengths spaced at less than 1,5 m centre to centre, should be tested in groups of three anchors.

(3) Grouted anchors for suitability tests should comply with EN ISO 22477-5.

(4) The apparent tendon free length of a grouted anchor should comply with EN 1537.

8.9.5 Acceptance tests

(1) Acceptance tests shall be carried out on all anchors prior to their lock off and before they become operational.

(2) Acceptance tests shall be used to verify that specified limiting criteria are not exceeded at the proof load, P_P, given by Formulae (8.14) for Test Method 1 or (8.15) for Test Method 3:

$$P_P = \gamma_{Ra,ULS} \cdot E_{d,ULS} \tag{8.14}$$

$$(13) P_P = \gamma_{Ra,SLS,test} \cdot E_{d,SLS} \tag{8.15}$$

where:

$\gamma_{Ra,ULS}$ is a partial factor on the anchor's geotechnical resistance at the ultimate limit state, given in 8.6.4;

$\gamma_{Ra,SLS,test}$ is a partial factor on the anchor's geotechnical resistance in suitability and acceptance tests at the serviceability limit state, given in 8.7.3.

NOTE Limit values for creep in acceptance tests are given in 8.9.1

(3) The apparent tendon free length of a grouted anchor shall comply with EN 1537.

(4) For grouted anchors, where tendon bond lengths of a group of anchors cross at spacings less than 1,5 m (centre to centre), the pre-stress should be checked on selected anchors after completion of the lock-off process.

8.10 Reporting

(1) prEN 1997-1:2022, 12, shall apply to anchors.

(2) In addition to prEN 1997-1:2022, 12, reporting for grouted anchors should comply with EN 1537 and EN ISO 22477-5.

(3) In addition to prEN 1997-1:2022, 12, reporting for non-grouted anchors should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

9 Reinforced fill structures

9.1 Scope and field of application

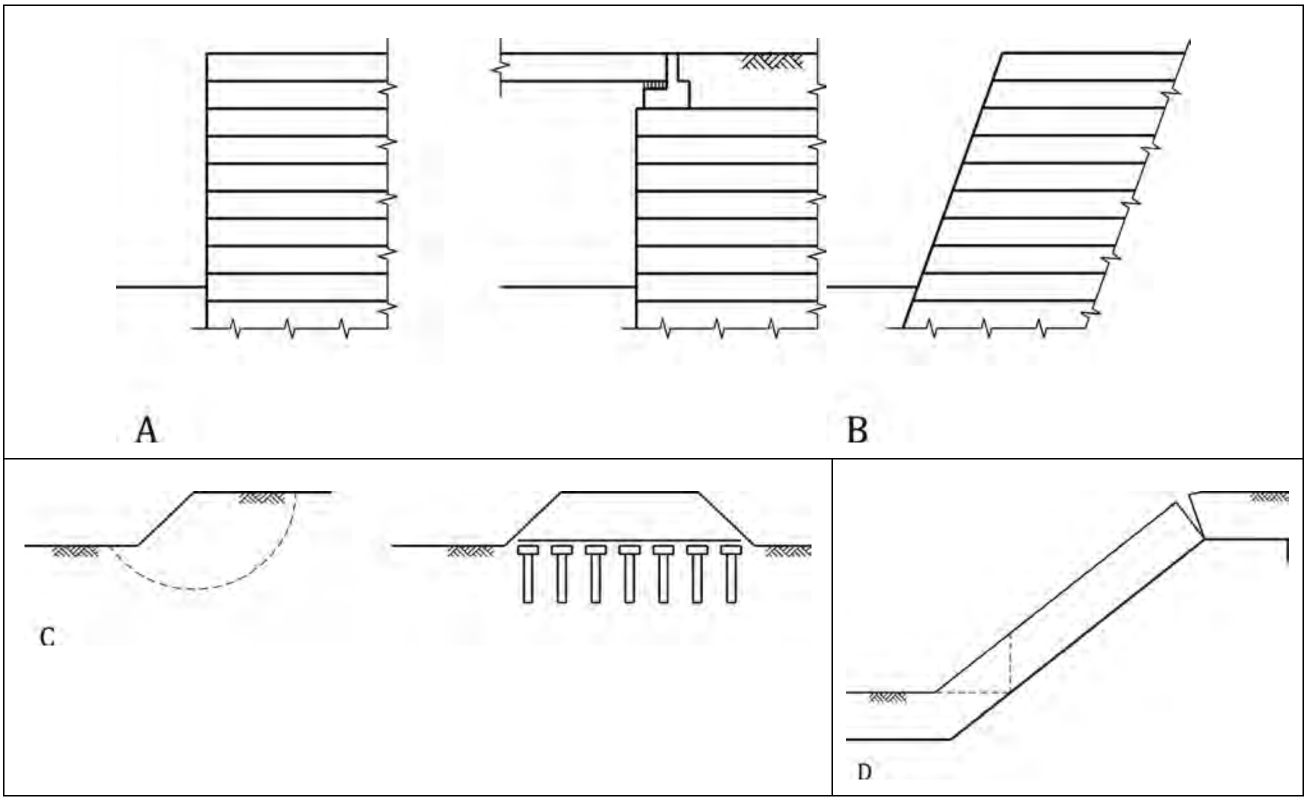
(1) This Clause shall apply to reinforced fill structures.

NOTE 1 Reinforced fill structures include those in Figure 9.1.

NOTE 2 Earthwork structures without reinforcement are covered by Clause 4 embankments.

NOTE 3 Design of asphalt reinforcement of pavements, is not covered by this standard.

NOTE 4 Geotextile encased columns are covered in Clause 11.



Key

A Reinforced wall and abutments

B Reinforced slope

C Basal reinforcement for embankments (including load transfer platforms over inclusions and voids overbridging)

D Venner reinforcement

Figure 9.1 — Reinforced fill structures within the scope of Clause 9

9.2 Basis of design

9.2.1 Design situations

(1) prEN 1997-1:2022, 4.2.2 shall apply to reinforced fill structures.

9.2.2 Geometrical properties

9.2.2.1 General

(1) prEN 1997-1:2022, 4.3.3 shall apply to reinforced fill structures.

9.2.2.2 Reinforcing elements

(1) If the design of a reinforced fill structure is sensitive to deviations in the location of the reinforcing elements or other geometrical properties, the verification of limit states shall include determination of allowable construction tolerances.

NOTE The sensitivity depend on type of reinforcement, type of reinforcing element and applied design method.

9.2.3 Zone of influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to reinforced fill structures.

9.2.4 Actions and environmental influences

9.2.4.1 General

(1) EN 1997-1:2022, 4.3.1 shall apply to reinforced fill structures.

9.2.4.2 Permanent and variable actions

(1) Design value of the force in the reinforcement elements shall be obtained from verification of limit states for the reinforced structure.

(2) The design resistance of reinforcement elements shall be sufficient to prevent the following limit states being exceeded by the reinforced fill structure:

- failure by overall stability, determined in accordance with Clause 4.
- failure by loss of bearing capacity determined in accordance with Clause 5.
- failure by sliding determined in accordance with Clause 5.
- failure by loss of static equilibrium determined in accordance with Clause 7.

(3) Traffic load should be included in verifications of reinforced fill structures.

NOTE Guidance on traffic load is given in prEN 1991-2:2022, Clause 6.9 and 8.10.

(4) Seepage forces due to different groundwater levels behind and in front of a reinforced structure shall be considered as actions, in accordance with 9.4, as appropriate.

9.2.4.3 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to reinforced fill structures.

9.2.4.4 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.5 shall apply to reinforced fill structures.

(2) The effects of temperature on the durability due to chemical degradation of geosynthetic reinforcing elements shall be determined using the equivalent constant in-soil temperature, T_{eq}.

(3) The effects of temperature on the creep of geosynthetic reinforcing elements shall be determined using the equivalent constant in-soil temperature, T_{eq}.

(4) The value of T_{eq} may be specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

(5) In the absence of a specified temperature or site-specific in-soil temperature data, the value of T_{eq} should be taken as either:

- a temperature midway between the average yearly air temperature and the average daily air temperature for the hottest month at the site; or
- a temperature derived from a validated temperature-dependent kinetic degradation model applied to site-specific in-soil temperature range and variations.

(6) Measures should be taken to avoid adverse swelling or expansion of frost susceptible soils in the ground near the surface of reinforced structures.

NOTE Possible measures include selection of suitable backfill material, drainage, or insulation.

(7) Chemical components of ground or groundwater that can adversely affect the durability of the reinforcement element or the resistance at the ground/reinforcement interface shall be considered.

(8) Temporary degradation of geosynthetic reinforcement by UV exposure shall be considered.

9.2.5 Limit states

9.2.5.1 Ultimate Limit State

(1) In addition to prEN 1997-1:2022, 8, the following ultimate limit states shall be verified for all reinforced fill structures:

- rupture of the reinforcing element;
- rupture of any connection between a reinforcing element and the facing of the structure or between the reinforcing elements themselves;
- failure along slip surfaces that pass wholly or partially through the reinforced block;
- failure at the interface between the ground and the reinforcing element beyond the assumed slip surface (pullout);
- failure by sliding between the ground and reinforcing element;
- failure by sliding between the reinforced block and its foundation;
- structural failure of any facing element;
- potential brittle failure in the reinforcing elements;
- failure of the connection between any facing elements;
- bearing failure of the foundation;
- squeezing of any weak foundation soils;
- excessive deformation in the reinforcement elements over the design life of the structure.

(2) Potential ultimate limit states other than those given in (1) should be verified.

9.2.5.2 Serviceability Limit State

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all reinforced structures:

- deformations of the reinforced fill structure itself;
- differential settlement along the facing due to subsoil deformation;
- differential movement between facing and reinforcing element;
- deformation of the reinforced fill structure, which can cause serviceability limit states of nearby structures or services that rely on it;
- bulging and deformation of the face;
- cracking or spalling of precast facing panels due to differential settlement or movement.

(2) Potential serviceability limit states other than those given in (1) should be verified.

9.2.6 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply to reinforced fill structures.

9.2.7 Ground investigation

9.2.7.1 General

(1) prEN 1997-2:2022, 5 shall apply to reinforced fill structures.

(2) Chemical properties of ground and groundwater should be determined for durability assessment of any reinforcing elements, connections and facing elements.

9.2.7.2 Minimum extent of field investigation

(1) The depth and horizontal extent of the field investigations shall be sufficient to determine the ground conditions within the zone of influence in accordance with prEN 1997-1:2022, 4.1.2.1

(2) The depth of the in-situ testing for application of reinforced fill as wall and abutments shall comply with 7.2.7.2.

(3) The depth of the in-situ testing for application of reinforced fill as reinforced slope, basal reinforcement and reinforced embankments shall comply with 4.2.7.2.

9.2.8 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to reinforced fill structures.

9.3 Materials

9.3.1 Ground properties

(1) prEN 1997-2:2022, Clause 7 to12 shall apply to reinforced fill structures.

NOTE For classification of fill see EN 16907-2.

9.3.2 General related to durability

(1) prEN 1997-1:2022, 4.1.6 shall apply to reinforced fill structures.

(2) Determination of the loss of strength of reinforcing elements for fills shall, for the structures intended design service life, take account of the long-term effects of sustained load in reinforcement (creep) and long-term changes in fill properties.

(3) In addition to (1) the potential damage of the reinforcement during transport, storage and installation shall be considered.

9.3.3 Geosynthetics

(1) In addition to prEN 1997-1:2022, 5.3, geosynthetic reinforcing elements should comply with EN 13251.

(2) The characteristic tensile strength of geosynthetic reinforcement, T_{k} should be determined in accordance with EN ISO 10319.

(3) When the strength of geosynthetic material is required for specific elongation, either total or relative between given times, the characteristic tensile strength including the creep reduction $T_{\mathrm{k,cr}}$ shall be determined from isochronous creep curves.

NOTE Relative elongation between given times can be related to post construction elongation or specified design service life in voids overbridging application.

(4) In addition to 9.3.2 (1), a reduction factor η_{gs} shall be applied to the tensile strength of geosynthetic reinforcing elements to account for loss of strength.

(3) The representative tensile resistance $R_{t,rep,el}$ of a geosynthetic reinforcing element shall be determined from Formula (9.1):

$$R_{t,rep,el} = \eta_{gs} T_k \tag{9.1}$$

where:

T_k is the characteristic tensile strength of the reinforcing element see (2);

η_{gs} is a reduction factor accounting for anticipated loss of strength with time and other influences.

(5) The reduction factor η_{gs} should account for the adverse effect of:

– tensile creep due to sustained static load over the design service life of the structure at the design temperature;
– the adverse effects of mechanical damage during transportation, installation and execution;
– weathering;
– chemical and biological degradation of the reinforcing element over the design service life of the structure at the design temperature;
– intense and repeated loading over the design service life of the structure (fatigue); and
– joints and seams for geosynthetic reinforcing elements and polymeric coated steel woven wire mesh.

NOTE Guidance on determination of the reduction factor is given in F.8.1

9.3.4 Steel

(1) Reinforcement in the form of strips, bars or rods, welded wire ladders and meshes shall comply with EN 10025 (all parts), or EN 10080, as appropriate for the type of steel used.

(2) The nominal yield strength fy for unprotected steel used in reinforced fill structures shall be not more than 500 Mpa.

(3) The nominal yield strength f_y for protected (galvanized) steel used in reinforced fill structures shall be not more than 600 Mpa.

NOTE Strengths of steels are limited for durability reasons and the risk of embrittlement. The susceptibility of steel to hydrogen embrittlement and stress corrosion cracking is influenced by the microstructure of the steel as well as the strength of the steel.

(4) The provisions on ductility of prEN 1993-1-1:2022, 5.2.2, shall apply to all elements.

(5) Alternative to (4), reinforcing steel manufactured to EN 10080 that complies with Class B of prEN 1992-1-1:2021 Table 5.5 may be used.

NOTE Typical steels used that meet the requirements of this document are given in Annex F9.

(6) If a steel reinforcing element is galvanised, the hot dip galvanized coating shall comply with EN ISO 1461.

(7) Reinforcing elements made from stainless steel or aluminium alloys shall only be used if they comply with a standard specified by the relevant authority or, where not specified, agreed for a specific project by appropriate parties.

(8) The design tensile resistance of steel reinforcing elements in reinforced fill structures $R_{td,el}$ shall be determined from Formula (9.2):

$$R_{td,el} = A_r f_{yd} \tag{9.2}$$

where:

f_{yd} is the design yield strength of the steel:

for structural steel complying with EN 10025 (all parts), $f_{yd} = f_y / \gamma_{M0}$, where f_y is the characteristic yield strength of the steel and γ_{M0} is a partial factor; and

for reinforcing steel complying with EN 10080, $f_{yd} = f_{0.2k} / \gamma_S$ where $f_{0.2k}$ is the characteristic proof strength at 0.2 % strain of the steel and γ_S is a partial factor;

A_r is the reduced gross cross-sectional area of the reinforcing element at the weakest section, allowing for the effects of potential corrosion.

NOTE Values of γ_{M0} and γ_S are given in Table 9.3 [NDP], unless different values are given in the National Annex.

(9) The design tensile resistance of steel reinforcing elements at terminations and connections $R_{td,con}$ in reinforced fill structures shall be determined from Formula (9.3):

$$R_{t,rep,el} = min\left(k_t A_{s,con} f_{ud}; A_r f_{yd}\right) \tag{9.3}$$

where:

f_{ud} is the design tensile strength of the steel;

for structural steel complying with EN 10025 (all parts), $f_{ud} = f_u / \gamma_{M2}$ where f_u is the characteristic tensile strength of the steel and γ_{M2} is a partial factor; and

for reinforcing steel complying with EN 10080, $f_{ud} = f_{tk} / \gamma_t$ where f_{tk} is the characteristic tensile strength of the steel and γ_t is a partial factor

$A_{s,con}$ is the net reduced cross-sectional area of the reinforcing element, allowing for the effects of potential corrosion, at the termination or connection;

k_t (≤ 1) is a calibration factor accounting for the influence of the termination on the measured breaking strength of the element.

(10) The ultimate resistance of terminations and connections shall comply with prEN 1993-1-8.

(11) The value of kt should be determined by testing certified by a Technical Assessment Body.

(12) In the absence of a value determined by testing, the value of kt in Formula (9.4) may be taken as:

– for sections with smooth holes (i.e. holes without notches), including holes fabricated by drilling or water jet cutting, $k_t = 1{,}0$;

– for sections with rough holes (i.e. holes with notches), including holes fabricated by punching or flame cutting, $k_t = 0{,}9$; or
– for sections with threads, $k_t = 0{,}9$.

(13)The cross-sectional area of steel reinforcing elements shall be reduced by an amount based on the potential average loss of thickness Δe around the exposed surface caused by corrosion in the ground, as shown in Figure 9.2.

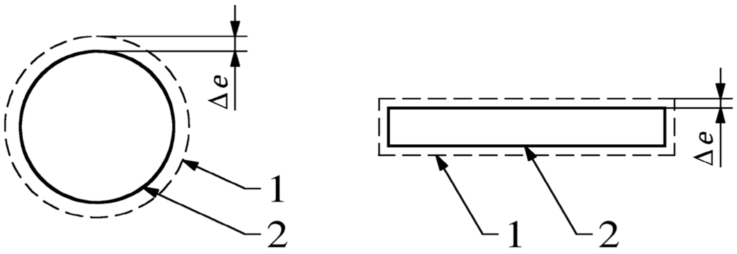


Figure 9.2 — Reinforced fill structures within the scope of Clause 9

(14)For soils and fills that comply with the electro-chemical properties of Table B.1 of EN 14475:2006, the value of Δe shall be determined from Formula (9.4):

$$\Delta e = k_{cc} max(AT^{n} - e_{z}; 0) \qquad (9.4)$$

where:

A	is the loss of metal (including zinc) per face over the first year;
T	is the design service life of the structure in years;
n	is an exponent accounting for reduction in corrosion rate in time;
e_z	is the initial local zinc coating thickness (minimum 70μm); and.
K_{cc}	is a corrosion concentration factor, accounting for concentrated areas of corrosion and depending on the steel manufacturing process.

NOTE 1 For steel in surrounding soils that comply with the electrochemical properties of Table B.1 of EN 14475:2006, values for *A* and *n* are given in in Table 9.1 (NDP) unless the National Annex give a different value.

NOTE 2 Values of kcc are given in Table 9.2 (NDP), unless the National Annex give different values

Table 9.1 — (NDP) Corrosion parameters for fill steel reinforcement

Steel	*A* (µm)		*n*	
	Land-based	Fresh water	Land-based[a]	Fresh water[b]
Galvanized[c]	25	40	0.65	0.60
Non-galvanized			0.80	0.75

[a] Land-based = without influence of groundwater or surface water

[b] Fresh water = installed fresh water or regularly submerged [EN 14490]

[c] Hot-dip galvanisation per EN ISO 1461, with a minimum local coating thickness of 70µm

Table 9.2 — (NDP) Corrosion concentration factor, k_{cc}

Steel	Strip thickness[a] (mm)	Bar diameter (mm)	*Corrosion concentration factor k_{cc}*[b,c]	
			Steel reinforcing element with uniform strength distribution across its section	Steel reinforcing element with non-uniform (or unknown) strength distribution across its section
Galvanized	4-6	6-18	2.0	1,5
	> 12	> 40	1.0	1.0
Non-galvanized	4-6	6-18	2.5	2,0
	> 12	> 40	1.0	1.0

[a] For strips 6-12 mm thick and bars 18-40 mm in diameter, interpolate between the values given

[b] Some manufacturing methods result in steel properties varying across the section with higher strengths towards the outer surface. This can affect tensile resistance disproportionally.

[C] Annex F.9 for examples of steels with uniform and non-uniform strength distributions.

(15) The value of k_{cc} may be determined by testing, provided the test data is certified by a Technical Assessment Body and the value of kcc is not less than that given for steel with a uniform strength distribution.

(16) For soils and fills that do not comply with the electro-chemical properties of Table B.1 of EN 14475:2006, the value of Δe shall be determined by tests in the specific ground conditions.

(17) The reduced cross-sectional area of a steel reinforcing element Ar shall not be less than 50 % of its initial cross-sectional area

9.3.5 Polymeric coated steel woven wire meshes

(1) Reinforcement in the form of polymer coated woven wire mesh should comply with EN 10218-2, in case of steel wire only and EN 10223-3 for the whole reinforcement product.

(2) Polymeric coated steel woven wire meshes shall be treated with a zinc-aluminium alloy coating (Zn95Al5 or Zn90Al10) conforming to EN 10244-2, the minimum coating unit weight shall comply with Table 2 of EN 10244-2:2009 and further protected by:

- PVC coating conforming to EN 10245-2; or
- PE coating conforming to EN 10245-3; or
- PET coating conforming to EN 10245-4; or
- PA coating conforming to EN 10245-5.

(3) The characteristic tensile strength of polymeric coated steel woven wire mesh reinforcement shall be determined in accordance with EN ISO 10319.

(4) The representative tensile resistance $R_{\mathrm{t,rep,el}}$ of a polymeric coated woven wire mesh reinforcing element shall be determined from Formula (9.5):

$$R_{\mathrm{t,rep,el}} = \eta_{\mathrm{pwm}} T_{\mathrm{k}} \tag{9.5}$$

where:

T_{k} is the characteristic tensile strength of the reinforcing element;

η_{pwm} is a reduction factor accounting for anticipated loss of strength with time and other influences.

(5) In addition to 9.3.2 (1), a reduction factor η_{pwm} shall be applied to the tensile strength of polymeric coated steel woven wire meshes to account for the loss of strength.

NOTE Guidance on determination of the reduction factor is given in F.8.2

(6) The evaluation of η_{dmg} shall account for the decrease of tensile strength at short term due to damage during transportation, installation and execution.

(7) The evaluation of η_{cor} shall account for the loss of protection to the metallic wires caused by mechanical damage during execution to the polymeric and zinc-aluminium alloy coatings as well as to the metallic wires.

NOTE The polymeric and a zinc-aluminium alloy coating have no structural function, since theirs only purpose is to protect the metallic wires.

(8) If the polymeric coated steel woven wire mesh is cut, the coating should be treated as damaged.

9.3.6 Other materials

(1) Materials other than those specified in 9.3.3, 9.3.4, and 9.3.5 should only be used for reinforcement if they comply with a standard specified by the relevant authority or, where not specified, agreed for a specific project by appropriate parties.

9.4 Groundwater

9.4.1 General

(1) prEN 1997-1:2022, 6, shall apply to reinforced fill structures.

9.4.2 Groundwater control system

(1) Clause 12 shall apply to reinforced fill structures.

(2) If a groundwater control system in not provided, then the reinforced fill structure shall be designed to withstand potential water pressures.

9.5 Geotechnical analysis

9.5.1 General

(1) prEN 1997-1:2022, 7 shall apply to reinforced fill structures.

(2) The external and compound stability of a reinforced fill structure, should be analysed according to Clauses 4, 5, or 7, with the beneficial effect of reinforcing elements.

(3) The internal stability of a reinforced fill structure shall be analysed according to the type of reinforced fill structure.

NOTE The residual effects of compaction can be significant, when determining the design load and elongation of the uppermost layers of reinforcement.

(4) Horizontal and vertical deformations of a reinforced fill structure shall be analysed according to Clauses 4, 5, or 7, as appropriate.

(5) The compound stability of reinforced slopes, walls, and bridge abutments may be verified using a method not given in 9.5.2.1(1) provided it has been validated against comparable experience.

(6) Verification of the compound stability of a reinforced fill structure shall include the potential beneficial effect of any reinforcing elements.

9.5.2 Mode of failure for reinforced fill structures

9.5.2.1 Reinforced slopes, walls, and bridge abutments

(1) The internal stability of reinforced slopes, walls, and bridge abutments should be verified using one or more of the following methods:

– coherent gravity method;
– tie-back wedge method;
– multiple wedge method;
– slope stability methods;
– numerical methods.

NOTE Details of some of these methods are given in Annex F.3.

(4) Other methods than those given in (1) may be used.

9.5.2.2 Basal reinforcement for embankments

(1) When analysing potential excessive deformation on embankment edges, resistance to extrusion shall be verified.

(2) Potential excessive deformation due to consolidation should be verified.

(3) Resistance to horizontal sliding over the basal reinforcement shall be verified.

NOTE Details of these checks are given in Annex F.4.

(4) Temporary roads and/or working platforms with basal reinforcement over low strength fine soil shall be analysed as low height embankments.

(5) If the height of the embankment prevents uniform distribution of concentrated loads above the reinforcing element, local bearing resistance shall be verified according to Clause 5.

9.5.2.3 Load transfer platforms over piles and rigid inclusions

(1) Load transfer platforms may be used over piles and discrete inclusions to allow bigger spacing and limit differential deformation on embankment surface.

(2) Rigid inclusions shall be designed according to Clause 11 and piles according to Clause 6.

(3) When analysing embankment edges outside the inclusion zone, analyses according to 9.5.2.2 shall be performed.

(4) The load distribution from an embankment through the load transfer platform should be analysed using one or more of the following methods:

- Hewlett and Randolph method ;
- EBGEO method ;
- Concentric Arches method;
- numerical methods.

NOTE Details of these methods are given in Annex F.5.

(5) Load transfer through a load transfer platform may be analysed using a method not given in (4) provided it has been validated against comparable experience.

9.5.2.4 Overbridging systems in areas prone to subsidence

(1) Overbridging systems that include reinforcing elements may be used over areas prone to subsidence to limit differential deformation on surface.

(2) The structure shall be designed to identify the location of any new void readily and quickly and to ensure the void can be remediated within the specified short-term design period.

(3) In persistent design situations, it shall be verified that the reinforcement satisfies the long-term strain criteria required to ensure that the surface deformations remain within limiting design value of the deformation and that the supporting ground around the void will remain stable for the design life of the structure.

(4) Loads in reinforcing elements should be determined assuming that all of the following failure mechanisms, depending on the ratio of the structure's height above the void (H) to the diameter of the void (D):

- failure of the bridging zone without lateral support, which generally applies to $H/D \leq 1$;
- failure of the bridging zone with lateral support, which generally applies to $H/D > 1$;
- failure below developed arch in stabilised soil, which generally applies to permanent design situations.

NOTE Details of these methods are given in Annex F.6.

(5) Loads in reinforcing elements may be determined using a method not given in (5) provided it has been calibrated and validated against comparable experience.

9.5.2.5 Veneer stability

(1) It shall be verified that the resistance of reinforcing elements along the underlying slope is greater than the load effect generated by the cover soil sliding over the weakest linear slip surface.

NOTE The reinforcement is in direct contact to the cover soil and the active soil mass.

(2) The loads shall be determined using the plane of least frictional resistance in the veneer cover package.

(3) The stability of the veneer layer subject to traffic load shall be verified for a transient design situation.

(4) The stability of the anchorage at the top of the veneer, and any intermediate anchorages down the slope, shall be verified.

(5) The stability of the veneer shall be verified considering the formation of a water table inside the veneer soil.

NOTE Further details are given in Annex F.7.

9.5.3 Resistance of reinforcing elements

9.5.3.1 General

(1) The representative tensile resistance ($R_{t,rep}$) of a reinforcing element shall be determined from Formula (9.3):

$$R_{\mathrm{t,rep}} = \min\left(R_{\mathrm{t,rep,el}}; R_{\mathrm{rep,po}}; R_{rep,ds}; R_{\mathrm{rep,con}}\right) \quad (9.6)$$

where:

$R_{t,rep,el}$ is the representative tensile resistance strength of the reinforcing element;

$R_{rep,po}$ is the representative value of the pull-out resistance mobilised along the interface between the fill and the reinforcing element;

$R_{rep,ds}$ is the representative value of the direct shear resistance;

$R_{rep,con}$ is the representative value of the resistance at the connection both at the point between the facing and the reinforcing element (i.e., connection device), and the reinforcement at the connection point.

(2) Where the reinforcing element is assumed to carry shear loads, the shear structural resistance shall be determined according to the relevant Eurocode for combined axial, shear, and bending actions.

(3) Any shear resistance that is assumed in the calculation shall be limited to punching shear capacity of the surrounding ground.

9.5.4 Pull-out resistance

9.5.4.1 General

(1) The resistance of a reinforcing element to pull-out from the fill shall be verified both from the point of maximum tension, or the intersection point between the reinforcement and the verified failure line, towards non-connected ends.

(2) The representative pull-out resistance ($R_{rep,po}$) of a reinforcing element shall be determined from Formula (9.7):

$$R_{\text{rep,po}} = \int_0^{L_{\text{po}}} P(x) \cdot \tau_{\text{po}}(x) \cdot dx \qquad (9.7)$$

where:

P(x) is the length of the perimeter of the reinforcing element at point X;

τ_{po} is the representative shear resistance against pull-out along the soil-reinforcement interface;

x is distance along the length of the reinforcing element;

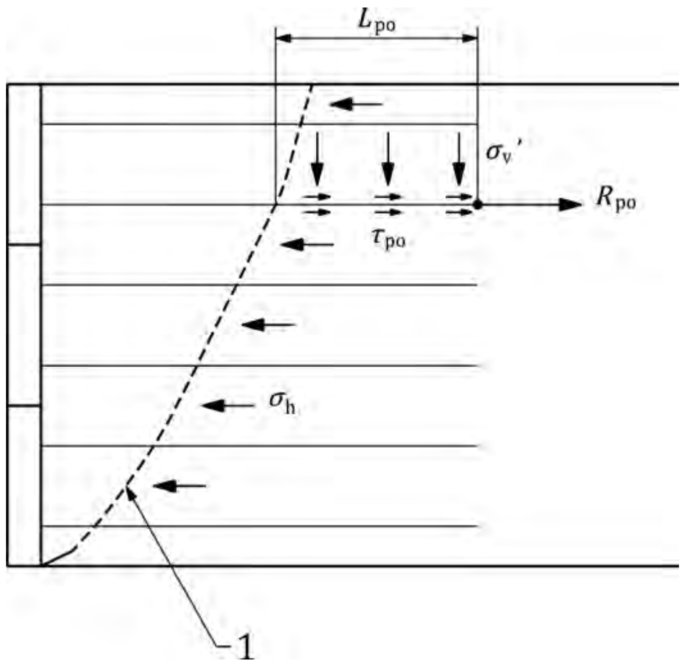
L_{po} is the total length of the reinforcing element beyond the failure surface (or line of maximum tension) where pull-out stresses are mobilized.

NOTE Pull-out resistance can be influenced by dynamic action.

(3) The perimeter for the reinforcing element at point x should be determined with consideration of type of reinforcing element and the interaction between multiple layers.

(4) If the reinforcing element is situated between two different soils the properties of the weaker should be used for determination for the representative pull-out resistance.

NOTE Figure 9.3 gives an example of pull-out analysis of the reinforcing element embedded in the resistant zone.



Key

1	failure surface
R_{po}	pull-out resistance
τ_{po}	shear resistance against pull-out
L_{po}	length of the reinforcing element beyond the failure surface

Figure 9.3 — Example of pull-out analysis at the embedded end of reinforcing elements

(5) The pull-out resistance shall be based on documented tests in comparable situations or from project-specific tests.

(6) The pull-out resistance from the face of the structure should be increased by any mechanical connection resistance between facing and reinforcing element as determined according to 9.5.6.

9.5.4.2 Sheet reinforcement for fill

(1) For sheet reinforcement (geogrids and geotextiles), the value of τ_{po} in Formula (9.7) shall be determined from Formula (9.8):

$$\tau_{\mathrm{po}}(x) = k_{\mathrm{po}} \tan \varphi_{\mathrm{rep}}\, \sigma'_{\mathrm{n}}(x) \tag{9.8}$$

where:

φ_{rep} is the representative coefficient of friction of the surrounding soil;

σ'_n is the normal effective stress acting on the reinforcing element at point x;

x is a variable which represents space along the length of the reinforcing element.;

k_{po} is a pull-out factor determined in laboratory pull-out tests in representative conditions, from comparable experience, or from field tests.

(2) If validated by comparable experience, cohesion may be added to Formula (9.8).

9.5.4.3 Discrete fill reinforcement

(1) For discrete fill reinforcement (strips and ladders), the value of τ_{po} in Formula (9.7) shall be determined from Formula (9.9):

$$\tau_{\mathrm{po}}(x) = \mu_{\mathrm{po}}\sigma'_{\mathrm{n}}(x) \tag{9.9}$$

where, in addition to the symbols given for Formula (9.8):

μ_{po} is the coefficient of interaction determined in laboratory tests in representative conditions or from field tests.

(2) If validated by comparable experience, cohesion or passive resistance may be added to Formula (9.9).

9.5.5 Resistance in direct shear

(1) The representative resistance to direct shear ($R_{k,ds}$) shall be determined from Formula (9.10):

$$R_{\mathrm{rep,ds}} = B\int_0^{L_{\mathrm{ds}}} \tau_{\mathrm{ds}}(x)\cdot dx = B\int_0^{L_{\mathrm{ds}}} f_{\mathrm{ds}}\sigma'_{\mathrm{n}}(x)\cdot dx \tag{9.10}$$

where:

B is the breadth of the reinforcing element;

τ_{ds} is the resistance against direct shear along the soil-reinforcement interface;

x is distance along the length of the reinforcing element;

L_{ds} is the total length of the reinforcing element along which direct shear stresses are mobilized;

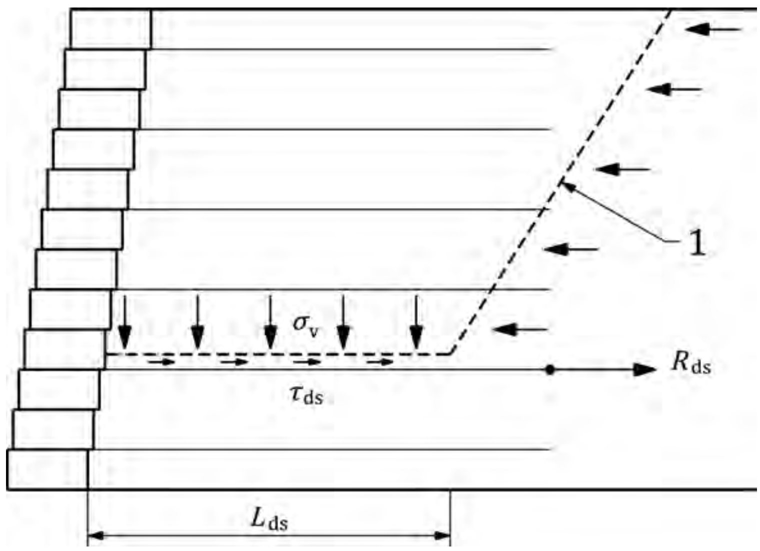
f_{ds} is a direct shear factor determined from direct shear tests or comparable experience;

σ'_n is the normal effective stress acting on the reinforcing element at the distance x.

NOTE 1 The vertical effective stress is a good approximation for the normal effective stress provided the inclination of the reinforcing element is less than 10° from horizontal.

(2) If validated by comparable experience, cohesion may be added to Formula (9.8).

NOTE Figure 9.4 gives an example of horizontal sliding analysis of a reinforced fill structure. The symbols are defined in Formula (9.10).



Key

1 failure surface

Figure 9.4 — Example of horizontal sliding analysis of a reinforced fill structure

(3) The value of f_{ds} for geosynthetic and polymeric coated steel woven wire meshes reinforcements shall comply with EN ISO 12957-1 for direct shear or EN ISO 12957-2 for shear along an inclined plane.

(4) Mobilized resistance between the base of the reinforced fill structure and the subsoil, shall be determined according to Clause 5.

9.5.6 Resistance of connections

(1) The resistance of the connection between the facing and reinforcing element shall be determined by testing the specific connection or by calculation.

(2) If it is determined by calculation, the representative tensile resistance of a mechanical connection for geosynthetics or polymer steel woven wire meshes ($R_{k,con}$) shall be determined from Formula (9.11):

$$R_{rep,con} = \eta_{el,con} T_{rep} \tag{9.11}$$

where:

T_{rep} is the representative tensile strength of the reinforcing element;

$\eta_{el,con}$ is a reduction factor accounting for anticipated loss of strength with time and from other influences at the connection.

(3) The reduction factor $\eta_{el,con}$ shall be calculated from Formula (9.12) for geosynthetics or Formula (9.13) for polymer steel woven wire meshes:

$$\eta_{el,con} = \eta_{gs} \eta_{con,c} \tag{9.12}$$

$$\eta_{el,con} = \eta_{pwm} \eta_{con,c} \tag{9.13}$$

where:

$\eta_{con,c}$ is a reduction factor accounting for the reduction of resistance due to the connection;

η_{gs}, η_{pwm} are reduction factors accounting for the durability of the material (see F.8.).

(4) For steel reinforcing elements, if the determination is by calculation, $R_{rep,con}$ shall comply with prEN 1993-1-8.

(5) For connector components, $R_{rep,connector}$ shall be determined according to the material constituting the component and the relevant Eurocode.

(6) For the strength of the facing at connection, $R_{rep,con,fac}$ shall be determined according to the material constituting the component and the relevant Eurocode.

(7) When reinforcement is maintained by pull-out capacity between facing bloc, $R_{rep,con,po}$ shall be determined by testing.

(8) Where the reinforcing element is assumed to carry shear loads, the shear resistance of connection between facing and reinforcing element shall be determined according to the relevant Eurocode for combined axial, shear, and bending actions.

9.6 Ultimate limit states

9.6.1 General

(1) The design value of the ultimate limit state resistance of a reinforcement element shall comply with formula (9.14)

$$E_{\mathrm{d}} \leq \min(R_{\mathrm{t,d,el}}, R_{\mathrm{d,po}}, R_{\mathrm{d,ds}}, R_{\mathrm{d,con}}) \tag{9.14}$$

where:

E_d is the maximum value of the design value of the effects of actions in ultimate limit state (see 9.2.3.2);

$R_{t,d,el}$ is the design value of the resulting resistance of the reinforcement element;

$R_{d,po}$ is the design value of interface resistance between fill and reinforcement elements at the ultimate limit state (pullout);

$R_{d,ds}$ is the design value of direct shear mobilised along the interface between the fill or ground and the reinforcing element;

$R_{d,con}$ is the design tensile resistance of a connection for geosynthetics or polymer woven wire mesh.

9.6.2 Verification by the partial factor method

9.6.2.1 Rupture of the reinforcing elements (tensile)

9.6.2.1.1 Geosynthetics

(1) The design tensile resistance ($R_{t,d,el}$) of a geosynthetic reinforcing element shall be determined from Formula (9.15):

$$R_{\mathrm{t,d,el}} = \frac{R_{\mathrm{t,rep,el}}}{\gamma_{\mathrm{Rd,re}}\gamma_{\mathrm{M,re}}} \tag{9.15}$$

where:

$R_{t,rep,el}$ is the representative tensile resistance of the reinforcing element;

$\gamma_{M,re}$ is a partial factor, given in 9.6.2.6;

$\gamma_{Rd,re}$ is a model factor accounting for additional uncertainty owing to extrapolation of measured strengths to the design service life.

NOTE 1 A method to determine the value of $\gamma_{Rd,re}$ is given in ISO TR 20432, where it has the symbol f_s.

NOTE 2 The value of γ_{Rd} is 1.0 unless the National Annex gives another value.

9.6.2.1.2 Polymeric coated steel woven wire mesh

(1) The design tensile resistance ($R_{td,el}$) of polymeric-coated woven wire mesh reinforcing element shall be determined from Formula (9.16):

$$R_{td,el} = \frac{R_{t,rep,el}}{\gamma_{Rd}\gamma_{M,pwm}} \tag{9.16}$$

where:

$R_{t,rep,el}$ is the representative tensile resistance of the reinforcing element;

$\gamma_{M,pwm}$ is a partial factor, given in 9.6.2.5;

γ_{Rd} is a model factor accounting for additional uncertainty owing to extrapolation of measured strengths to the design service life.

NOTE 1 A method to determine the value of γ_{Rd} is given in ISO TR 20432, where it has the symbol f_s.

NOTE 2 The value of γ_{Rd} is 1.0 unless the national annex gives another value.

9.6.2.2 Failure at the interface between the fill and the reinforcing elements (pull-out)

(1) The design pull-out resistance ($R_{d,po}$) of a reinforcing element shall be determined from Formula (9.17):

$$R_{d,po} = \frac{R_{rep,po}}{\gamma_{R,po}} \tag{9.17}$$

where:

$R_{rep,po}$ is the representative pull-out resistance of the reinforcing element;

$\gamma_{R,po}$ is a partial factor, given in 9.6.2.5.

9.6.2.3 Failure due to sliding in direct shear along interface

(1) The design resistance to direct shear along the interface between the fill or ground and the reinforcing element ($R_{d,ds}$) shall be determined from Formula (9.18):

$$R_{\mathrm{d,ds}} = \frac{R_{\mathrm{rep,ds}}}{\gamma_{\mathrm{R,ds}}} \tag{9.18}$$

where:

$R_{\mathrm{rep,ds}}$ is the representative resistance to direct shear;

$\gamma_{\mathrm{R,ds}}$ is a partial factor, given in 9.6.2.6.

9.6.2.4 Rupture of the connections

(1) The design tensile resistance of a connection for geosynthetics or polymer woven wire meshes ($R_{\mathrm{d,con}}$) shall be determined from Formula (9.19):

$$R_{\mathrm{d,con}} = \frac{R_{\mathrm{rep,con}}}{\gamma_{\mathrm{R,con}}} \tag{9.19}$$

where:

$R_{\mathrm{rep,con}}$ is the representative tensile resistance at the connection;

$\gamma_{\mathrm{R,con}}$ is a partial factor for the connection, given in 9.6.2.6;

9.6.2.5 Failure of facing elements

(1) prEN 1997-3:2022, Clause 10 shall apply to reinforced fill structures.

9.6.2.6 Partial factors

(1) Partial factors for the verification of reinforced fill structures at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach.

NOTE Values of the partial factors are given in Table 9.3 (NDP) for persistent and transient design situations unless the National Annex gives different values.

Table 9.3 — (NDP) Partial factors for the verification of resistance of reinforced fill structures for fundamental (persistent and transient) design situations

Verification of	Partial factor on		Symbol	Resistance factor approach (RFA)
Overall and compound stability	See Clause 4			
Bearing resistance and sliding	See Clause 5			
Overturning	See Clause 7			
Pull-out failure of reinforcing elements	Pull-out resistance of	sheet fill reinforcement	$\gamma_{R,po,gs}$	1.25
		discrete fill reinforcement	$\gamma_{R,po,dis}$	1.25
		polymeric coated steel wire mesh reinforcement	$\gamma_{R,po,pwm}$	1.25
Direct shear failure along interface	Resistance to direct shear along interface for sheet fill reinforcement		$\gamma_{R,ds}$	1.25
Rupture of reinforcing element	Tensile strength of	geosynthetic reinforcement	$\gamma_{M,re}$	1.1
		structural steel per EN 10025-2 or EN 10025-4	$\gamma_{M0,}$	specified in prEN 1993-1-1
		steel reinforcement	γ_{M2}	specified in prEN 1993-1-1
		reinforcing steel per EN 10080	γ_{S}	specified in prEN 1992-1-1
		polymeric coated steel wire mesh reinforcement	$\gamma_{M,pwm}$	1.25
Rupture of connections between reinforcing elements	Tensile strength of polymeric coated steel wire mesh reinforcement		$\gamma_{R,con}$	1.25
	Tensile strength of polymeric coated steel woven wire mesh connection			1.35
	Geosynthetic			1.35
Rupture of connections to facing	Tensile strength		$\gamma_{R,con}$	1.35

9.7 Serviceability limit states

9.7.1 General

(1) prEN 1997-1:2022, 9 shall apply to reinforced fill structures.

9.7.2 Serviceability limit states of whole structure and it subsoil

(1) Verification of serviceability limit state due to loading of the reinforced fill structure including subsoil should comply with Clauses 4, 5, and 7.

(2) It shall be verified that the deformation of the reinforced fill structure is within the limiting values for the used facing elements.

NOTE The type of facing, if any, determines the amount of settlement that can be withstood. Guidance for typical values for different facing types is given in EN 14475.

9.7.3 Serviceability limit states of reinforced fill structure itself

(1) Total and differential deformation of the reinforced fill structure both vertically and horizontally shall be in compliance with the specified limiting values.

(2) Internal deformation of the reinforced fill structure shall comply with the specified limiting values.

9.7.4 Serviceability limit states of reinforcing element

(1) Elongation of the reinforcing elements both in the short and long term shall be in compliance with specified limiting values.

NOTE The serviceability limits for on post construction internal strains due to creep are usually taken as < 0.5 % for bridge abutments and < 1 % for retaining walls.

9.7.5 Serviceability limit states of facing element

(1) prEN 1997-3:2022, Clause 10 shall apply to reinforced fill structures.

9.8 Implementation of design

9.8.1 General

(1) prEN 1997-1:2022, 10 shall apply to reinforced fill structures.

(2) The execution and control of reinforced fill structures shall comply with EN 14475.

(3) The execution specification shall include level of the excavation with construction tolerances.

(4) Groundwater control measures shall be specified in accordance with Clause 12.

(5) The execution specification shall state requirements on properties of the fill needed to fulfil the verification of the limit states.

9.8.2 Inspection

(1) In addition to prEN 1997-1:2022, 10.3, the Inspection Plan should include, but is not limited to:

- verification of the quality of foundation ground, including as necessary placement of a concrete screed or a drainage layer properly compacted;
- verification of excavation levels within the specified tolerances;
- verification of properly compacted fill, if used;
- verification of the type, number, and arrangement of reinforcing elements;
- verification of the quality of the assembly of parts of the reinforcing elements;
- verification of facing system alignment/reinforcement connections;
- verification of adequate performance of any drainage system installed.

9.8.3 Monitoring

9.8.3.1 General

(1) In addition to prEN 1997-1:2022, 10.4, the Monitoring Plan should include, but is not limited to:

- behaviour of temporary support systems;
- monitoring of the behaviour of reinforcement element;
- lateral and vertical displacements and distortions.

9.8.4 Maintenance

(1) prEN 1997-1:2022, 10.5 shall apply to reinforced fill structures.

9.9 Testing

(1) prEN 1997-1:2022, 11 shall apply to reinforced fill structures.

9.9.1 Interface strength

(1) The determination of interface shear strength between fill and geosynthetic or polymeric coated steel woven wire mesh reinforcement in the laboratory should comply with EN ISO 12957 (all parts) with respect to the position of the reinforcing element in the reinforced structure.

(2) The determination of pull-out resistance of geosynthetic or polymeric coated steel woven wire mesh reinforcement from soil in the laboratory shall comply with EN 13738.

9.9.2 Connection strength

(1) The determination of the tensile strength at connections between reinforcing elements and facing elements shall be tested with appropriate standards, considering the type of connection

9.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to reinforced fill structures.

10 Ground reinforcing elements

10.1 Scope and field of application

(1) This clause shall apply to ground reinforcing elements that provide resistance to prevent a limit state of the geotechnical structure being exceeded.

NOTE 1 Ground reinforcing element include rock bolts; rock anchors; soil nails; sprayed concrete; wire mesh, and facing elements.

NOTE 2 See Clause 8 for anchors that retain a structure fixed into soil or rock..

NOTE 3 Other stand-alone nets and safety nets than wire meshes, snow fences or avalanche protections are not covered by this clause.

NOTE 4 Reinforcing elements in underground openings are not covered by this clause.

(2) This Clause shall apply to the verification of ultimate limit states, serviceability limit states, durability and robustness of the ground reinforcing elements themselves.

(3) In addition to prEN 1997-1:2022, Clauses 4, 5, 6, 7 and 9 of this document apply, as appropriate for the geotechnical structure being designed.

10.2 Basis of design

10.2.1 Design situations

(1) In addition to prEN 1997-1:2022, 4.2.2 design situations for ground reinforcing elements shall include but are not limited to:

- temporary or permanent nature of the reinforcing element or structure;
- method and sequence of excavation and drilling;
- location of discontinuities, weathered zones and other interfaces relevant for the design of the reinforcing element;
- chemical components of ground or groundwater that can adversely affect the durability of the reinforcing element and the resistance at the grout/ground interface;
- potential brittle failure of the reinforced structure;
- effect of corrosion.

10.2.2 Geometrical properties

(1) prEN 1997-1:2022, 4.3.3 shall apply to ground reinforcing elements.

(2) Accessibility of drilling and installation equipment shall be taken into account in determining the geometrical properties of the reinforcing element.

10.2.3 Zone of Influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to ground reinforcing elements.

10.2.4 Actions and environmental influences

10.2.4.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to ground reinforcing elements.

10.2.4.2 Permanent and variable actions

(1) The design resistance of reinforcing elements shall be sufficient to prevent the following limit states being exceeded by the reinforced structure:

- failure by overall or local stability determined in accordance with Clause 4;
- failure by loss of bearing resistance determined in accordance with Clause5;
- failure by sliding determined in accordance with Clause 5;
- failure by loss of equilibrium determined in accordance with Clause7.

10.2.4.3 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to ground reinforcing elements.

10.2.4.4 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.5 shall apply to ground reinforcing elements.

(2) Chemical components of ground or groundwater that can adversely affect the durability of the reinforcing element or the resistance at the ground/grout interface shall be accounted for in the verification of durability.

10.2.5 Limit states

(1) In addition to prEN 1997-1:2022, 8, the following ultimate limit states shall be verified:

- rupture of the reinforcing element;
- failure at the interface between the ground and the reinforcing element (pull-out);
- rupture of the connection between reinforcing elements or to facing;
- failure by loss of bearing-resistance in the ground below reinforcing element (punching)
- loss of force or resistance by displacement of the resisting element due to creep;
- loss of force or resistance by deformations or fall-out of ground behind.

(2) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified:

- Ground movement aspect;
  - elongation of the reinforcing element;
  - bulging and deformation of any facing element;
  - deformation adversely affecting the function, comfort or appearance;
- Structural aspects;
  - deformation causing damage to structure;
  - cracking or spalling of any precast facing panels, blocks or sprayed concrete;
- Hydraulic aspect;
- Environmental effects.

(3) Potential limit states other than those given in (1) and (2) should be verified.

(4) If the ground reinforcing structure consist of multiple types of elements, the resistance of each element type and the combined reinforcing resistance shall be verified.

(5) In addition to (2) and (3), the verification of the limit states shall prevent a potential brittle failure of the reinforced structure.

10.2.6 Robustness

(1) In addition to prEN 1997-1:2022, 4.2.2, the appropriate sub-clauses on robustness in Clauses 4, 5, 6, 7 and 9 shall apply to the geotechnical structures being designed.

(2) Specification of measures to enhance robustness of a reinforced structure with rock should include;

- installation of rock bolts and rock anchors prior to blasting, to avoid creation of adversely orientated fractures, opening or enlarging existing discontinuities;
- installation of rock bolts and rock anchors before excavation, if anticipated adversely orientated discontinuities cannot be foreseen by any means before excavation.

(3) A progressive failure of the structure due to the collapse of a single reinforcement element shall be prevented.

10.2.7 Ground investigation

(1) prEN 1997-2:2022, 5 shall apply to ground reinforcing elements.

(2) The ground investigation should determine potential obstacles for the execution and performance of the ground reinforcement element during the design service life, including, but not limited to:

- obstruction to drilling;
- the drillability of the ground;
- abrasivity;
- borehole stability;
- potential flow of groundwater in or out of a borehole;
- geometrical properties of discontinuities and weakness zones;
- resistance capacity or lack of it of the resisting ground;
- adhesion at interface surfaces;
- borehole axis deviations; and
- potential loss of grout from the borehole.

10.2.8 Geotechnical reliability

(1) prEN 1997-1:2022,4.1.2.3 shall apply to ground reinforcing elements.

10.3 Materials

10.3.1 Ground

(1) prEN 1997-1:2022, 5.1 and EN 1997-2 shall apply to ground reinforcing elements.

10.3.2 Steel

(1) prEN 1997-1:2022, 5.6 shall apply to ground reinforcing elements.

10.3.3 Grout

(1) prEN 1997-1:2022, 5.4 shall apply to ground reinforcing elements.

10.3.4 Cast and sprayed concrete

(1) prEN 1997-1:2022, 5.5 shall apply to ground reinforcing elements.

10.3.5 Steel fibres

(1) Steel fibres in sprayed concrete should comply with EN 14487-1.

(2) Fibres of other materials in sprayed concrete may be used.

(3) If other material than steel fibres are used, 10.3.8 shall apply.

10.3.6 Coatings

(1) For steel reinforcing elements, the hot dip galvanized coating to steel should comply with EN ISO 1461.

(2) For a zinc-aluminium alloy coated steel welded wire meshes the coating should comply with EN 10244-2.

(3) Epoxy coating should comply with EN 13438.

(4) Polymeric coated steel should comply with EN 10245 (all parts).

10.3.7 Concrete panels and other facing elements

(1) The properties of concrete facing panels should comply with prEN 1992-1-1.

(2) The properties of precast products should comply with EN 15258.

(3) The properties of concrete facing blocks should comply with EN 771-3.

(4) Facing elements made of the same material as the reinforcing elements for fill applications shall comply with the corresponding standard, defined in 9.3.

(5) Facing elements of steel, masonry, or timber shall comply with prEN 1993-1-1, prEN 1996-1-1, and EN 1995-1-1, respectively.

10.3.8 Other materials

(1) Materials other than steel, grout, concrete, steel fibres, coatings, shall only be used for reinforcing elements if they comply with a standard specified by the relevant authority or, where not specified, as agreed for a specific project by appropriate parties.

10.3.9 Durability

(1) prEN 1997-1:202, 4.1.6 shall apply to ground reinforcing elements.

NOTE 1 For steel element see EN 1993-5:2007, 6.

NOTE 2 For steel soil nails see EN 14490, instead of EN 1993-5:2007, 6.

(2) The design service life for steel reinforcing shall be achieved by using one or more of the following measures:

– use of additional steel thickness as corrosion allowance (see EN 1993-5:2007, 6.4);
– grout, mortar or concrete protection;
– grouted duct;
– protective surface coating;

- appropriate steel material (see EN 1993-5:2007, 6.1);
- greased nail head constructions.

(3) Galvanic steel corrosion of different connecting elements shall be prevented.

(4) Where the corrosion protection is provided by sacrificial thickness allowance, ground-specific loss of steel thickness (Δe) should be determined.

NOTE Values of $\Delta e/2$ for black steel elements without any corrosion protection measures for different service lives are given in EN 1993-5:2007, Tables 6.1 and 6.2.

(5) For soil nails corrosion protection provided by grout cover (with or without duct), surface coating, or use of stainless steel should comply with EN 14490.

(6) For other steel elements corrosion protection provided by grout or cement cover, surface doating or use for stainless steel may comply with EN 14490.

(7) The selection of an appropriate system of measures for durability should consider:

- the feasibility for inspection and maintenance;
- variation of corrosion along the nail/bolt due to variation in ground conditions;
- local corrosion at connections.

10.4 Groundwater

(1) prEN 1997-1:2022, 6 shall apply to ground reinforcing elements.

NOTE For groundwater control measures, see Clause 12.

10.5 Rock bolts and rock anchors

10.5.1 Geotechnical analyses

10.5.1.1 General

(1) In addition to prEN 1997-1:2022, 7, the geotechnical analyses shall address all relevant limit state verifications listed in 10.2.4.

(2) Rock bolts and rock anchors to reinforce rock mass shall be verified.

(3) Rock anchors reinforcing rock mass may be designed and verified according to Clause 8 or 10.

NOTE A rock anchor, anchors rock into deeper rock to reinforce the rock mass by enhancing shear resistance of possible slip surfaces, such as discontinuities, weakness and weathered zones, by increasing the normal loads as a result of pre-stressing.

10.5.1.2 Resistance

(1) The design should include, but is not limited to:

- type of element;
- connection to an external structure (or absence of it);
- grouting (or absence of it);
- use of an additional bearing plate (or absence of it);
- effects of corrosion and corrosion protection needs; and

– type of loading.

(2) The installation direction shall be determined in relation to the geometrical properties of the discontinuities and weathered zones and to the direction of the action forcing upon it.

(3) The length, spacing, type and diameter shall be determined by the structure's geometrical properties, rock quality and the depth of discontinuities, weakness or weathered zones causing possible failure.

(4) Rock anchors shall be prestressed.

(5) For rock bolts tension may be applied.

NOTE Tensioning avoids superficial loosening and is usually between 25 and 50 kN.

(6) In case of tensioning or pre-stressing, its influence both on the tendon elements and on the ground shall be addressed.

10.5.1.3 Resistance at the interface of the rock bolt (pullout)

(1) The minimum total length of a rock bolt shall include a sufficient length in the rock beyond potential failure surfaces.

(2) The length shall be sufficient to avoid pull-out of the interface between the bolt and the surrounding grout or rock and/or failure at the interface between the grout and the rock.

(3) The representative pull-out resistance ($R_{\text{rep,po}}$) should be determined from Formula (10.1)

$$R_{\text{rep,po}} = P \cdot \tau_{\text{po}} \cdot L_{po} \tag{10.1}$$

where:

P is the representative perimeter of the interface area, either drilled hole or the rock bolt/anchor;

τ_{po} is the representative interface shear resistance against pull-out along the bolt-grout, bolt-rock or grout-rock interface;

L_{po} is the representative length of the element beyond potential failure surfaces, where pull-out stresses are mobilised.

10.5.2 Ultimate limit states

10.5.2.1 Verification by partial factor method

(1) Partial factors for the verification of rock bolts and rock anchors at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach.

(2) The design tensile resistance ($R_{\text{td,el}}$) of steel shall comply with prEN 1993-1-1:2022, 8.

(3) The design shear resistance ($R_{\text{sd,el}}$) of steel shall comply with prEN 1993-1-1:2022, 8.

(4) For rock bolts loaded in tension and shear the angle between loading action direction and the angle of rock bolt installation shall be considered.

(5) The design pull-out resistance ($R_{\text{d,po}}$) shall be determined from Formula (10.2).

$$R_{\mathrm{d,po}} = \frac{R_{\mathrm{rep,po}}}{\gamma_{\mathrm{R,po}}} \tag{10.2}$$

where:

$R_{\mathrm{rep,po}}$ is the representative pull-out resistance;

$\gamma_{\mathrm{R,po}}$ is a partial factor, given in Table 10.1.

NOTE Values of the partial factors are given in 10.1 (NDP) for persistent and transient design situations, unless the National Annex gives different values.

Table 10.1 — (NDP) Partial factors for the verification of resistance of rock bolts for persistent and transient design situations

Verification of	Partial factor on	Symbol	Resistance Factor Approach (RFA)
Structural resistance of reinforcing element and any connections.	Steel		See prEN 1993-1
Geotechnical resistance, mobilised at the interface between rock bolt, grout and/or rock.	Pullout	$\gamma_{\mathrm{R,po}}$	1,5

10.5.2.2 Verification by prescriptive rules

(1) prEN 1997-1:2022, 4.5 shall apply to rock bolts and rock anchors.

(2) Prescriptive rules may be used to verify rock bolts for transient design situations and for structures belonging to GC1 and GC2, provided there is comparable experience with the rock bolt type and ground conditions.

(3) If prescriptive rules are used for verification, the inspection plan shall include quality measures to ensure that the installed bolts fulfil the limitations specified for the prescriptive rule.

10.5.2.3 Verification by testing

(1) prEN 1997-1:2022, 4.6 shall apply to rock bolts and rock anchors.

NOTE For testing see 10.5.5.

10.5.2.4 Verification by Observational Method

(1) prEN 1997-1:2022, 4.7 shall apply to rock bolts and rock anchors.

(2) The rock bolt and rock anchor spacing, length and diameter shall be determined by the rock quality or weakness or weathered zone causing potential failure.

10.5.3 Serviceability limit state

(1) prEN 1997-1:2022, 9 shall apply to rock bolts and rock anchors.

10.5.4 Implementation of design

(1) prEN 1997-1:2022, Clause 10 shall apply to rock bolts and rock anchors.

(2) Ground conditions shall be inspected at site by geotechnical mapping.

(3) Grouted rock bolts without bearing plates shall be grouted over their full length of the rock bolt.

(4) Grouted rock bolts should be installed in groundwater-controlled rock conditions.

(5) If groundwater-controlled rock conditions cannot be achieved, additional measures should be used.

10.5.5 Testing

(1) prEN 1997-1:2022, Clause 11 shall apply to rock bolts and rock anchors.

(2) Acceptance tests, investigation tests and visual inspection of grouting shall be used to confirm an adequate installation and to control the quality of the grout.

(3) The required number of acceptance test shall de defined depending on the type, size, Geotechnical Category, and condition of the structure to be supported.

NOTE 1 The minimum number of investigation test, acceptance tests and visual inspection is given in Table 10.2 (NDP), unless the national annex give different values.

NOTE 2 Investigation tests are considered as sacrificial bolt/anchor test.

NOTE 3 An investigation test is e.g. core drilling of the grouted bolt on its full length.

NOTE 4 Acceptance tests are considered as production tests.

Table 10.2 — (NDP) Minimum number of investigation and acceptance tests, and visual inspection of grouting for rock bolts and rock anchors.

Geotechnical Category	Investigation tests	Investigation tests	Visual inspection of grouting
GC2	minimum of 3	minimum 1 %, with a minimum of 3.	Minimum 75 % of the grouted bolts/anchors.
GC3	minimum of 5	minimum 2 %, with a minimum of 5.	Minimum 100 % of the grouted bolts/anchors.

(4) Acceptance test should be performed on the installed elements included in the final structure.

(5) Rock bolts subjected to investigations tests shall be replaced by new bolts.

(6) For acceptance test on the grout should comply with EN 12390-2.

(7) Non-destructive in situ testing, such as acoustic or ultrasonic testing, should be used to confirm an adequate installation of the rock bolt and to control the quality of the grout.

NOTE The tests are e.g. boltometer tests and RBT (Rock Bolt Tester) tests.

(8) In alternative to (6) non-destructive in situ testing may be used, if specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(9) The acceptance criterion for grouted rock bolts shall be the verification of 10.5.4 (3).

(10) If other acceptance criteria are used, these should be established by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

10.6 Soil Nails

10.6.1 Geotechnical analyses

10.6.1.1 General

(1) prEN 1997-1:2022, 7 shall apply to soil nails.

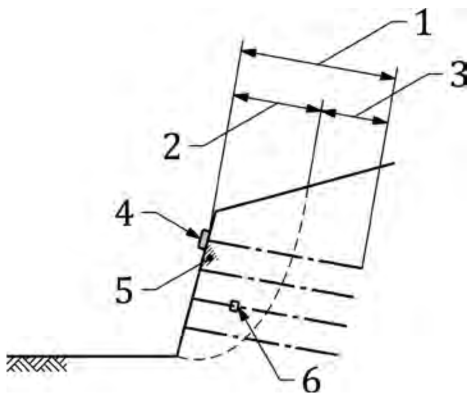
(2) In addition to prEN 1997-1:2022, 7, the geotechnical analysis shall address all limit state verifications listed in 10.2.4.

(3) Horizontal and vertical displacement of a structure reinforced with soil nail should be analysed according to 4, 5, or 7.

10.6.1.2 Resistance at the interface of the soil nail (pull-out)

(1) The resistance of a soil nail due to pull-out from the ground shall be verified for both the part of the soil nail in front and behind the potential critical failure surface.

NOTE Figure 10.1 gives an illustration of a soil nailed wall/cutting.



Key

1 Total length

2 Active zone

3 Passive zone

4 Connection to facing

5 Stability between nails and facing

6 Long nails may have joints and couplings

Figure 10.1 — Example of a wall/cutting reinforced with soil nails.

(2) The representative pull-out resistance ($R_{\mathrm{rep,po}}$) of a soil nail shall be determined from Formula (10.3)

$$R_{\mathrm{rep,po}} = P \cdot \tau_{\mathrm{po}} \cdot L_{po} \tag{10.3}$$

where:

P is the representative perimeter of the failure surface enclosing the soil nail per unit length, where pull-out resistance is mobilised;

τ_{po} is the representative interface shear resistance against pull-out along the ground-soil nail interface;

L_{po} is the total length of the soil nail in the zone, where pull-out resistance are mobilised.

NOTE Pull-out resistance can be influenced by dynamic actions.

(3) For cases with large variations along the soil nail, of either the normal stress acting on the soil nail or the ground conditions, Formula (10.3) should be replaced with an integral of the shear resistance over the considered length.

(4) Representative value of pull-out resistance between core and grouted body shall be determined according to prEN 1992 (all parts).

NOTE The failure between core and grouted body can be neglected for soil nails that has been enhanced and verified to avoid this failure mode.

(5) The perimeter of the soil nail, *P*, should be determined as a nominal value with consideration of nail type and ground properties.

NOTE For soil nails that is not circular e.g. L-shape or grouted soil nails, the perimeter is estimated based on assumed shape of the failure surface enclosing the soil nail.

(6) The perimeter of a grouted soil nail may be determined as a nominal value of the perimeter of the drilled hole for installation.

(7) Comparable experience shall be used to determine the representative value of the interface shear resistance, τ_{po}, with consideration of reinforcing type, installation method and ground conditions.

(8) The interface shear resistance shall be confirmed by project-specific investigation tests, before or during execution, see 10.6.5.1.

NOTE Investigation test is used to confirm the ultimate interface friction in the passive zone, active zone or the entire length of the nail.

(9) As alternative to (8), for GC1 and GC2, prescriptive rules regarding values of interface shear resistance for different ground conditions and soil nail types may be specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(10) An adequate installation and satisfactory performance of the production soil nails at the proof load shall be demonstrated by acceptance tests, see 10.6.5.1.

(11) The representative pull-out resistance from the active zone (Figure 10.1) may be increased by any resistance at the connection to the facing determined according to Formula (10.7).

10.6.2 Ultimate limit state

10.6.2.1 General

(1) The design value of the ultimate limit state resistance of a soil nail ($R_{\mathrm{d,SN}}$) shall along its entire length satisfy Formula (10.4) and Formula (10.5)

$$E_{\mathrm{d}} \leq R_{\mathrm{d,SN}} \tag{10.4}$$

$$R_{\mathrm{d,SN}} = \min(R_{d,po}\,, R_{d,el}, R_{d,con}) \tag{10.5}$$

where:

E_{d} is the maximum value of the design value of the effects of actions (see 10.2.4.2);

$R_{\mathrm{d,po}}$ is the design value of a soil nails interface resistance (pull-out);

$R_{\mathrm{d,el}}$ is the design value of the resulting resistance of the core of the soil nail and any joints/couplings that is part of it;

$R_{\mathrm{td,con}}$ is the design value of the resulting resistance of the joints/couplings of different sections/parts of one soil nail or the connection to the facing.

10.6.2.2 Verification by partial factor method

10.6.2.2.1 General

(1) Partial factors for the verification of soil nails at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach

10.6.2.2.2 Failure at the interface between the ground and the soil nail (pull-out)

(1) The design pull-out resistance ($R_{\mathrm{d,po}}$) of a soil nail shall be determined from Formula (10.6).

$$R_{\mathrm{d,po}} = \frac{R_{\mathrm{rep,po}}}{\gamma_{\mathrm{R,po}}} \tag{10.6}$$

where:

$R_{\mathrm{rep,po}}$ is the representative pull-out resistance of the reinforcing element;

$\gamma_{\mathrm{R,po}}$ is a partial factor, given in Table 10.3(NDP)

(2) The representative pull-out resistance shall be determined from investigation tests or by comparable experience.

NOTE The criteria to determine the pull-out resistance are given in 10.6.1.2.

(3) The design pull-out resistance shall be verified by acceptance tests according to 10.6.5.2.

NOTE The minimum number of investigation and acceptance test is given in Table 10.2 (NDP), unless the national annex gives different values.

Table 10.2 — (NDP) Minimum number of investigation and acceptance tests for soil nails

Geotechnical Category	Investigation tests	Acceptance tests
GC2	Minimum 1 test per distinct geotechnical unit, with a total of minimum 3 test per site.	Minimum 2 % of the production nails, with a minimum of 3 nails.
GC3	Minimum 2 test per distinct geotechnical unit, with a total of minimum 5 test per site.	Minimum 3 % of the production nails, with a minimum of 5 nails.

10.6.2.2.3 Rupture of the soil nail (tensile and shear)

(1) The design tensile resistance ($R_{td,el}$) of steel soil nails shall comply with prEN 1993-1-1:2022, 8, considering any anticipated loss of strength with time.

(2) The design shear resistance ($R_{sd,el}$) of steel soil nails shall comply with prEN 1993-1-1:2022, 8, considering any anticipated loss of strength with time.

(3) If it can be proven, with comparable experience, that the contribution from the shear resistance of the nail to the total resistance of the soil nail is significant, the shear resistance may be added as contribution.

(4) Where the corrosion protection is provided by sacrificial thickness allowance, the reduced cross-sectional area shall be determined from 10.6.5.2.

(5) When the design includes shear and bending effects of the soil nail, the structural resulting resistance shall be determined according to the prEN 1993-1-1:2022, 8.2.10 for combined axial, shear, and bending actions.

10.6.2.2.4 Tensile resistance of connections, joints and couplings

(1) The design tensile resistance of a connection, joint or coupling ($R_{d,con}$) shall be verified for the same design load as the soil nail itself.

(2) For steel soil nails, $R_{d,con}$ shall comply with prEN 1993-1-1:2022, 8.

10.6.2.2.5 Partial factor

(1) The ultimate geotechnical resistance of a reinforcing element should be verified using a factor $\gamma_{R,po}$ on resistance according to Formula (10.6).

NOTE Values of the partial factors are given in Table 10.3(NDP) for persistent and transient design situations unless the National Annex gives different values.

Table 10.3 — (NDP) Partial factors for the verification of resistance of soil nails for persistent and transient design situations

Verification of	Partial factor on	Symbol	Resistance Factor Approach (RFA)
Structural resistance of reinforcing element and any connections.	Steel		See EN 1993-1-1
Geotechnical resistance, mobilised at the interface between soil nail and ground	Pull-out	$\gamma_{R,po}$	1,5

10.6.2.3 Verification by prescriptive rules

(1) Prescriptive rules may be used to verify soil nails for transient design situations, provided there is comparable experience with the soil nail type in the specific ground conditions.

(2) If prescriptive rules are used for verification, the Inspection plan shall include quality measures to ensure that the installed soil nails fulfil the limitations specified for the prescriptive rules.

(3) If the inspection in (2) gives that the soil nail is not complying with the limitations specified, testing according to 10.6.5 shall be performed to confirm the design.

10.6.3 Serviceability limit state

(1) prEN 1997-1:2022, 9 shall apply to soil nails.

10.6.4 Implementation of design

(1) In addition to prEN 1997-1:2022, 10, EN 14490 shall apply to soil nails.

10.6.5 Testing

10.6.5.1 General

(1) prEN 1997-1:2022,11 shall apply to soil nails.

10.6.5.2 Pull-out resistance

(1) Testing of soil nails should comply with EN 14490:2010, Annex C.

NOTE 1 Investigation tests are in EN 14490 referred to as sacrificial nail test.

NOTE 2 Acceptance tests are in EN 14490 referred to as production nail test.

NOTE 3 Limiting values for acceptance criteria in investigation and acceptance tests are given in Table 10.4(NDP), unless the National Annex gives different values.

Table 10.4 — (NDP) Acceptance criterion for investigation and acceptance test of Soil nails.

Acceptance criteria	Investigation test	Acceptance test
Creep rate [a] at maximum proof load, P_p	2 mm	2 mm
Maximum measured extension of the head of the test nail at the proof load, P_p	< the elastic extension of L_{db}[b]	< the elastic extension of L_{db}

[a] The creep rate is defined as $(s_2\text{-}s_1)/\log(t_2/t_1)$, where s_1 and s_2 are the measured nail displacement at time 1 and time 2 respectively. [time 2 > time 1]

[b] L_{db} is the debonded length of the test nail, or if no specific part is debonded the elastic extension calculated as the theoretical extension of any debonded length of the test nail.

(2) The proof load for acceptance tests, P_p shall be equal to the design value of the effect of actions E_d (see Formula (10.4)).

(3) The design pull-out resistance has been verified with the acceptance test when the specified creep rate in Table 10.4 (NDP) is not exceeded at the value of P_p.

(4) For investigation tests the target proof load, $P_{p,}$ should be estimated from the expected representative pull-out resistance (see Formula (10.3.))

(5) The representative pull-out resistance is determined as maximum test load in the investigation test, where the creep rate does not exceed the acceptance criterion.

NOTE Values of the acceptance criterion for different tests are given in Table 10.4 (NDP).

(6) The acceptance criteria of the creep rate may be adjusted to a smaller value in the design.

(7) The test nails should be evenly distributed throughout the structure.

(8) Investigation test should be performed for the part of the soil nail, which has to provide the design pull-out resistance.

(9) Acceptance test may be performed on the production nails full length, without debonding a specific test part of the nail.

10.6.5.3 Face stability test

(1) If the execution involves excavation, the face stability should be tested in accordance with EN 14490.

(2) If the stability of the face can be verified by comparable experience, the face stability test may be omitted.

10.7 Wire mesh

10.7.1 Geotechnical analyses

10.7.1.1 General

(1) Wire mesh solutions may be used, to support loosened rock, spalling rock or rock blocks.

(2) Wire mesh solutions may be used to support soil or fill in combination with geotextile or other additional layers.

(3) Wire mesh solutions may be reinforced with steel ropes, if the mesh otherwise exceeds a limit state.

10.7.1.2 Rupture of wires

(1) Wire mesh shall be designed to be connected to the ground appropriately, that its connection element extends into firm ground beyond any discontinuity or weathered zone.

(2) The capacity of the wires, ropes and connection of the wires in the wire mesh shall be verified.

(3) The allowance of any small rock piece or crumb to fall through the mesh opening shall be defined to dimension the type of mesh or meshes and size of mesh opening.

10.7.1.3 Rupture of connections

(1) The design resistance of a connection ($R_{d,con}$) shall be verified for at least the same as the design resistance of the wire mesh itself.

(2) If the wire mesh is connected to bolts or nails, the connection resistance of the wire mesh to the bearing plates shall be verified.

(3) If the wire mesh is connected to bolts or nails, the size of bearing plates shall be appropriately sized with respect to the size of the mesh opening.

(4) If the wire mesh is connected to or embedded in sprayed concrete, the wire mesh verification shall comply with the verification of the sprayed concrete.

10.7.2 Ultimate limit state

10.7.2.1 General

(1) prEN 1997-1:2022, 4.2 shall apply to wire mesh.

(2) The characteristic tensile strength of polymeric coated steel woven wire mesh reinforcing should be determined in accordance with EN ISO 10319.

10.7.2.2 Verification by partial factor method

(1) Partial factors for the verification of wire mesh at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach.

(2) The design tensile resistance ($R_{td,el}$) of steel of the wires shall comply with prEN 1993-1-1:2022, 8.

(3) The design connection resistance ($R_{d,con}$) of a wire mesh shall be determined from Formula (10.7)

$$R_{d,con} = \frac{R_{rep,con}}{\gamma_{R,con}} \tag{10.7}$$

where:

$R_{rep,con}$ is the representative connection resistance of the wire mesh to its connection element;

$\gamma_{R,con}$ is a partial factor, given in Table 10.5 (NDP).

NOTE Values of the partial factors are given in Table 10.5 (NDP) for persistent and transient design situations unless the National Annex gives a different value.

Table 10.5 — (NDP) Partial factors for the verification of resistance of wire meshes for persistent and transient design situations

Verification of	Partial factor on	Symbol	Value of partial factors
Structural resistance of steel wires.	Steel		See prEN 1993-1-1
Geotechnical resistance Connection wire mesh and its connection element.	Connection	$\gamma_{R,con}$	1,5

10.7.2.3 Verification by prescriptive rules

(1) prEN 1997-1:2022, 4.5 shall apply to wire mesh.

(2) Prescriptive measures may be used to verify wire mesh for transient design situations and for structures belonging to GC1 and GC2, provided there is comparable experience with the wire mesh interaction with the ground conditions.

(3) If prescriptive rules are used for verification, the Inspection plan shall include quality measures to ensure that the installed wire meshes fulfil the limitations specified for the prescriptive rule.

10.7.2.4 Verification by testing

(1) NOTE See prEN 1997-1:2022, 4.6 shall apply to wire mesh.

(2) When wire mesh is to be verified by testing also its connection should be tested.

10.7.2.5 Verification by Observational Method

(1) prEN 1997-1:2022, 4.7 shall apply to wire mesh.

(2) The extent and locations of the wire meshes to be installed in relation to the observed conditions at site should be part of the verification by the Observational Method.

10.7.3 Serviceability limit state

(1) If project specific serviceability criterion is specified, the limit states of deformation and excessive deformation should be verified.

10.7.4 Implementation of design during

(1) prEN 1997-1:2022, 10 shall apply to wire mesh.

(2) Ground conditions shall be inspected at site by geotechnical mapping.

(3) For structures belonging to GC2 or GC3 loosened rock hanging on to the wire mesh should be checked.

(4) If the wire mesh is connected to bolts or nails, the bearing plates shall be visually inspected to see if they are fully connected to the mesh and ground surface.

(5) If the wire mesh is not fully connected, further inspection, assessment and measures shall be designed and implemented.

(6) If the wire mesh is embedded in sprayed concrete, it shall be checked that the wire mesh is fully covered by sprayed concrete on both sides of the mesh.

10.7.5 Testing

(1) Testing shall comply with prEN 1997-1:2022, 11, and with the appropriate sub-clause in this standard for the involved geotechnical structure.

10.8 Sprayed concrete

10.8.1 Geotechnical analyses

(1) The thickness, the resistance class and the reinforcement of the sprayed concrete shall be defined by the demand of bearing capacity to resist the loads of soil or rock blocks, grade of jointed rock mass, weathered zones and weakness of the rock mass to prevent outfall of ground.

NOTE Normally rock blocks are bolted before spraying with concrete and they will cover most of the support of the rock bolts.

(2) For reinforced fill structures and structures reinforced with soil nails the sprayed concrete shall be designed to resist the earth pressure from the ground according to Clause 7.

(3) The minimum thickness should consider the execution restrictions.

(4) The minimum thickness should be defined taking into account the adverse effect of geometric tolerances and variation in the surface unevenness.

NOTE Thicknesses of 30 mm or greater are recommended.

10.8.2 Ultimate limit state

10.8.2.1 General

(1) prEN 1997-1:2022, 4.2 shall apply to sprayed concrete.

(2) For the specifications and conformity of sprayed concrete EN 14487-1 should apply.

10.8.2.2 Verification by partial factor method

(1) Partial factors for the verification of sprayed concrete at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using the Resistance Factor Approach.

(2) For sprayed concrete reinforcing verification prEN 1992 (all parts) shall apply.

10.8.2.3 Verification by prescriptive rules

(1) prEN 1997-1:2022, 4.5 shall apply to sprayed concrete.

(2) If prescriptive rules are used for verification, the Inspection plan shall include quality measures to ensure that the installed sprayed concrete fulfil the limitations specified for the prescriptive rule.

10.8.2.4 Verification by testing

(1) prEN 1997-1:2022, 4.6 shall apply to sprayed concrete.

NOTE For testing during execution see 10.8.5.

10.8.2.5 Verification by Observational Method

(1) prEN 1997-1:2022, 4.7 shall apply to sprayed concrete.

(2) The extent and thickness of the sprayed concrete to be installed in relation to the observed conditions at site should part of the verification by the Observational Method.

10.8.3 Serviceability limit state

(1) prEN 1997-1:2022, 9 shall apply to sprayed concrete.

10.8.4 Implementation of design

(1) prEN 1997-1:2022, 10 shall apply to sprayed concrete.

(2) Ground conditions shall be inspected at site by geotechnical mapping.

(3) The ground surface should be verified for preparation / proper cleaning to achieve adhesion bondage between ground and sprayed concrete.

(4) Sprayed concrete should be specified to be installed in dry or controlled water conditions to avoid reduction of adhesion.

(5) Water leakages should be checked to be within specified limits before execution of sprayed concrete.

(6) Preparation of the ground surface, according (2), (3) and (4) may be omitted, if transient design situations demand for immediate spraying of concrete.

(7) For water leakage areas groundwater control should be considered according to 12.

10.8.5 Testing

(1) prEN 1997-1:2022, 11 shall apply to sprayed concrete.

(2) EN 14487 (all parts) and EN 14488 (all parts) should apply.

(3) Sprayed concrete shall be tested to verify its energy absorption capacity in accordance with EN 14488-5.

(4) The sprayed concrete shall be tested on its adhesion/bond strength to the ground surface in accordance with EN 14488-4.

(5) Nominal sprayed concrete thicknesses shall be verified.

(6) Thicknesses may be verified by surface scanning before and after constructing or by measuring it in small, drilled holes through the sprayed concrete.

10.9 Facing element

10.9.1 Geotechnical analyses

(1) In addition to prEN 1997-1:2022, 7. The geotechnical analysis shall address all limit state verifications listed in 10.2.4.

(2) Horizontal and vertical deformations of a structure reinforced with facing elements shall be analysed according to Clauses 4, 5, 7 or 9, as appropriate.

10.9.2 Ultimate limit state

(1) The structural resistance of geosynthetic facing elements shall comply with 9.6.

(2) The structural resistance of facing elements of concrete, steel, masonry, and timber shall comply with prEN 1992-1-1, prEN 1993-1-1, prEN 1996-1-1 and prEN 1995-1-1, respectively.

(3) The design strength of facing elements may be determined by testing.

NOTE Guidance about design assisted by testing is given in prEN 1990:2021, Annex D.

(4) The bending and shear resistance to bulging between facing elements shall be verified to prevent bulging of the facing between reinforcement / facing connections.

(5) The shear resistance between facing elements and reinforcement when the connection relies purely on friction shall be verified.

(6) The stability against toppling of the facing elements not connected to ground reinforcements above the top layer of reinforcement shall be verified.

(7) The punching resistance of the facing shall be verified.

(8) The flexural resistance and reinforcement detailing of concrete, steel, and other hard facings shall be verified.

(9) The durability of the facing material itself and all connections for the design service life shall be verified.

NOTE 1 The connection strength of mechanical connections between facing elements and reinforcing elements, and/or between consecutive facing elements depends on the type and material of the connection and on the tensile load distribution along the reinforcing element.

NOTE 2 The stability of a frictional connection between facing elements and reinforcing element and/or between consecutive facing elements depends on the shear resistance between facing elements and reinforcements and between consecutive facing elements.

10.9.3 Serviceability limit state

(1) The bulging of segmental block and flexible facing systems shall be limited to ensure compliance with the specification.

(2) The deformations of the structure face shall be limited to avoid spalling and cracking of facing panels, blocks or sprayed concrete.

(3) Bulging at the toe of a reinforced veneer system shall be limited to values given in the specification.

10.9.4 Implementation of design

(1) prEN 1997-1:2022, 10 shall apply to facing elements.

(2) Ground conditions shall be inspected at site by geotechnical mapping.

10.9.5 Testing

(1) Execution shall comply with prEN 1997-1:2022, Clause 11, and with the appropriate sub-clause in this standard for the involved geotechnical structure.

10.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to facing elements.

11 Ground improvement

11.1 Scope and field of application

(1) This Clause shall apply to ground improvement for the following geotechnical structures:

- slopes, cuttings, and embankments (see 4);
- spread foundations (see 5);
- retaining structures (see 7).

(2) Ground improvement design shall be classified according to Table 11.1:

- diffused ground improvement (classes AI and AII); or
- discrete ground improvement (classes BI and BII).

NOTE 1 Examples of ground improvement techniques for these two families are given in Annex G.

NOTE 2 Groundwater control techniques are addressed in Clause 12.

Table 11.1 — Classification of ground improvement

Class	Family	
	A – Diffused	**B – Discrete**
I	**AI – Diffused with no unconfined compressive strength** The improved ground has an increased shear strength higher than that of the original ground. The improved ground can be modelled as a ground with improved properties.	**BI – Discrete with non-rigid inclusions** Inclusions, installed in the ground, with higher shear capacity and stiffness compared to the surrounding ground. The unconfined compressive strength of the inclusion is not measurable.
II	**AII – Ground improvement zone with unconfined compressive strength** The improved ground is modified from its original natural state, has a measurable unconfined compressive strength and is significantly stiffer than the surrounding ground. Usually, it comprises a composite of a binder and ground.	**BII – Discrete with rigid inclusions** Rigid inclusions, installed in the ground, with unconfined compressive strength significantly stiffer than the surrounding ground. The inclusions can be an engineered material such as timber, concrete/grout or steel or a composite of a binder and ground.

(3) For techniques belonging to class BII, the following conditions should be satisfied:

- structural loads are transferred through a load transfer platform into the ground directly to the rigid inclusions;
- no structural connection with the foundation is existing (presence of a load transfer platform or, in absence of load transfer platform, only contact between the improved ground and the foundation).

(4) In the absence of a load transfer platform, additional verifications may be considered during the design and the execution according to the design situations.

NOTE In this context, examples of important issues are; stress concentrations at the top of the inclusions and internal forces into the spread foundation or the raft.

(5) If the ultimate resistance of the initial ground supporting the structure is not sufficient and in absence of a load transfer platform, a single element of class BII used to transfer the structural loads to the ground shall be designed as a pile (see 6).

11.2 Basis of design

11.2.1 Design situations

(1) prEN 1997-1:2022, 4.2.2 shall apply to ground improvement.

(2) For ground improvement subject to alteration over time, design of temporary works shall specify the maximum design service life or specify any extensions to the period of temporary use.

NOTE Some forms of ground improvement might not have sufficient design service life for a temporary use which could be extended. An example would be the use of some chemical grouts which deteriorate relatively quickly.

11.2.2 Geometrical properties

(1) prEN 1997-1:2022, 4.3.3 and prEN 1990:2021, 6.3 and 8.3.7 shall apply to ground improvement.

(2) Geometric tolerances shall not less than those specified in the execution standards specified in 11.8.

(3) In addition to prEN 1990:2021 6.3 and 8.3.7, and pr1997-1:2022, 4.3.3, minimum deviation Δa of geometrical properties shall be considered in ground improvement design.

NOTE Values of Δa are given in Table 11.2 (NDP) unless the National Annex gives different values.

Table 11.2 — (NDP) Minimum deviation of geometrical properties used in ground improvement design

Geometrical property	Value of Δa		
	No measurement and no comparable experience is available	Comparable experience is available	Property is determined by direct or indirect measurements
Soil mix/bored/vibrated inclusion diameter	5 % of a_{nom}	To be defined according to comparable experience	To be defined according to measurements
Individual et Grout inclusion diameter	max (20 % of a_{nom}; 0.2 m)	max (10 % of a_{nom}; 0.1 m)	max (5 % of a_{nom}; 0.05 m)
Compaction Grout inclusion diameter	max (20 % of a_{nom}; 0.2 m)	max (10 % of a_{nom}; 0.1 m)	To be defined according to measurements
Stone or sand inclusion diameter	10 % of a_{nom}	5 % of a_{nom}	To be defined according to measurements
Driven or vibrated steel/wood or concrete inclusion diameter	The value of Δa is specified by the relevant standard, or by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.		
Inclusion/installation location (setting out, depth range, or depth)	The value of Δa is specified by the relevant standard, or by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.		
Deviation with depth	The value of Δa is specified by the relevant standard, or by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.		

11.2.3 Zone of influence

(1) prEN 1997-1:2022, 4.1.2.1 shall apply to ground improvement.

11.2.4 Actions and environmental influences

11.2.4.1 General

(1) prEN 1997-1:2022, 4.3.1 shall apply to ground improvement

(2) In addition to (1) relevant clauses of prEN 1997-3:2022 shall apply to ground improvement.

(3) The ground improvement method should be selected considering the following:

- the design situation and load variation;
  - thickness and properties of the ground or fill material;
  - water pressure in the various strata;
  - nature, size and position of the structure to be supported by the ground;
  - prevention of damage to adjacent structures or services during execution;
  - whether the ground improvement is temporary or permanent;
- in terms of anticipated deformations, the relationship between the ground improvement method and the construction sequence;
- the effects on the environment including pollution by deleterious substances or changes in groundwater level;
- the durability of the improved ground;
- any long term deterioration of the ground.

11.2.4.2 Cyclic and dynamic actions

(1) prEN 1997-1:2022, 4.3.1.3 shall apply to ground improvement.

(2) prEN 1997-3:2022, 6.2.3.3 shall apply to rigid inclusion.

11.2.4.3 Actions due to ground displacement

(1) The adverse effects of vertical and horizontal ground movement on ground improvement inclusions shall be considered.

(2) A sensitivity analysis should be carried out to determine for each design situation whether the upper or lower representative improved ground property is the less favourable.

11.2.4.4 Downdrag

(1) For Class II ground improvement, downdrag shall be considered at the perimeter of the improved ground zone.

(2) The calculation of the maximum drag force shall consider the following:

- the shear resistance at the interface between the soil and the ground improvement zone;
- downward movement of the ground due to self-weight compression;
- any surface load around the ground improvement; or
- changes in groundwater levels.

(4) An upper bound to the drag force on a ground improvement zone may be determined from the weight of the surcharge or change in groundwater level causing the movement, considering any changes in groundwater pressure due to groundwater lowering, consolidation or execution.

(5) Interaction calculations should take account of the displacement of the ground improvement relative to the surrounding moving ground.

NOTE 6.5.2.2 in this document provides guidelines to assess the drag force.

11.2.4.5 Heave

(1) Where heave of the ground results in transfer of load to the ground improvement, it shall be considered as an action.

(2) If ground improvement is subject to heave that results in tensile forces or stresses, the introduction of reinforcement should be considered.

11.2.4.6 Transverse loading

(1) Transverse actions originating from ground movements, vehicles, or other sources around or above a ground improvement zone shall be included in the verification of limit states.

(2) Transverse loading of discrete ground improvement should be evaluated by considering the interaction between the ground improvement inclusion, treated as stiff or flexible beams, and the moving soil mass.

(3) If ground improvement is subject to transverse loading that results in tensile forces or stresses exceeding the material’s tensile strength, the introduction of reinforcement shall be considered.

(4) Potential extrusion of low strength fine soil around or between discrete ground improvement inclusions should be considered.

11.2.4.7 Environmental influences

(1) prEN 1997-1:2022, 4.3.1.5 shall apply to ground improvement.

11.2.5 Limit states

11.2.5.1 Ultimate limit states

(1) In addition to prEN 1997-1:2022, 8.1, ultimate limit states for ground improvement shall be as for:

– slopes, cuttings, and embankments (see Clause 4);
– spread foundations (see Clause 5);
– retaining structures (see Clause 7).

(2) In addition to prEN 1997-1:2022, 8.1, the following ultimate limit states shall be verified:

– bearing resistance failure below the ground improvement inclusion or zone;
– uplift or insufficient tensile resistance of the ground improvement;
– failure in the ground due to transverse loading of the ground improvement;
– failure of the ground improvement inclusion or zone in compression, tension, bending, buckling or shear;
– combined failure in the ground and in ground improvement inclusion or zone;
– limit states caused by changes in groundwater conditions or groundwater pressures (see 11.4).

(3) Potential ultimate limit states other than those given in (1) and (2) should be verified.

11.2.5.2 Serviceability limit states

(1) In addition to prEN 1997-1:2022, 9, serviceability limit states for ground improvement shall be as defined for:

– slopes, cuttings, and embankments (see Clause4);
– spread foundations (see Clause 5);
– retaining structures (see Clause 7).

(2) In addition to prEN 1997-1:2022, 9, the following serviceability limit states shall be verified for all ground improvement:

– ground improvement zone or inclusion settlement and differential settlements;
– heave;
– transverse movement;
– movement or distortion of the supported structure caused by ground improvement zone movement.

(3) Potential serviceability limit states other than those given in (1) and (2) should be verified.

11.2.6 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply to ground improvement.

11.2.7 Ground investigation

11.2.7.1 General

(1) prEN 1997-2:2022, 5 shall apply to ground improvement.

11.2.7.2 Minimum extent of field testing

(1) The depth and horizontal extent of the field investigation shall be sufficient to determine the ground conditions within the zone of influence of the structure according to prEN 1997-1:2022, 4.2.1.1.

(2) For all ground improvement classes, the minimum depth of in situ testing (d_{min}) below the anticipated depth of any proposed ground improvement should be determined according to Formula (11.1):

$$d_{\text{min}} = max(5\ m; 3D; B_{\text{gi}}) \tag{11.1}$$

where:

D is the base diameter (for circular ground improvement inclusions) or one-third of the perimeter (for non-circular ground improvement) of the inclusion with the largest base;

B_{gi} is the smaller plan dimension of a rectangle circumscribing the ground improvement zone, limited to the depth of the zone of influence.

(3) For inclusions founded on or in strong homogenous ground, d_{min} should be determined according to Formula (11.2):

$$d_{\text{min}} = max(2\ m; 3D) \tag{11.2}$$

(4) The minimum depth of field investigation for ground improvement by soil replacement may be determined according to Formula (11.2) taking D as the depth of replaced soil.

(5) The minimum depth of field investigation within medium strong (and stronger) rock masses may be reduced provided there is comparable experience to allow the properties of the rock mass to be predicted.

11.2.8 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to ground improvement.

(2) Ground improvement shall be classified as either Geotechnical Category GC2 or GC3.

11.3 Materials

11.3.1 Ground properties

(1) prEN 1997-2:2022, Clause 7 to 12 shall apply to ground improvement.

(2) Ground improvement parameters shall be adjusted to account for potential deterioration of the ground improvement over its design service life.

11.3.2 Improved ground properties

11.3.2.1 General

(1) The representative properties of improved ground should be initially selected based on comparable experience.

(2) The final representative values of the improved ground properties shall be verified by at least one of the following;

– field investigation; or
– laboratory testing of exhumed material incorporated within the ground improvement, or
– comparable experience; or
– calculation; or
– monitoring.

(3) Field investigation of discrete ground improvement should verify the response of the system, either by testing individual inclusions or by testing the system.

(4) When determining values of improved ground properties, the following shall be considered:

– information from relevant tests in appropriate improved ground conditions;
– the value of each improved ground property compared with local and general experience;
– variation or tolerances of improved ground properties relevant to the design;
– results of any laboratory or large-scale field trials and measurements from neighbouring constructions;
– correlations between the results from more than one type of test;
– any significant deterioration in improved ground properties that can occur during the lifetime of the structure.

11.3.2.2 Class I ground improvement

(1) The determination of the representative values of the improved ground property shall comply with prEN 1997-1:2022, 4.3.2.

(2) To avoid degradation, material used for Class BI inclusion shall be sufficiently durable and chemically inert according to the anticipated ground and groundwater conditions during execution and design service life.

(3) The specification of material for Class BI inclusions should allow it to be compacted to form a dense inclusion fully interlocked with the surrounding ground.

11.3.2.3 Class II ground improvement

(1) The unconfined compressive strength should be determined on cylindrical undisturbed samples with a height to diameter ratio of two.

(2) Where the sample dimensions differ, a correction complying with EN 12716:2018, A.1, may be applied.

(3) The stiffness of ground improvement materials should be determined either from laboratory tests on undisturbed samples, documented correlations, or by monitoring of deformation.

(4) During the design, the representative value of unconfined compressive strength, $q_{\mathrm{u,rep,imp}}$ should be determined as a nominal value of the unconfined compressive strength shall according to engineering judgement and comparable experience;

NOTE $q_{\mathrm{u,rep,imp}}$ includes the factors η_{t} and η_{c} see Formula (11.4)

(5) If more than 10 samples (≥ 10) are tested, a characteristic value of the unconfined compressive strength, $q_{\mathrm{uk,imp}}$ should be determined according to prEN 1997-1:2022, 4.3.2, using a log-normal distribution.

NOTE When assessing characteristic values, the confidence level is 90 % unless the National Annex gives a different value.

(6) Based on testing, if fewer than 10 samples are tested, the representative value of unconfined compressive strength $q_{uk,imp}$ should be determined using Formula 11.3.

$$q_{\mathrm{uk,imp}} = k_{\mathrm{field}}\, \mu_{\mathrm{norm}} \tag{11.3}$$

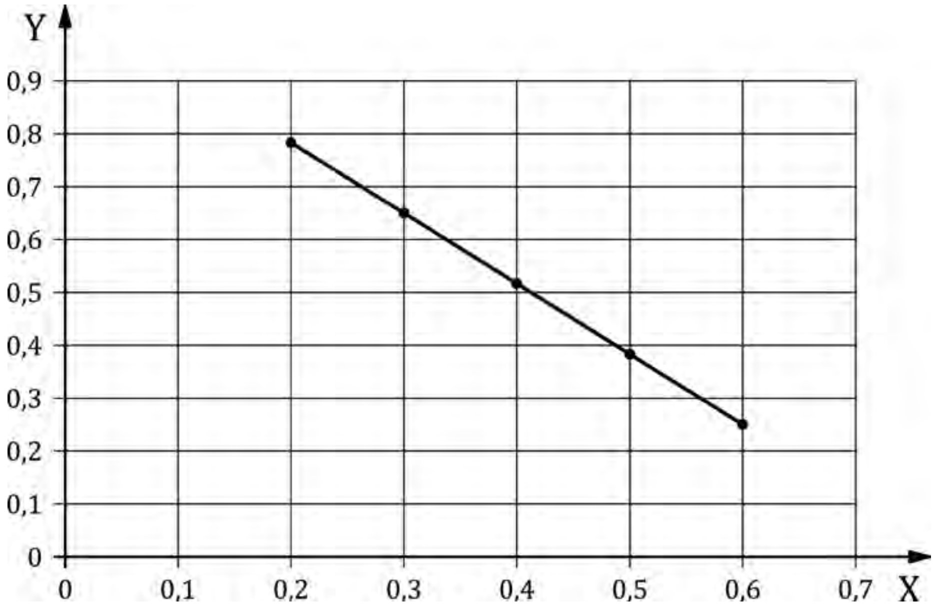
where:

μ_{norm} is the mean normal strength of field samples;

k_{field} is a factor depending on the coefficient variation (Figure 11.1).

NOTE The value of k_{field} is 0.52 unless the National Annex gives a different value.

(7) As an alternative to (6) Figure 11.1 may be used to determine the correlation coefficient k_{field} based either on measured coefficient of variation, V_{meas}, or on comparable experience.



Key

X Coefficient of variation V_{norm}

Y k_{field}

Figure 11.1 — Relationship between Coefficient of variation V_{norm} and k_{field}

(8) Other approaches may be used to assess the characteristic value of the unconfined compressive strength $q_{uk,imp}$.

NOTE These approaches can be based on the analysis of the minimal value, the mean, the standard deviation, the modes or the cumulative frequency for the measured values taking into account the different types of ground (sub-population).

(9) If undisturbed sampling is impractical, the strength may be determined by documented correlations from other in-situ tests.

(10) The selected field strength and coefficient of variation shall be documented in the Geotechnical Design Report.

(11) The design value of unconfined compressive strength (q_{ud}) of improved ground shall be determined from Formula (11.4):

$$q_{ud} = \frac{q_{u,rep,imp}}{\gamma_M} = \frac{\eta_t \cdot \eta_c \cdot q_{uk,imp}}{\gamma_M} \tag{11.4}$$

where:

$q_{u,rep,imp}$ is the representative value of the unconfined compressive strength of the improved ground;

$q_{uk,imp}$ is the characteristic value of the unconfined compressive strength of the improved ground;

γ_M is a partial material factor;

η_t is a factor accounting for the difference in time between testing (typically 28 days) and when the improved ground is exposed to the designed stresses;

η_c is a reduction factor accounting for long term effects.

NOTE 1 The value of η_c is 0.85 unless the National Annex gives a different value.

NOTE 2 The value of γ_M is given in Table 4.7 in prEN 1997-1:2022.

(12) The value of η_t should be determined directly from testing for the specific type of ground improvement.

(13) In the absence of testing and comparable experience, the value of η_t for Ordinary Portland cement-based inclusions should be determined from Formula (11.5):

$$\eta_t = 0.375 + 0.187 \ln t \leq 1.40 \tag{11.5}$$

where:

t is the time in days since the ground improvement inclusion was installed.

NOTE When t = 28 days, η_t = 1.0.

(14) The design strength of concrete, wood, and steel inclusions shall be determined in accordance with prEN 1992-1-1, EN 1995-1-1, and prEN 1993-1-1, respectively.

11.3.2.4 Weight density

(1) For diffused ground improvement in Class I, the improved or modified weight density should be estimated from empirical data, comparable experience, reduction in volume or field testing.

(2) For Class II ground improvement, especially for jet grouting and deep soil mixing, the improved or modified weight density should be determined.

(3) The weight density in (2) should be determined by considering the volume of binder being incorporated within the volume of installed inclusion, with consideration of empirical data, comparable experience, reduction in volume and/or field investigation.

NOTE 1 Density assessment can be impacted by incomplete filling of voids or bleeding within inclusions prior to set.

NOTE 2 Samples can be taken during execution to verify the design assumptions of the weight density of the improved ground.

11.4 Groundwater

(1) pEN 1997-1:2022, 6 shall apply to ground improvement.

11.5 Geotechnical analysis

11.5.1 General

(1) An analysis of the interaction between structure, ground improvement and ground should be carried out to verify that the ultimate and serviceability limit states are not exceeded.

(2) The method of analysis selected should consider the stiffness ratio of discrete inclusions to the surrounding ground.

11.5.2 Diffused ground improvement design (AI and AII classes)

(1) For Class AI and AII ground improvement techniques the resulting modified ground properties should be used in the verification of the corresponding structure in accordance with:

- slopes, cuttings, and embankments (see Clause 4);
- spread foundations (see Clause 5);
- retaining structures (see Clause 7).

NOTE 1 Design of slopes, cuttings and embankments, spread foundations and retaining structures with the use of AI and AII techniques is similar to the design of these geotechnical structures without the use of any ground improvement technique.

NOTE 2 For AI and AII techniques, the main issue is the assessment of the improved ground properties.

NOTE 3 This calculation model is applicable when the behaviour of the improved ground can be conveniently modelled by conventional ground models. In order to follow this method, the designer can evaluate the change of ground properties (i.e. cohesion, friction angle, permeability, etc.) and can consequently define the "improved representative values" for the material properties.

(2) For material with unconfined compressive strength in Class AII, ultimate limit states may be verified by demonstrating that design effects of actions do not exceed the stress envelope.

NOTE See Annex G.3 for further guidance.

11.5.3 Discrete ground improvement design (BI and BII classes)

(1) Where Class BI or BII ground improvement is used to support or retain a structure an interaction calculation model shall include:

- the evaluation of the interaction effects between the ground, discrete inclusions, and the overlying structure, embankment, or load transfer platform;
- the derivation of the neutral plane for Class BII corresponding to the point where the inclusion settlement equals the ground settlement (see Figure 11.2);
- the derivation of the distribution ratio to determine the proportion of the load applied to individual discrete inclusions;
- a verification of the structural resistance of the individual discrete inclusions;
- a verification of buckling resistance depending on slenderness and soil support parameter (see Annex C13 especially for BII techniques).

NOTE The interaction effects relevant to Class BII ground improvement are similar to those relevant for a piled raft (see Figure 11.2), whereby a load transfer platform causes additional interaction effects influencing the load distribution between rigid inclusions and supporting ground and initialising negative skin friction in the upper part of the rigid inclusions.

(2) The representative total resistance $R_{\text{sys,rep}}$ of a ground improvement system with rigid inclusions should be determined from Formula (11.6):

$$R_{\text{rep,sys}} = \sum_{i=1}^{n} R_{\text{ri,i}} + R_{\text{g}} \tag{11.6}$$

where:

$R_{ri,i}$ is the resistance of a rigid inclusion *i*, depending on its position within the group;

n is the number of rigid inclusions;

R_g is the resistance of the ground supporting the load transfer platform or the raft or single footing in the net area between the columns.

(3) Analysis of inclusions may be based on numerical modelling including nonlinear stress-strain model for the ground and the interactions between ground and inclusions.

(4) The resistance of a rigid inclusion R_{ri} shall be assessed according to Clause 6, depending on the technique used to carry out the rigid inclusion.

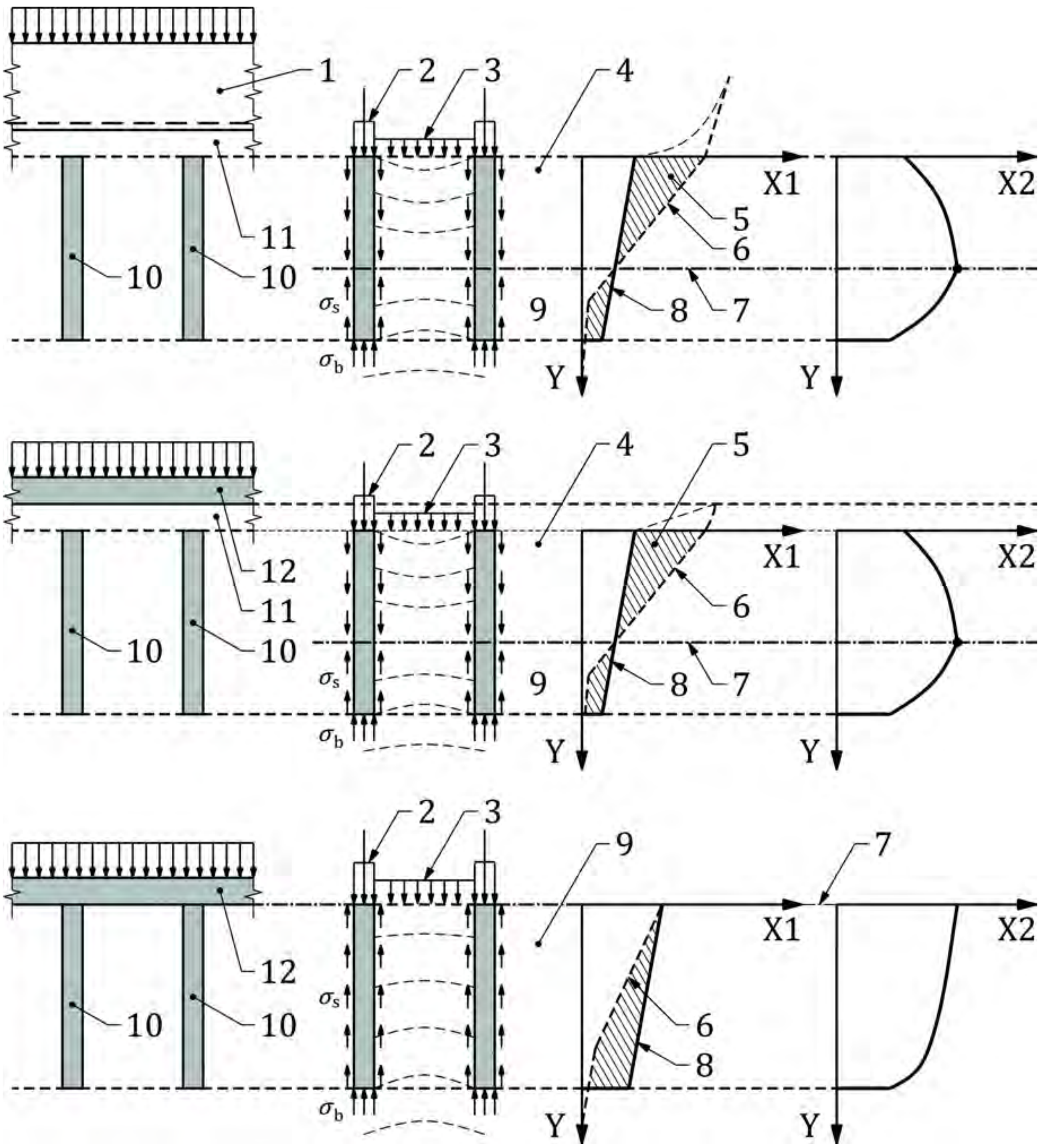
(5) Rigid inclusions may be allowed to reach the limiting value of the geotechnical resistance provided an ultimate limit state is not exceeded either in the overall system or in the structural inclusions.

NOTE The limiting value of the rigid inclusions is not the same as that of a single column, since it can include group effects and further interaction effects as shown in Figure 11.2

(6) Load transfer platforms incorporating tensile elements should be designed in accordance with Clause 9.

(7) Load transfer platforms without tensile elements should be designed in accordance with Clause 5.

(8) For embankments, when the embankment and the load transfer platform are merged, they should be verified accordingly.



Key

X1	Settlement	6	S ground
X2	Inclusion axial force	7	Neutral plane
Y	Depth	8	S inclusion
1	Embankment	9	Positive skin friction
2	σ inclusion	10	Inclusion
3	σ ground	11	Load transfer platform
4	Negative skin friction	12	Structure (e.g. raft)
5	Differential settlement		

Figure 11.2 — Interaction effects of a ground improvement with rigid inclusions

11.6 Ultimate limit states

11.6.1 General

(1) For all form of ground improvement, the following ultimate limit states shall be verified:

- overall stability;
- external stability (including sliding, bearing capacity and loss of static equilibrium if relevant);
- compound stability;
- internal stability.

(2) Methods used to verify ultimate limit states for different class and family of ground improvement and different geotechnical structures should be selected according to Table 11.3.

NOTE Table 11.3 (NDP) gives appropriate verification methods unless the National Annex gives different methods.

Table 11.3 — (NDP) Methods used to verify ultimate limit states of ground improvement

Class	Family	
	A – Diffused	**B – Discrete**
I	1. Determine improved ground properties according to 11.3 and prEN 1997-1:2022, 4.3.2 2. Verify ULS according to 11.2.4, 11.5.2 and appropriate clauses of prEN 1997-3:2022	1. Determine properties of non-rigid inclusion according to 11.3 and prEN 1997-1:2022, 4.3.2 2. Verify ULS of the system using separate ground and inclusion properties; 3. Verify ULS according to 11.2.4, 11.5.3 and appropriate clauses of prEN 1997-3:2022 4. Verify compression and shear resistance in inclusion and soil according to 11.2.3 and 11.2.4. (bulging, etc.) 5. For Geotextile Encased Inclusion, determine the strength of the reinforcing element of according to 9.6
II	1. Determine improved design ground properties according to 11.3 2. Verify ULS according to 11.2.4 with calculation methods in 11.5.2 3. Verify structural resistance	1. Determine improved design ground properties of the rigid inclusion in according to 11.3 and especially 11.3.2.3 2. Verify ULS according to 11.2.4 3. Verify structural resistance of the rigid inclusions

11.6.2 Class BI and BII ground improvement

(1) The design resistance of Class BI and BII ground improvement ($R_{\text{sys,d}}$) should be determined from Formula (11.7):

$$R_{\text{d,sys}} = \frac{R_{\text{rep,sys}}}{\gamma_{\text{R,sys}}\gamma_{\text{Rd,sys}}} \; or \left(\frac{\sum_i^n R_{\text{ri,i}}}{\gamma_{\text{Rd}}\gamma_{\text{Rc}}} + \frac{R_{\text{g}}}{\gamma_{\text{g}}}\right) \tag{11.7}$$

where:

$R_{rep,sys}$ is the representative value of the total resistance of the ground improvement system with rigid inclusions;

$\gamma_{R,sys}$ is a partial resistance factor for the rigid inclusion system, given in 11.6.3;

$\gamma_{Rd,sys}$ is a model factor.

NOTE 1 The value of $\gamma_{Rd,sys}$ is 1.0 unless the National Annex gives a different value.

NOTE 2 The values of γ_{Rd} is given in Table 6.3

NOTE 3 The value of γ_{Rc} is given in Table 6.7.

NOTE 4 The value for γ_g is taken equal to $\gamma_{R,raft}$ = 1.4, unless the National Annex gives a different value.

11.6.3 Partial factors

(1) Partial factors for the verification of structures using ground improvement with technique BI and BII at the ultimate limit state shall be determined according to prEN 1997-1:2022, 4.4.1, using either the Material Factor Approach or the Resistance Factor Approach

NOTE 1 The National Annex can specify which Factor Approach to us.

NOTE 2 Values of the partial factors for BI and BII techniques are given in Table 11.4 (NDP) for persistent, transient and accidental design situations unless the National Annex gives different values.

Table 11.4 — (NDP) Partial factors for the verification of ultimate resistance of ground improvement for fundamental (persistent and transient) and accidental design situations

Verification of	**Partial factor on**	**Symbol**	**Material factor approach (MFA), both combinations (a) and (b)**		**Resistance factor approach (RFA)**
			(a)	**(b)**	
Overall stability	See Clause 4				
Compressive resistance of diffused ground improvement (AI and AII) or discrete ground improvement (BI)	Actions and effects-of-actions[a]	γ_F and γ_E	VC1	VC3	Refer to other clauses as appropriate
	Ground properties[b,c]	γ_M	M1	M2	
Axial compressive resistance of discrete rigid inclusions	Actions and effects-of-actions[1]	γ_F and γ_E	VC4	VC3	VC1
	Ground properties	γ_M	M1	M3	Not factored
	Bearing resistance of LTP	γ_R	Not factored		Refer to Clauses 5 and 9
	Overall system resistance	$\gamma_{R,sys}$	Not used		1.4 (1.2)[d]
Transverse resistance of	Actions and effects-of-actions[a]	γ_F and γ_E	VC4 (EFA)[e]	VC3	Not used

<table>
<tr><th rowspan="2">Verification of</th><th rowspan="2">Partial factor on</th><th rowspan="2">Symbol</th><th colspan="2">Material factor approach (MFA), both combinations (a) and (b)</th><th rowspan="2">Resistance factor approach (RFA)</th></tr>
<tr><th>(a)</th><th>(b)</th></tr>
<tr><td rowspan="2">discrete and diffused ground improvement</td><td>Ground properties[b,c]</td><td>γ_{M}</td><td>M1</td><td>M2</td><td rowspan="2"></td></tr>
<tr><td>Transverse resistance</td><td>γ_{Re}</td><td colspan="2">Not factored</td></tr>
<tr><td colspan="6">[a] Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in prEN 1990.
[B] Values of the partial factors for Sets M1 and M2 are given in prEN 1997-1:2022 Annex A.
[c] Including the properties if any improved ground
[d] Values in brackets are given for accidental design situations.
[E] See prEN 1997-1:2022, 8.2</td></tr>
</table>

11.7 Serviceability limit states

(1) Serviceability limit states of structures founded on ground improvement shall be verified according to all relevant clauses of prEN 1997-3:2022, by calculation or testing.

11.8 Implementation of design

11.8.1 General

(1) prEN 1997-1:2022, 10 shall apply ground improvement.

(2) The execution of ground improvement techniques shall comply with an appropriate standard, as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(3) Where no execution standard exists, the method of execution control shall be specified in the Execution specification.

11.8.2 Inspection

(1) prEN 1997-1:2022, 10.3 shall apply ground improvement.

(2) Where ground improvement is to be installed within ground that contains natural or artificial chemicals or materials, additional inspection tests shall be carried out to ensure that the required improved ground properties are achieved.

(3) Inspection tests may be based on:

– laboratory testing of improved ground samples;
– laboratory testing of binders utilising groundwater;
– other testing to determine specific properties.

(4) Where materials are to be used for which there is no European testing standard available, inspection tests shall be carried out as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(5) Installation parameters for the ground improvement should be monitored and recorded either in real time using bespoke instrumentation or manually by site personnel in agreement with the corresponding execution standard.

11.8.3 Monitoring

(1) prEN 1997-1:2022, 10.4 shall apply ground improvement.

11.8.4 Maintenance

(1) prEN 1997-1:2022, 10.5 shall apply ground improvement.

(2) Where the ground improvement is exposed to the effects of the environment, which can cause deterioration of performance over time, the design shall specify the maintenance activities and protection of the ground improvement to deterioration and loss of resistance.

NOTE Some ground improvement, for example, jet grouted or soil mixing retaining walls can be negatively exposed to freeze/thaw and wet/dry cyclic effects so need to be protected.

11.9 Testing

11.9.1 General

(1) prEN 1997-1:2022, 11 shall apply to ground improvement.

(2) The types of testing should be determined according to the ground improvement technique.

NOTE Execution standards usually contain lists of typical tests relevant to the specific techniques.

(3) Ground improvement techniques trial test before or at the beginning of execution may be conducted, comprising:

- extraction and testing of ground samples to verify the suitability of the foreseen ground treatment; or
- extraction and testing of improved ground samples; or
- execution of trial elements for verification of geometry; or
- execution of trial elements with extraction and testing of samples of treated soil; or
- trial execution and verification by field testing or load testing.

(4) The minimum number of control test should vary based on local experience, ground conditions and the applied ground improvement technique.

(5) For class AII: testing on extracted treated soil samples to verify unconfined compressive strength and other properties;

- for class BI: field testing inside and/or in between inclusions, dummy footing test on improved ground (individual inclusion and surrounding ground), zone load test on a group of inclusions (group of inclusions and surrounding ground);
- for class BII: load test on isolated rigid inclusions, zone load test on a group of rigid inclusions (group of rigid inclusions and surrounding ground,) UCS test on rigid inclusion material.

(6) The minimum frequency of control testing shall be given by the execution standard or by the relevant authority or, where not specified, as agreed by the relevant parties for a specific project.

NOTE The minimum frequencies for control test for each ground improvement class are given in Table 11.5 (NDP) unless the National Annex gives different values.

11.9.2 Investigation tests

(1) Investigation tests should be zone loading tests, dummy footings (or skip tests) or extraction and testing of samples.

(2) For AII and BII classes, samples for testing should be taken either by core drilling (EN 12504-1), fresh sampling (EN 12390-2) or, from spoil return if they can be expected to be representative.

(3) The diameter of the sample should be correlated with the largest grain size.

(4) Prior to testing the suitability of the samples for testing may be assessed in accordance with EN 12716:2018, Annex B.

Table 11.5 — (NDP) Testing frequency for ground improvement (control tests)

<table>
<tr><th>Ground Improvement Class</th><th>Type of test</th><th colspan="2">Number of tests</th></tr>
<tr><td>AI</td><td>Field and laboratory testing to the full depth of the improved ground</td><td colspan="2">As per prEN 1997-2:2022 Clause 5.4.3</td></tr>
<tr><td>AII</td><td>Tests on extracted treated-soil samples</td><td colspan="2">1 test per 125m² with minimum of 4 tests</td></tr>
<tr><td rowspan="4">BI</td><td rowspan="3">Dummy footing tests or zone load test</td><td>≤ 600 elements</td><td>1 in 100</td></tr>
<tr><td>601 to 2000 elements</td><td>6 + 1 additional per 200 (maximum 13)</td></tr>
<tr><td>≥ 2000 elements</td><td>13 + 1 additional per 250</td></tr>
<tr><td>Field and laboratory testing to the full depth of the improved ground</td><td colspan="2">As per prEN 1997-2:2022 Clause 5.4.3</td></tr>
<tr><td rowspan="4">BII</td><td rowspan="3">Load test on isolated inclusion or zone lad test</td><td>≤ 600 elements</td><td>1 in 100</td></tr>
<tr><td>601 to 2000 elements</td><td>6 + 1 additional per 200 (maximum 13)</td></tr>
<tr><td>≥ 2000 elements</td><td>13 + 1 additional per 250</td></tr>
<tr><td>UCS test on rigid inclusion material</td><td colspan="2">1 UCS test per 125m² with minimum of 4 tests</td></tr>
</table>

11.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to ground improvement.

12 Groundwater control

12.1 Scope and field of application

12.1.1 General

(1) This clause shall apply to groundwater control measures, to prevent limit states in the geotechnical structure due to changes in groundwater and/or surface water.

(2) The design and verification of the geotechnical structure, including the implicated groundwater control measures, shall be conducted in accordance with the clauses for that geotechnical structure.

NOTE Geotechnical structures are, but not limited to: dams, levees, embankments, slopes, cuttings, excavations, reinforced fill structures, retaining structures and foundations.

(3) The serviceability criteria and corresponding limiting design value for the groundwater control measures shall be determined by the appropriate clause in this standard.

NOTE 1 Embankments, slopes, cuttings and excavations see Clause 4.

NOTE 2 Spread foundations see Clause 5.

NOTE 3 Retaining structures see Clause 7.

NOTE 4 Reinforced fill structures see Clause 9.

(4) This clause shall not apply to the verification of water retention by dams and levees.

NOTE 1 For these structures additional provisions are needed.

NOTE 2 Methods of assessing critical hydraulic gradients are given in The International Levee Handbook, CIRIA Report C731 (2013).

(5) This clause shall apply to the verification of the appropriate ground water control measures to verify the limit states, durability and robustness of the geotechnical structure involved.

(6) This clause shall apply to verification of the limit states, durability and robustness of the groundwater control measure itself.

(5) Groundwater control measures shall be defined according to this clause to ensure that serviceability criterion according to prEN 1997-1:2022, 9 and 4.2.5 is not violated.

NOTE Serviceability criterion is expressed as a limiting design value including required water level, allowed water flow and groundwater pressure.

(6) Groundwater control measures may be divided in three main groups:

- measures to reduce the hydraulic conductivity;
- dewatering or infiltration to control the groundwater and/or surface water;
- impermeable barriers to control the groundwater by preventing and/or cutting off the flow.

(7) Ground improvement techniques to increase strength, stiffness, and/or accelerate consolidation and consolidation-rate of the ground shall be verified in accordance with Clause 11.

(8) Measures from different groups may be combined to achieve the needed groundwater control.

12.1.2 Reduction of hydraulic conductivity

(1) Reasons to measures to reduce hydraulic conductivity may include, but are not limited to:

- create a barrier in any groundwater flow under, around or aside a geotechnical structure;
- reduce and control water ingress into the excavation;
- reduce and control water egress out of or out to the surrounding environment;
- create suitably dry conditions for excavation and/or installation of ground reinforcement elements;
- control uplift from groundwater pressure on the geotechnical structure;
- reduce groundwater pressure downstream the geotechnical structure;
- environmental and contamination reasons.

(2) The following techniques may be for groundwater control to reduce hydraulic conductivity, but are not limited to:

- grouting;
- soil mixing;
- ground freezing.

12.1.3 Dewatering and infiltration

(1) Reasons to dewatering or infiltration may include, but are not limited to:

- create controlled groundwater and/or surface water flow under, around or aside the geotechnical structure;
- control water ingress into the excavation;
- maintain a controlled existing, transient or new permanent level of groundwater;
- control uplift from groundwater and/or reduce pressure on the geotechnical structure.

12.1.4 Impermeable barriers

(1) Reasons to impermeable barriers may include, but are not limited to:

- create a barrier in groundwater flow under, around or aside a geotechnical structure;
- maintain a controlled existing, transient or new permanent level of groundwater;
- control or cut off water ingress into the excavation;
- increase length of seepage path to decrease gradients;
- control groundwater pressure.

12.2 Basis of design

12.2.1 Design situation

(1) prEN 1997-1:2022, 4.2.2 shall apply to groundwater control.

(2) The selection of measures for groundwater control shall be determined according to its purpose for the geotechnical structure involved.

(3) In addition to prEN 1997-1:2022, 4.2.2, the design situations for groundwater control measures shall include, but are not limited to:

- temporary or permanent nature of the groundwater control;

- location of discontinuities, weathered zones and layers in the ground with high hydraulic conductivity;
- impact within in the zone-of-influence due to the groundwater control measures.

12.2.2 Geometrical properties

(1) prEN 1997-1:2022, 4.3.3 shall apply to measures for groundwater control

12.2.3 Actions and environmental influences

(2) prEN 1997-1:2022, 4.3.1 shall apply to measures for groundwater control.

(3) The limiting design value of the involved geotechnical structure's serviceability criterion for the groundwater pressure and/or groundwater flow shall be obtained from one of the following:

- verification of limit states for the involved geotechnical structure;
- limiting values to avoid impact in the zone of influence.

NOTE The limiting design value of the relevant geotechnical structures serviceability criterion can be expressed as:

- required groundwater level or surface water level;
- allowed hydraulic conductivity;
- allowed flow of water; or
- maximum groundwater pressure acting on the structure.

12.2.4 Limit States

(1) prEN 1997-1:2022, 4.2.5, 8.1.4, 9.1, and 9.4 shall apply to measures for groundwater control.

(2) In addition to (1) prEN 1990:2021, 8.4.1 shall apply to measures for groundwater control.

(3) Potential limit states other than those given in prEN 1997-1 and prEN 1990, should be verified.

NOTE Examples of other limit states are e.g. environmental demands.

(4) For uplift, hydraulic heave, internal erosion, and piping prEN 1997-1:2022, 8.1.4 shall apply.

12.2.5 Robustness

(1) prEN 1997-1:2022, 4.1.4 shall apply to measures for groundwater control.

12.2.6 Ground investigation

(1) In addition to prEN 1997-2:2022, 5, provisions for groundwater and geohydraulic properties prEN 1997-2:2022, 11, shall apply.

(2) The zone of influence in the ground, into which groundwater control measures extends, shall be included in the ground investigation.

(3) Ground investigations shall provide results to identify groundwater properties, hydrogeological conditions and hydraulic properties.

NOTE Examples of groundwater properties are level, quality, and flow.

(4) Water flow or hydraulic measurements should be applied to identify hydrogeological conditions.

12.2.7 Geotechnical reliability

(1) prEN 1997-1:2022, 4.1.2 shall apply to measures for groundwater control.

12.3 Material

12.3.1 Ground

(1) prEN 1997-1:2022, 5.1 and prEN 1997-2:2022 shall apply to measures for groundwater control.

12.3.2 Groundwater

(1) prEN 1997-1:2022, 6.1 and prEN 1997-2:2022 shall apply to measures for groundwater control

12.3.3 Grouting materials

(1) prEN 1997-1:2022, 5.4 shall apply for cement-based grout.

12.3.4 Materials for Dewatering and infiltration

(1) prEN 1997-1:2022, 5.3 shall apply for geosynthetic drainage systems.

12.3.5 Materials for Impermeable barriers

(1) prEN 1997-1:2022, 5.3 shall apply to geomembrane, geosynthetic or plastic barrier

(2) prEN 1997-1:2022, 5.4 shall apply to impermeable grouted barriers

(3) prEN 1997-1:2022, 5.5 shall apply to concrete barriers or sealings.

(4) prEN 1997-1:2022, 5.6 shall apply to steel pile and sheet pile barriers.

(5) EN 1538 should apply to diaphragm walls.

(6) In addition to prEN 1997-1:2022, 5.3, ISO/TS 13434 may be applied for geomembrane, geosynthetic or plastic barriers.

12.3.6 Other materials

(1) Materials other than specified shall only be used, if they comply with a standard specified by the relevant authority or, where not specified, as agreed for a specific project by appropriate parties.

12.4 Groundwater

(1) prEN 1997-1:2022, 6 shall apply to measures for groundwater control.

12.5 Reduction of hydraulic conductivity

12.5.1 Geotechnical analysis

12.5.1.1 General

(1) Techniques and materials to reduce hydraulic conductivity shall be selected to avoid violation of the limiting design value of groundwater pressure or groundwater flow required in 12.2.4 for the geotechnical structure involved.

(2) The selection of an appropriate technique to reduce hydraulic conductivity should account for:

– suitability with respect to ground conditions;
– design service life;
– design situation;
– impact within the zone of influence;
– environmental influences;
– possibility of inspection and maintenance.

(3) For design considerations of grouting EN 12715 should apply.

(4) Penetrability of grout or other injection material in the ground shall be incorporated in the geotechnical analysis.

(5) The effects and risks of execution techniques shall be incorporated in verification of limit states.

12.5.1.2 Material specification

(1) The grout design shall take into account the following, but is not limited to:

– time related properties, related to mixing, hydration and hardening;
– ratios of material and water components;
– rheological properties, such as viscosity;
– penetration related properties, such as grain size vs. apertures;
– pressures of grouting and groundwater;
– salinity of groundwater;
– necessity and types of additives;
– chemical ingredients and their effect on the environment.

12.5.1.3 Design specification

(1) The grouting technique and sequence shall be considered in the design and verification of grouting.

(2) The execution specification should include on-site verification and stop-criteria, based on pressure, flow or mass regulation.

(3) The design of grouting shall take into account other possible measures, structures or elements in the ground, that affects grouting results.

(4) The execution specification of grouting shall include, but is not limited to:

– required limitation for groundwater control;
– required grout penetration depth or spread;
– geometry of the grouting holes, including location, length, direction, overlap and frequency;
– grouting pressures, flows and volumes;
– depth of packer in relation with grouting pressure and failure due to grouting pressure;
– type and use of equipment;
– sequence or sequences of grouting of the holes;
– timing of the grouting in relation with excavation works.

(5) The selection of appropriate grouting may include multiple different types of grouting materials.

12.5.2 Ultimate limit states

(1) In addition to prEN 1997-1:2022, 8.1.4, the following ultimate limit states, potentially caused by the groundwater control, shall be verified in accordance with the geotechnical structure involved:

- failure of the ground due to excessive grout pressure;
- failure of the packers due to excessive grout pressure.

(2) The limiting design value of the hydraulic conductivity of the ground inside the zone of influence of the geotechnical structure involved shall be verified.

12.5.3 Serviceability limit states

(1) In addition to prEN 1997-1:2022, 9.4, the following serviceability limit states, potentially caused by the groundwater control, shall be verified:

- filling of basement or other constructed underground opening with grout due to excessive grout inflow.

(2) The limiting design value of the geotechnical structure involved, may be expressed in terms of:

- limiting values of groundwater level changes within the zone-of-influence;
- limiting value of leakage per unit area;
- limiting value of groundwater flow;
- limiting value of hydraulic conductivity or transmissivity.

(3) Inspection and monitoring shall be used to verify the compliance with (1) during the design service life of the groundwater control system.

12.6 Dewatering and infiltration

12.6.1 Geotechnical analysis

(1) Techniques for dewatering and infiltration shall be selected to avoid violation of the limiting design value of groundwater level, pressure or flow required in 12.2.4 for the geotechnical structure involved.

(2) The selection of an appropriate dewatering or infiltration system should account for

- suitability for the considered ground conditions;
- design service life;
- design situation;
- impact within the zone of influence;
- environmental influences;
- possibility of inspection and maintenance.

NOTE Typical parts of drainage systems are listed below, but not limited to.

- drains, liners, infiltration and well pipes;
- ditches, wells, well points and bore holes;
- pumps, submersible, external and vacuum pumps;
- mains, basins, filters, separators and flow meters.

(3) The necessity for the use of pumps should be determined.

(4) Pumping capacity requirements should be established.

(5) The verification of the appropriateness of the selected dewatering or infiltration system shall include;

- quantity and pressure of any discharge;
- chemical content of any discharge.

(6) Unless it can be demonstrated by comparable experience and assessment of any water discharge that the dewatering or infiltration system will operate adequately without maintenance, a Maintenance Plan shall be specified.

(7) It shall be demonstrated, both by comparable experience and by assessment of any water discharge, that the drainage system will operate adequately without maintenance.

(8) The execution specification for the dewatering or infiltration system shall include, but is not limited to:

- required limitation for groundwater control;
- material selection;
- installation technique and sequence;
- type and use of equipment;
- timing of the groundwater control installation in relation with excavation works.

12.6.2 Ultimate limit states

(1) In addition to prEN 1997-1:2022, 8.1.4, the following ultimate limit state, potentially caused by the groundwater control, shall be verified in accordance with the geotechnical structure involved:

- Failure of ground outside the barrier due to increase in groundwater pressure, as a result of cut-off groundwater flow.

12.6.3 Serviceability limit states

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit states, potentially caused by the groundwater control, shall be verified:

- deformations of adjacent geotechnical structures due to lowering of groundwater;
- deformation of adjacent geotechnical structures due to infiltration.

(2) The limiting design value of the geotechnical structure involved, may be expressed in terms of:

- groundwater levels at different locations within the zone-of-influence;
- groundwater flow;
- drawdown;
- quantity of water to be pumped;
- head losses.

(3) Inspection and monitoring shall be used to verify the compliance with (1) during the design service life of the groundwater control system.

12.7 Impermeable barriers

12.7.1 Geotechnical analysis

(1) Techniques for barriers shall be selected to avoid violation of the limiting design value of groundwater level, pressure or flow required in 12.2.4, for the geotechnical structure involved.

(2) The selection of an appropriate barrier should account for

- suitability for the considered ground conditions;
- design service life;
- design situation;
- impact within the zone of influence;
- environmental influences;
- possibility of inspection and maintenance.

(3) The execution specification for the impermeable barrier shall include, but is not limited to:

- required limitation for groundwater control;
- material selection;
- installation technique and sequence;
- geometry of the impermeable barrier;
- type and use of equipment;
- timing of the grouting in relation with excavation works.

12.7.2 Ultimate limit states

(1) In addition to prEN 1997-1:2022, 8.1.4, the following ultimate limit states, potentially caused by the groundwater control, shall be verified in accordance with the geotechnical structure involved:

- Structural capacity of any vertical cut-off wall or horizontal bottom sealing.

(2) Verification of any structural resistance of the cut-off wall shall comply with clause 7.

12.7.3 Serviceability limit states

(1) In addition to prEN 1997-1:2022, 9, the following serviceability limit state potentially caused by the groundwater control, shall be verified:

- flooding of adjacent geotechnical structures and utilities due to installation of barriers, as a result of cut-off groundwater flow.

(2) The limiting design value of the geotechnical structure involved, may be expressed in terms of:

- groundwater levels at different locations within the zone-of-influence;
- groundwater flow;
- leakage under or around the barrier.

(3) Inspection and monitoring shall be used to verify the compliance with (1) during the design service life of the groundwater control system.

12.8 Implementation of design

12.8.1 General

(1) The hydraulic conductivity of all geotechnical units inside the zone of influence shall be considered both before and after execution to ensure that the design is applicable.

(2) For the application of the Observational Method during execution prEN 1997-1:2022, 10.6 shall apply.

(3) Execution of grouting should comply with EN 12715.

(4) Execution of jet-grouting should comply with EN 12716.

(5) Execution of vertical drainage should comply with EN 15237.

(6) Execution of barriers by diaphragm walls should comply with EN 1538.

(7) Execution of deep mixing should comply with EN 14679.

12.8.2 Supervision

(1) prEN 1997-1:2022, 10.2 shall apply to measures for groundwater control.

12.8.3 Inspection

12.8.3.1 General

(1) prEN 1997-1:2022, 10.3 shall apply to measures for groundwater control.

(2) Inspection shall include check of proper installation of groundwater control system and functionality control of it.

(3) Inspection shall include the check of the grouting equipment in relation with the design, demands and assumptions used in the design.

(4) Inspection shall include the ground or groundwater conditions on site in relation with the assumptions made in the Geotechnical Design Model.

(5) Groundwater conditions should be measured.

NOTE Table 12.1 (NDP) give measures to check the groundwater conditions within the zone of influence, unless the national Annex give different guideline.

Table 12.1 — (NDP) Measures for checking groundwater conditions within the zone of influence

Geotechnical Category	Measures / Measurements
GC3	All the items given below for GC2 and, in addition: - More detailed examination that includes additional measurements and observations.
GC2	All the items given below for GC1 and, in addition: - measurements of groundwater levels and groundwater pressures; - measurements of groundwater flow and chemistry, if they affect the method of construction or the performance of the structure.
GC1	All the items given below: - direct observations; - documented comparable experience; - any other relevant evidence.

(6) The following items should be inspected in relation to groundwater control:

- Groundwater flow and groundwater pressure regime;
- effects of dewatering operations on groundwater table;
- effectiveness of measures taken to control seepage inflow or egress;
- internal erosion processes and piping;
- chemical composition of groundwater;
- corrosion potential;
- adequacy of systems to ensure control of groundwater pressures in all aquifers where excess pressure could affect stability of slopes or base of excavation, including artesian pressures in an aquifer beneath the excavation;
- disposal of water from dewatering systems;
- depression of groundwater table throughout entire excavation to prevent boiling or quick conditions, piping and disturbance of formation by construction equipment;
- diversion and removal of rainfall or other surface water.

12.8.3.2 Reduction of hydraulic conductivity

(1) The reduction in hydraulic conductivity shall be measured or derived from other measurements.

(2) The ingress, flow and/or egress of water should be measured.

(3) The reduction in hydraulic conductivity water should be measured or derived from other measurements.

(4) Inspection shall include the compliance of grouting sequencing with the design, demands and assumptions used in the design.

(5) Control measures should be conducted on the hydraulic properties after execution

12.8.3.3 Dewatering and infiltration

(1) In addition to prEN 1997-1:2022, 10.3, the Inspection Plan should specify measures to check:

- efficient and effective operation of dewatering systems throughout the entire construction period, considering encrusting of well screens, silting of wells or sumps;
- wear in pumps;
- clogging of pumps
- control of dewatering to avoid disturbance of adjoining structures or areas;
- effectiveness, operation and maintenance of water recharge systems, if installed;
- effectiveness of any sub-horizontal borehole drains;
- standby equipment to maintain groundwater controls in case of pumping failure/power.

(2) The Inspection Plan should include:

- chemical composition of groundwater;
- durability of the reinforcing element.

12.8.3.4 Impermeable barriers

(1) The groundwater levels on both sides of the barrier shall be measured prior to installation.

(2) The groundwater levels, absence of flow ingress, and/or egress of water on both sides of the barrier should be measured after installation.

12.8.4 Monitoring

12.8.4.1 General

(1) In addition to prEN 1997-1:2022, 10.4 the Monitoring Plan should include:

- observations of piezometric levels;
- flow measurements.

(2) The results of monitoring should define the necessity and steer the implementation of further groundwater control.

(3) Groundwater levels and/or groundwater pressure shall be monitored.

(4) Groundwater level monitoring should be conducted continuously or semi-continuously in adequate intervals.

(5) Groundwater level monitoring should be conducted prior, during and after groundwater control works and works affecting groundwater levels.

12.8.4.2 Reduction of hydraulic conductivity

(1) Grouting time, pressures, flow and mass intake shall be monitored during grouting.

(2) Groundwater levels and changes herein shall be monitored.

(3) For work in freezing conditions the air and rock temperature should be monitored.

(4) In case of freezing conditions heating or frost prevention measures should be implemented.

12.8.4.3 Dewatering and infiltration

(1) Groundwater levels pressure under buildings or in adjoining areas should be monitored.

NOTE Especially important if deep drainage or permanent dewatering systems are installed or if deep basements are constructed.

(1) When pumps are installed, the pumping amounts shall be monitored.

(2) The effects of dewatering operations on the groundwater table shall be monitored.

12.8.4.4 Impermeable barriers

(1) The groundwater levels, on both sides of the barrier shall be monitored prior to installation and use.

(2) The groundwater levels, absence of flow ingress, and/or egress of water on both sides of the barrier should be monitored after installation and during use.

12.9 Testing

(1) prEN 1997-1:2022, 11 shall apply to measures for groundwater control.

NOTE For geohydraulic testing see prEN 1997-2:2022, 11

(2) Testing of grout material properties shall be conducted.

(3) One or more of the following testing methods should be used for design and verification of rock grouting:

– hydrostatic pressure build-up testing in the bore hole;
– water leakage measurements from the rock mass into the bore hole;
– water loss measurements from the bore hole into the rock mass.

(4) Testing of grouting should be conducted prior to start of grouting and after grouting.

(5) Pumps and pumping system should be tested prior to installation.

(6) The functioning of drainage systems should be tested.

NOTE An option to enhance is to rinse or flush after installation.

(7) Flow rate of geo-composite drains should be measured according to EN ISO 12958-1 or EN ISO 12958-2.

12.10 Reporting

(1) prEN 1997-1:2022, 12 shall apply to measures for groundwater control .

Annex A
(informative)

Slopes, cuttings, and embankments

A.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to Clause 4 regarding slopes, cuttings, and embankments.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

A.2 Scope and field of application

(1) This Informative Annex covers calculation methods for the stability of slopes, cuttings and embankments in soil, fill and rock.

A.3 Calculation models for analysing the stability of soil and fill

(1) A calculation method for analysing the stability of soils and fills should only be used if it is appropriate for the Ground Model, potential failure surface, and loading conditions.

NOTE 1 Table A.1 provides a non-exhaustive list of calculation models based on limiting equilibrium.

NOTE 2 Procedures for numerical models are given in prEN 1997-1:2022, 8.2.

(2) Three-dimensional effects may be considered in design verification when using a two-dimensional calculation method, provided the adjustment is on the safe side and the method is validated.

(3) When choosing a calculation model for analysing the stability, the following should be included in the Geotechnical Design Model, but is not limited to:

- weight density determined using the single source principle [see prEN 1990:2021, 6.1.1(4)];
- soil layering;
- occurrence and orientation of zones or layers of low strength;
- seepage and groundwater pressure distribution;
- drained or undrained behaviour or a combination;
- creep deformations due to shear;
- type of anticipated failure;
- possibility of progressive failure along the slip surface (strain compatibility);
- external actions, their duration and direction;
- use of stabilizing measures;
- adjacent or intersecting structures;
- strength anisotropy; and
- interface with underlying rock.

Table A.1 — Calculation methods for analysing the stability of soil and fill

Method[c]		Type of method[a,b]	Special design conditions/limitations	Comments and assumptions
1	Bishop (simplified and rigorous)	Slices, circular arc	Not recommended with external horizontal loads	Simplified ignores interslice shear forces when interslice forces are horizontal
2	Generalized limit equilibrium	Slices, any shape of surface	Applicable with all slope geometries and soil profiles	---
3	Janbu generalized (modified)	Slices, circular arc, non-circular, polyline		Location of interslice normal force is assumed by a line of thrust
4	Morgenstern-Price			Direction of interslice forces by variable user function
5	Spencer			Constant interslice forces function
6	Sarma	Slices, polyline	Seismic loading, critical acceleration. Static conditions: horizontal load set to zero	Can include non-vertical slices and multi-wedge failure mechanisms
7	Kinematical approach of limit analysis	Multiple body, blocks, circular, planar or logarithmic spiral	---	Based on the compatibility of velocity fields, no consideration to stress diffusion
8	Block/wedge method	Multiple body, polyline	Pre-defined planar failure surface. Divided into three segments	Earth-pressure can be used as driving and resisting force. No moment equilibrium
9	Multiple wedge method	Multiple body, blocks, wedges, plane surfaces	---	No moment equilibrium.
10	Infinite slope	Single body, plane surface	Long shallow slopes	
11	Culmann, finite slope		Steep slopes, drained analysis	
12	Logarithmic spiral	Single body; logarithmic spiral	Homogeneous soil, drained analysis	Satisfies moment and force equilibrium

[a] Where ground or embankment material is relatively homogeneous and isotropic, circular failure surfaces can normally be assumed, except when high external loads are present.

[B] Polyline includes interconnected plane surfaces.

[C] See 1) Bishop (1965); 2) Fredlund and Krahn (1977); 3) Janbu (1954); 4) Morgenstern and Price (1965); 5) Spencer (1967); 6) Sarma (1979); 8)9) DIN 4084:2009-01; 11) Coulomb (1776), adapted by Cullman (1866); 12) Froelich (1953).

A.4 Calculation models for analysing the stability of rock mass

(1) A calculation method for analysing the stability of rock mass should only be used if it is appropriate for the Ground Model, potential failure surface, and loading conditions.

NOTE Table A.2 provides a non-exhaustive list of calculation models for rock mass based on limiting equilibrium.

(2) When choosing a calculation method for analysing the stability of rock masses, the following should be included in the Geotechnical Design Model, but is not limited to:

- weight density;
- rock layering, weakness zones and discontinuities;
- Interfaces with soil and soil layers on top;
- geometrical properties of weakness zones and discontinuities;
- infill of weakness zones and discontinuities;
- seepage and groundwater pressure distribution;
- types of anticipated failure;
- external actions and their duration and direction;
- use of stabilizing measures; and
- adjacent or intersecting structures;

Table A.2 — Calculation models for analysing the stability of rock mass

No.	Type of failure	Method[a]	Special design conditions/limitations	Comments and assumptions
1	Circular failure Large slope derformations[g]	Bishop, Janbu, Morgenstern, Spencer[d] Limit equilibrium[e]	Blocky or weathered rock mass.[b] Tension crack with or without water	Method of slices, circular (see Table A4.1)
2	Plane failure	Limit equilibrium[e]	Tension crack with or without water	Plane surface, blocks
3	Wedge failure	Limit equilibrium[e]	Tension crack with or without water	Wedge
4	Block toppling	Limit equilibrium[e]	---	Blocks
5	Flexure toppling	Limit equilibrium[e]	---	Columns
6	Block-flexure toppling	Limit equilibrium[e]	---	Blocks and columns
7	Secondary toppling	Limit equilibrium[e]	---	---
8	Rock fall[c]	Limit equilibrium[e], rigid body, Goodman Shy[f]	Block trajectories, bounce heiths, velocities, energies, run out distances	Blocks

a All methods for 1 to 7 can address circular and plane failure.

B Only valid for failure not controlled by discontinuities.

C Rock fall is the results of type 2 to 7, but 8 addresses the consequence of rock fall to underlying structure.

D See Table A.1 for references
e Limit equilibrium methods include Finite Element, Finite Difference and Discrete Element Methods. see Poisel and Preh (2004), Wyllie (2017)
f See Goodman & Shi (1985)
g Without formation of a sliding plane, i.e. without detachment of rock mass (e.g. slope creep, kink band slumping)

Annex B
(informative)

Spread foundations

B.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to Clause 5 regarding spread foundations.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

B.2 Scope and field of application

(1) This Informative Annex covers:

- checklists;
- calculation models for bearing resistance; and
- calculation models for foundation settlement.

B.3 Checklists

(1) The following features may affect the resistance of a bearing stratum:

- depth of the adequate bearing stratum;
- inclination of the adequate bearing stratum;
- depth of the groundwater level;
- depth above which shrinkage and swelling of clay soils, due to seasonal weather changes, or to trees and shrubs, can cause appreciable movements;
- depth above which frost damage, including heave due to groundwater freezing, can occur;
- excavation below the level of the water table in the ground;
- ground movements and reductions in the resistance of the bearing stratum by seepage or climatic effects or by construction procedures;
- liquefaction caused by cyclic or dynamic loading;
- excavations for services close to the foundation potentially causing bearing failure or foundation movement beyond a serviceability limit state;
- high or low temperatures transmitted from the building, causing desiccation and settlement or groundwater freezing and heave;
- scour;
- variation of water content due to long periods of drought, and subsequent periods of rain, on the properties of volume-unstable soils in arid climatic areas;
- the presence of soluble materials, e.g. limestone, claystone, gypsum, salt rocks; and
- the presence of existing voids formed by geological processes or prior human activities.

(2) The following features of rock may affect the design of spread foundations on rock

- deformability and strength of the rock mass and the permissible settlement of the supported structure;
- presence of any weak layers, for example solution features or fault zones, beneath the foundation;

- presence of bedding joints and other discontinuities and their characteristics (for example filling, continuity, width, spacing);
- state of weathering, decomposition and fracturing of the rock; and
- disturbance of the natural state of the rock caused by construction activities, such as, for example, underground works or slope excavation, being near to the foundation.

B.4 Calculation model for bearing resistance using soil parameters

(1) The undrained bearing resistance factors in Formula (5.3) may be determined from Formula (B.1):

$$N_{\mathrm{cu}} = \pi + 2$$
$$N_{\gamma \mathrm{u}} = -2\sin\beta \qquad \text{(B.1)}$$

where:

β is the slope of the ground surface, downwards from the edge of the foundation.

(2) The following non-dimensional factors may be used in Formula (5.3):

- base factor b_{cu};
- depth factor d_{cu};
- ground inclination factor g_{cu};
- load inclination factor i_{cu}; and
- shape factor s_{cu}.

(3) The non-dimensional factors in (2) may be determined from Formula (B.2):

$$b_{\mathrm{cu}} = 1 - \frac{2\alpha}{\pi + 2} \qquad d_{\mathrm{cu}} = 1 + 0{,}33\tan^{-1}\left(\frac{D}{B}\right)$$

$$g_{\mathrm{cu}} = 1 - \frac{2\beta}{\pi + 2} \geq 0 \qquad i_{\mathrm{cu}} = \frac{1}{2}\left(1 + \sqrt{1 - \frac{T}{A'c_{\mathrm{u}}}}\right), T \leq A'c_{\mathrm{u}} \qquad \text{(B.2)}$$

$$s_{\mathrm{cu}} = 1 + 0{,}2\left(\frac{B'}{L'}\right) \quad \text{for a rectangular foundation or 1,2 for circular foundation}$$

where:

α is the inclination of the foundation base (in radians);

D is the embedment depth of the foundation;

B is the breadth of the foundation;

β is the inclination of the ground surface, downwards from the edge of the foundation (in radians);

B′ is the effective width of the foundation;

L′ is the effective length of the foundation;

T is the force applied tangentially to the base of the foundation;

A′ is the foundation's effective area on plan;

c_{u} is the soil undrained shear strength,

NOTE d_{cu} should be taken as 1.0 when the strength of the soil above the embedment depth D is less than that at the foundation level.

(4) The drained bearing resistance factors in Formula (5.7) may be determined from Formula (B.3):

$$N_{\mathrm{q}} = e^{\pi tan\varphi'} \tan^2\left(45 + \frac{\varphi'}{2}\right)$$
$$N_{\mathrm{c}} = \left(N_{\mathrm{q}} - 1\right)cot\varphi' \tag{B.3}$$
$$N_{\gamma} = 2\left(N_{\mathrm{q}} + 1\right)\tan\varphi' \text{ for a rough base (i.e. } \delta \geq \varphi'/2)$$

where:

φ′ is the soil angle of internal shearing resistance;

δ Is the angle of interface friction between the foundation and the ground.

(5) The following non-dimensional factors may be used in Formula (5.7):

- base factors b_c, b_q, and b_γ;
- depth factors d_c, d_q, and d_γ;
- ground inclination factors g_c, g_q, and g_γ;
- load inclination factors i_c, i_q, and i_γ; and
- shape factors s_c, s_q, and s_γ.

(6) The non-dimensional factors in Formula (5.7) may be calculated from Formula (B.4):

$$b_{\mathrm{c}} = b_{\mathrm{q}} - \left(\frac{1 - b_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi'}\right); b_{\mathrm{q}} = b_{\gamma} = (1 - \alpha\tan\varphi')^2$$
$$d_{\mathrm{c}} = d_{\mathrm{q}} - \left(\frac{1 - d_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi'}\right); d_{\gamma} = 1$$
$$d_{\mathrm{q}} = 1 + 2\tan\varphi' (1 - \sin\varphi')^2 (D/B) \text{ for } D/B \leq 1.0$$
$$d_{\mathrm{q}} = 1 + 2\tan\varphi' (1 - \sin\varphi')^2 \tan^{-1}(D/B) \text{ for } D/B > 1.0$$
$$g_{\mathrm{c}} = g_{\mathrm{q}} - \left(\frac{1 - g_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi'}\right) = \left(\frac{g_{\mathrm{q}} N_{\mathrm{q}} - 1}{N_{\mathrm{q}} - 1}\right); g_{\mathrm{q}} = g_{\gamma} = (1 - \tan\beta)^2$$
$$i_{\mathrm{c}} = i_{\mathrm{q}} - \left(\frac{1 - i_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi'}\right) = \left(\frac{i_{\mathrm{q}} N_{\mathrm{q}} - 1}{N_{\mathrm{q}} - 1}\right); i_{\mathrm{q}} = \left(1 - \frac{T}{N}\right)^m; i_{\gamma} = \left(1 - \frac{T}{N}\right)^{m+1}$$
$$m = m_B = \frac{2 + (B'/L')}{1 + (B'/L')} \text{ when } T \text{ acts in the direction of } B' \tag{B.4}$$
$$m = m_L = \frac{2 + (L'/B')}{1 + (L'/B')} \text{ when } T \text{ acts in the direction of } L'$$
$$m = m_\theta = m_L \cos^2\vartheta + m_B \sin^2\vartheta \text{ for other loading directions}$$
$$s_{\mathrm{c}} = \left(\frac{s_{\mathrm{q}} N_{\mathrm{q}} - 1}{N_{\mathrm{q}} - 1}\right)$$
$$s_{\mathrm{q}} = 1 + \left(\frac{B'}{L'}\right)\sin\varphi' \text{ for a rectangular or circular } (B' = L') \text{ foundation}$$
$$s_{\gamma} = 1 - 0.3\left(\frac{B'}{L'}\right) \text{ for a rectangular or circular } (B' = L') \text{ foundation}$$

where, in addition to the symbols defined for Formula (B.2):

φ′ is the angle of effective friction;

N is the force applied normally to the base of the foundation;

θ is the angle on plan between the L axis and the direction of T.

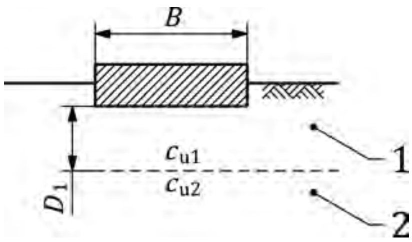
NOTE d_c, d_q, and d_γ should be taken as 1.0 when the strength of the soil above the foundation depth D is less than that at foundation level.

(7) To account for the effect of groundwater level on groundwater pressure and effective weight density in Formula (5.7), when all the ground is fully saturated and there is no seepage, the following values for q' and γ' may be adopted:

– for groundwater level at ground surface:
  $q' = (\gamma - \gamma_w)D$ and $\gamma' = (\gamma - \gamma_w)$
– for groundwater level at a depth D_w below the ground surface but above the foundation level:
  $q' = \gamma D_w + (\gamma - \gamma_w)(D - D_w)$ and $\gamma' = (\gamma - \gamma_w)$
– for groundwater at the foundation level:
  $q' = \gamma D$ and $\gamma' = (\gamma - \gamma_w)$
– for groundwater at a depth exceeding 1.5 B below the foundation level:
  $q' = \gamma D$ and $\gamma' = \gamma$.

B.5 Calculation model for bearing resistance on ground underlain by a weaker layer

NOTE Figure B.1 illustrates foundation on a stronger layer over a weaker layer



Key

1 Stronger layer

2 Weaker layer

B Width of the foundation

D_1 Thickness of the upper layer below the base of the foundation

C_{u1} Shear strength in total stress analyses in upper (stronger) layer

C_{u2} Shear strength in total stress analyses in lower (weaker) layer

Figure B.1 — Foundation on a stronger layer over a weaker layer

(1) In total stress analysis, the bearing resistance R_{Nu} of a rectangular spread foundation founded on a stronger fine soil layer above a weaker fine soil layer, as shown in Figure B.1, may be determined from Formula (B.5):

$$R_{\mathrm{Nu}} = A'(k_1 c_{\mathrm{u1}} N_{\mathrm{cu}} b_{\mathrm{cu}} s_{\mathrm{cu}} i_{\mathrm{cu}} + q)$$
$$k_1 = \frac{c_{\mathrm{u2}}}{c_{\mathrm{u1}}}\left(1 + \frac{D}{B}\right)\left(1 + \frac{D_1}{L}\right) \leq 1.0 \qquad \text{(B.5)}$$

where:

c_{u1} is the undrained strength of the upper (stronger) layer;

c_{u2} is the undrained strength of the lower (weaker) layer;

D_1 is the thickness of the upper layer below the base of the foundation.

NOTE This formula assumes that the stress from the foundation spreads at a rate of 1 horizontal to 2 vertical through the stronger layer.

(2) The bearing resistance R_N of a rectangular spread foundation founded on a stronger coarse soil layer above a weaker fine soil layer may be determined from Formula (B.6):

$$R_{\mathrm{Nu}} = A\left(1+\frac{0.2B}{L}\right)(\pi+2)c_{\mathrm{u2}} + A'\gamma_1' {D_1}^2\left(1+\frac{2D}{D_1}\right)\left(\frac{K_{\mathrm{ps}} tan\varphi_1{'}}{B}\right) + A'\gamma_1 D$$
$$\lambda = \frac{q_2}{q_1} = \frac{(\pi+2)c_{\mathrm{u2}}}{0.5\gamma_1{'}BN_\gamma} \quad \text{(B.6)}$$

where:

φ'_1 is the coefficient of friction for effective stress analyses for upper coarse soil layer;

c_{u2} is the undrained strength of the lower fine soil layer;

D_1 is the thickness of the upper layer;

λ is the ratio of the bearing pressure in the lower layer (q_2) to that in the upper layer (q_1);

q_2 is the bearing pressure in the lower layer;

γ'_1 Is the effective weight density of the upper layer;

K_{ps} is a punching shear coefficient given in Table B.1.

Table B.1 — Values of the punching shear coefficient K_{ps}

$\lambda = q_2/q_1$	Value of K_{ps} for φ'_1 equal to…		
	30°	35°	40°
0	0.8	1.2	2.1
0.2	1.8	2.7	4.3
0.4	2.8	4.4	6.9
1.0	5.4	7.9	12.4

B.6 Calculation model for bearing resistance from pressuremeter test results

(1) The bearing resistance R_N of a spread foundation to normal loads may be determined from the result of Ménard Pressuremeter Tests using Formula (B.7):

$$R_{\mathrm{N}} = A\,\sigma_{\mathrm{v0}} + A'\,k_{\mathrm{p}}\,p^*_{\mathrm{LM,e}} \quad \text{(B.7)}$$

where:

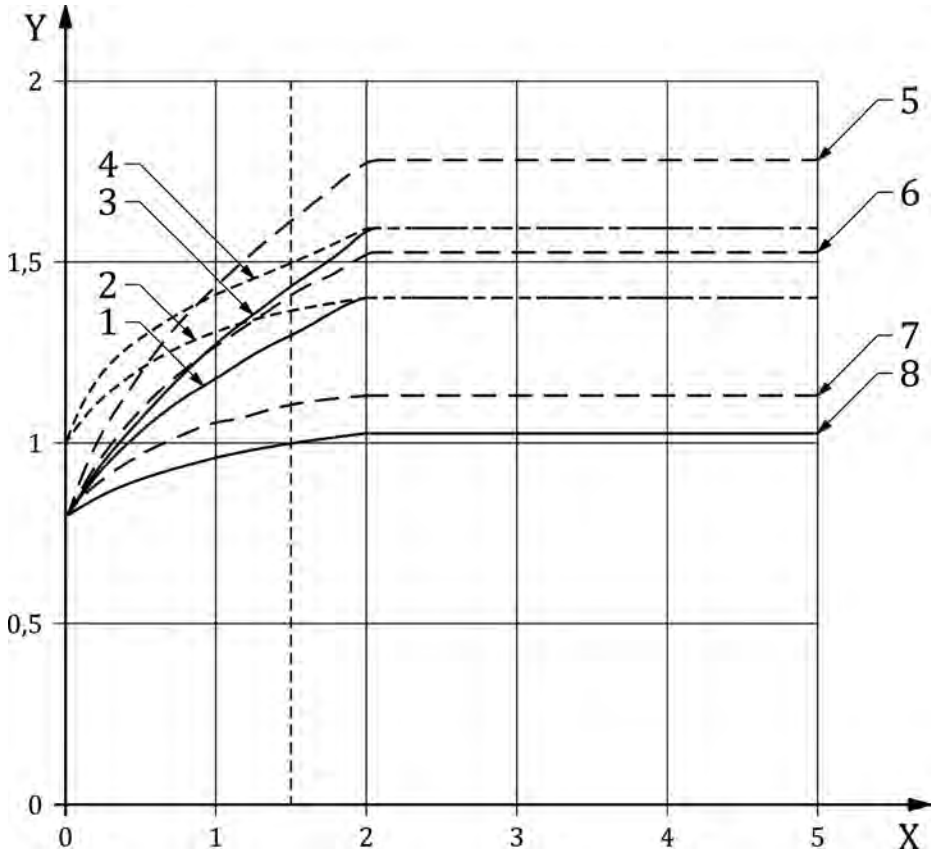
A	is the area of the foundation on plan;
A′	is the effective area of the foundation on plan;
σ_{v0}	is the total vertical stress at the level of the foundation base (after the execution of the foundation);
k_p	is a bearing resistance factor given by graphs according to ground type and foundation shape in Table B.2;
$p^*_{LM,e}$	is the geometric mean on a thickness of 1.5B below the foundation base, of the representative values of the net limit pressure, defined in Formula (B.8);
$p_{LM(z)}$	is the representative value of the Ménard limit pressure at a depth z;
$p_{0(z)}$	is the total (initial) stress at a depth z, defined as $p_{0(z)}=K_0\,(\sigma_{v(z)}-u_{(z)})+u_{(z)}$;
K_0	is the at-rest earth pressure coefficient;
$\sigma_{v(z)}$	is the total vertical stress at the level of the Ménard Pressuremeter Test at a depth z;
$u_{(z)}$	is the groundwater pressure at the level of the Ménard Pressuremeter Test at a depth z.

NOTE 1 The effect of the load inclination is considered by an additional parameter applied on k_p

NOTE 2 This method is described in NFP 94-261.

$$p^*_{\mathrm{LM,e}} = \sqrt[n]{\prod_{i=1}^{n} p^*_{\mathrm{LM}}} = \sqrt[n]{\prod_{i=1}^{n} \left(p_{LM(zi)} - p_{0(zi)}\right)} \tag{B.8}$$

NOTE Figure B.2 give the resistance factor k_p for different ground and foundation shapes.



Key

X	D_e/B	1	Q_1	3	Q_3	5	Q_5	7	Q_7
Y	k_p	2	Q_2	4	Q_4	6	Q_6	8	Q_8

Figure B.2 — Bearing resistance factor k_p versus equivalent embedment depth D_e divided by foundation width B for ground types and foundation shapes given in Table B.2

(2) Weak ground above the foundation level should not be accounted for in the assessment of the equivalent embedment depth, D_e, defined as the thickness of ground above the foundation level having a similar limit pressure as the ground below the foundation.

Table B.2 — Correlations for deriving the bearing resistance factor kp for spread foundations

Ground type	Correlation curves from Figure B.2 to obtain the bearing resistance factor k_p	
	Strip foundation	**Square pad**
Clay and silt	Q1	Q2
Sand and gravel	Q3	Q4
Chalk	Q5	Q6
Marl and weathered rock	Q7	Q8

B.7 Calculation model for settlement evaluation based on adjusted elasticity method

(1) The total settlement *s* of a spread foundation on fine or coarse soil may be determined from Formula (B.9):

$$s = \frac{pB(1-\upsilon^2)I_s}{E_m} \tag{B.9}$$

where:

p is the bearing pressure linearly distributed on the base of the foundation;

B is the width of the foundation;

I_s is an influence factor;

E_m is the representative value of the ground elasticity modulus (see also (4) for rocks) ; and

υ is Poisson's ratio of the ground.

NOTE 1 The value of I_s depends on the stiffness and shape of the foundation area, the variation of stiffness with depth, the thickness of the compressible formation, the distribution of the bearing pressure and the point for which the settlement is determined.

NOTE 2 Values of I_s to calculate the average settlement of a spread foundation on a deep elastic soils are given in Table B.3.

Table B.3 — Values of the influence factor I_s

Foundation stiffness	Value of the influence factor I_s for foundation shape…					
	Circle	Square	Rectangle with L/B equal to			
			2	5	10	100
Flexible	0,85	0,95	1,30	1,83	2,25	3,69
Rigid	0,79	0,82	1,20	1,70	2,10	3,47

(2) If no reliable settlement results, measured on neighboring similar structures in similar conditions are available, the design drained modulus E_m of the deforming stratum for drained conditions may be estimated from the results of laboratory or in-situ tests.

(3) The adjusted elasticity method should only be used if the stresses in the ground are such that no significant yielding occurs and if the stress-strain behaviour of the ground is considered to be linear.

NOTE Great caution is required when using the adjusted elasticity method in the case of non-homogeneous ground.

(4) In case of a spread foundation on rocks, the design value of E_m may be determined from Formula (B.10).

$$E_m = E_{rm} \tag{B.10}$$

where:

E_{rm} is the rock mass modulus (see prEN 1997-2:2022, 9.1.4 (5));

NOTE In literature, there are other expressions for E_{rm} that can be used considering their applicability and limitations.

B.8 Calculation model for settlement evaluation based on stress-strain method

(1) The total settlement of a spread foundation on fine or coarse soil may be evaluated using the stress-strain calculation method as follows:

- computing the stress distribution in the ground due to the loading from the foundation;
  - this may be determined on the basis of elasticity theory, generally assuming homogeneous isotropic soil and a linear distribution of bearing pressure;
- computing the strain in the ground from the stresses using stiffness moduli values or other stress-strain relationships determined from laboratory tests (preferably calibrated against field tests), or field tests; and
- integrating the vertical strains to find the settlements;
  - using the stress-strain method a sufficient number of points within the ground beneath the foundation should be selected and the stresses and strains computed at these points.

B.9 Calculation model for settlements without drainage

(1) The short-term components of settlement of a foundation on fine soil, which occur without drainage, may be evaluated using either the stress-strain method or the adjusted elasticity method.

(2) The values adopted for the stiffness parameters should in this case represent the undrained behaviour with $\upsilon = \upsilon_u = 0.5$

B.10 Calculation model for settlements caused by consolidation

(1) To calculate the settlement of a spread foundation caused by consolidation, a confined one-dimensional deformation of the soil in an oedometer test may be assumed and the consolidation test curve used.

(2) Empirical corrections may be applied to the addition of settlements in the undrained and consolidation state to avoid overestimation of the total settlement.

B.11 Calculation model for time-settlement behaviour

(1) With fine soils the rate of consolidation settlement before the end of the primary consolidation may be estimated by using consolidation parameters obtained from a laboratory compression test.

(2) the rate of consolidation settlement should be obtained using permeability values obtained from in-situ tests.

B.12 Calculation model for settlement evaluation using pressuremeter test results

(1) The settlement of a spread foundation may be determined from the results of Ménard pressuremeter tests using Formula (B.11):

$$s = (q - \sigma_{v0}) \left[\frac{2B_0}{9E_d} \left(\frac{\lambda_d B}{B_0} \right)^{\alpha_r} + \frac{\alpha_r \lambda_c B}{9E_c} \right]$$
$$\frac{1}{E_d} = \frac{0.25}{E_1} + \frac{0.3}{E_2} + \frac{0.25}{E_{3\leftrightarrow 5}} + \frac{0.1}{E_{6\leftrightarrow 8}} + \frac{0.1}{E_{9\leftrightarrow 16}} \quad \text{(B.11)}$$

where:

B	is the width of the foundation;
B_o	is a reference width of 0,6 m;
E_c	is the value of E_M measured in a ground of thickness B/2 immediately below the foundation;
E_d	is the weighted harmonic mean of E_M measured in ground of thickness 8B below the foundation;
$E_{i\leftrightarrow j}$	is the harmonic mean value of E_M measured in layers B/2 thick, counted from 1 below the foundation down to 16 as a depth of 8B;
q	is the design normal pressure applied on the foundation;
α_r	is a rheological factor depending on the nature of ground, as given in Table B.5;
λ_d, λ_c	are shape coefficients depending on the ratio L/B, as given in Table B.4;
σ_{v0}	is the total (initial) vertical stress at the level of the foundation base.

Table B.4 — Shape coefficients for settlement of spread foundations

L/B	**Circle**	**Square**	**2**	**3**	**5**	**20**
λd	1	1,12	1,53	1,78	2,14	2,65
λc	1	1,1	1,2	1,3	1,4	1,5

Table B.5 — Correlations for deriving the rheological factor αr for spread foundations

Type of ground	**Description**	E_M/p_{LM}	α_r
Peat			1,00
Clay	Over-consolidated	> 16	1,00
	Normally consolidated	9 – 16	0,67
	Remoulded	7 – 9	0,5’
Silt	Over-consolidated	> 14	0,67
	Normally consolidated	5 – 14	0,50
Sand	---	> 12	0,50
		5 – 12	0,33
Sand and gravel	---	> 10	0,33
		6 – 10	0,25
Rock	Highly weathered rock		0,67
	Disintegrated rock mass	---	0,33

	Highly fractured rock mass		0,50
	Normally fractured, very blocky rock mass		0,67

B.13 **Calculation model for settlement evaluation using cone penetration test results**

(1) The settlement of a spread foundation on coarse soil under load pressure (q) may be determined from the results of cone penetration using Formula (B.12):

$$s = C_1\, C_2 (q - \sigma'_{\mathrm{v0}}) \int_0^{z_1} \frac{I_z}{C_3\, E'} dz \tag{B.12}$$

where:

C_1 Is $1 - 0{,}5 \times [\sigma'_{\mathrm{v0}}/(q - \sigma'_{\mathrm{v0}})]$;

C_2 is $1{,}2 + 0{,}2 \times \lg t$;

C_3 is the the correction factor for the shape of the spread foundation
1,25 for square foundations; and
1,75 for strip foundations with $L > 10B$;;

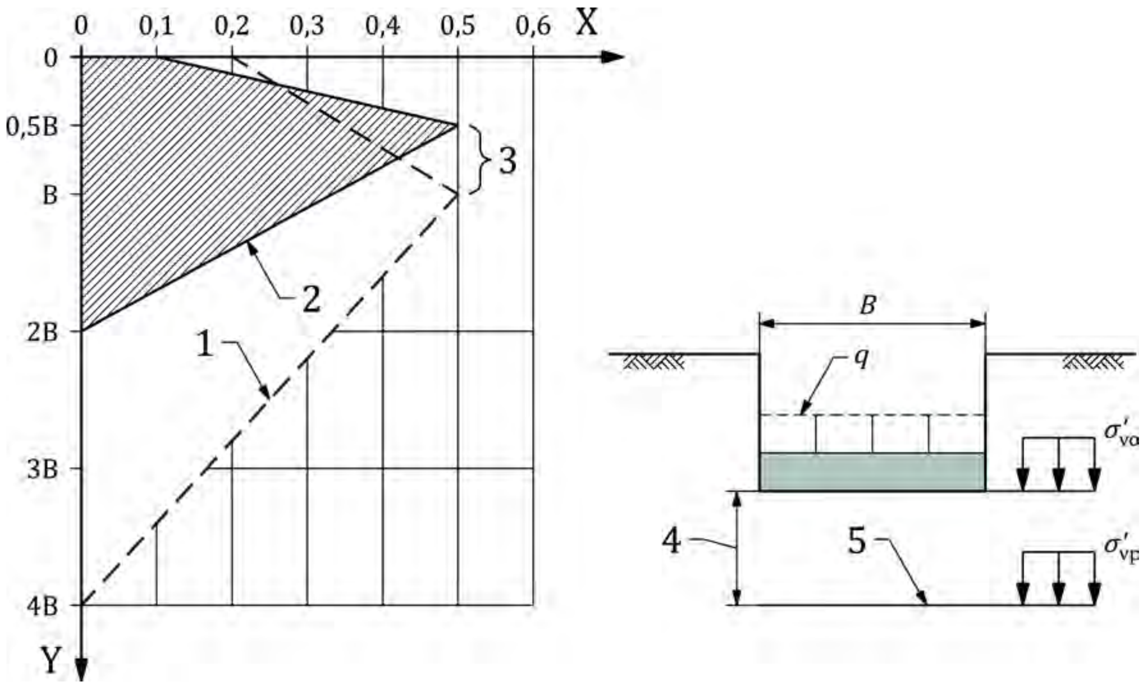
t is the time, in years

σ'_{v0} is the initial effective vertical stress at the level of the foundation

E' the value for Young's modulus of elasticity (E') derived from the cone penetration resistance (q_c), to be used in this method is: $E' = 2{,}0\ q_c$,.

I_z is a strain influence factor (see Figure B.3) where the distribution of the strain influence factor (I_z) are given for axisymmetric (circular and square) spread foundations and for plane strain (strip spread foundations)

NOTE Figure B.3 gives the influence factor for the calculation model published by Schmertmann (1970) and Schmertmann et al (1978)



Key

x	rigid footing vertical strain influence factor I_z
y	relative depth below footing
1	axi-symmetric (L/B=1)
2	plane strain ($L/B > 10$)
3	$B/2$ (axi-symmetric); B (plane strain)
4	depth to I_{zp}

Figure B.3 — Strain influence factor diagrams

B.14 Relative stiffness of a spread foundation and subgrade modulus

(1) The relative stiffness K_s of a rectangular spread foundation may be determined assuming elastic behaviour for the foundation and the ground and Formula (B.13):

$$K_s = 5.57\left(\frac{E_f}{E_g}\right)\frac{(1-\upsilon_g^2)}{(1-\upsilon_f^2)}\left(\frac{B}{L}\right)^{0.5}\left(\frac{D_f}{L}\right)^3 \tag{B.13}$$

where:

E_f is the Young's modulus of the foundation material;

E_g is the representative Young's modulus for the ground beneath the foundation (i.e. the value of Young's modulus at a depth equal to the radius of a circular footing or half the foundation width);

ν_g is Poisson's ratio of the ground;

ν_f is Poisson's ratio of the foundation material;

B is the foundation width;

L is the foundation length; and

D_f is the foundation depth (thickness).

(3) A foundation may be assumed to be rigid when K_s is greater than 10 and flexible when K_s is less than 0,05.

NOTE For K_s values between these values the relative deflection and the bending moments in the foundation are a function of K_s.

(2) When designing a spread foundation as a beam resting on a series of springs, the subgrade modulus k may be determined from Formula (B.14):

$$k = \frac{0.65E'}{B(1-\upsilon^2)} \tag{B.14}$$

where:

E' is Young's modulus of the ground;

υ is Poisson's ratio of the ground; and

B is the foundation width.

B.15 Linear elastic spring stiffnesses of surface foundation

(1) Ground reaction may be represented by springs for all degrees of freedom.

NOTE 1 In general, the springs are non-linear and frequency dependent.

NOTE 2 A rigid foundation on deformable ground has 6 degrees of freedom, 3 translational (in x, y, z directions) and 3 rotational (r_x, r_y, r_z about the x, y and z axes).

(2) For certain foundation shapes (circle, strip, rectangle) and ground profiles (for example, homogeneous half-space and soil layer on rock), the stiffness coefficients may be obtained from available solutions based on linear elasticity.

(3) The linear elastic spring stiffnesses of a rectangular foundation on the surface of a homogeneous half-space may be calculated using Formulae (B.15) to (B.20).

$$K_{\mathrm{yy}} = \frac{GL}{2-\nu}\left[2+2{,}5\left(\frac{B}{L}\right)^{0{,}85}\right] \tag{B.15}$$

$$K_{\mathrm{xx}} = \frac{GB}{2-\nu}\left[1{,}2+3{,}3\left(\frac{L}{B}\right)^{0{,}65}\right] \tag{B.16}$$

$$K_{\mathrm{zz}} = \frac{GL}{1-\nu}\left[0{,}73+1{,}54\left(\frac{B}{L}\right)^{0{,}75}\right] \tag{B.17}$$

$$K_{\mathrm{rx}} = \frac{GB^3}{8(1-\nu)}\left[0{,}4+3{,}2\left(\frac{L}{B}\right)\right] \tag{B.18}$$

$$K_{\mathrm{ry}} = \frac{GB^3}{8(1-\nu)}\left[3{,}6\left(\frac{L}{B}\right)^{2{,}4}\right] \tag{B.19}$$

$$K_{\mathrm{rz}} = \frac{GB^3}{8}\left[4{,}1+4{,}2\left(\frac{L}{B}\right)^{2{,}45}\right] \tag{B.20}$$

where:

G is the ground shear modulus;

B is the foundation width (smallest dimension);

L is the foundation length (largest dimension);

K_{xx} is the stiffness coefficient in the horizontal X direction;

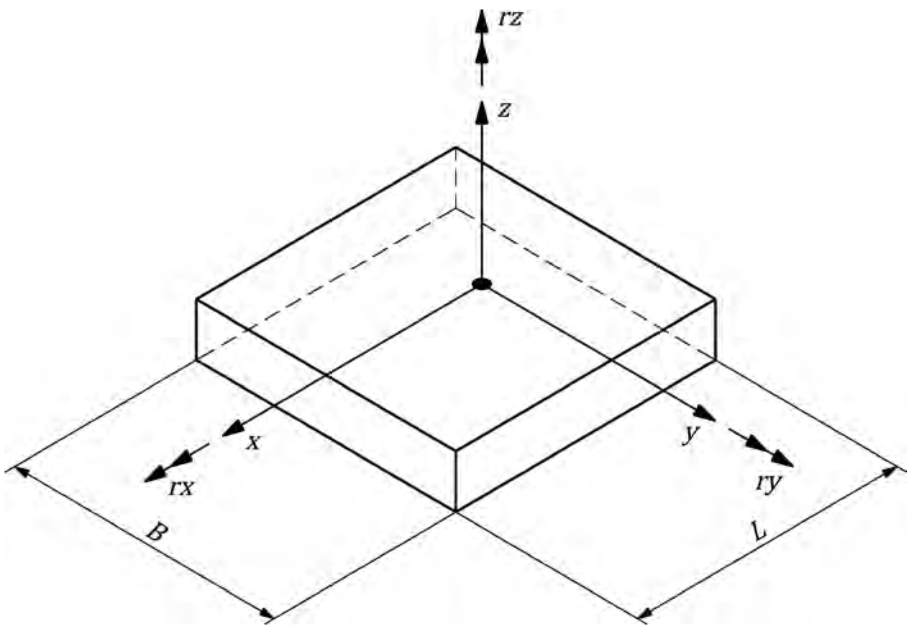
K_{yy} is the stiffness coefficient in the horizontal Y direction;

K_{zz} is the stiffness coefficient in the vertical Z direction;

K_{ry} is the rocking stiffness coefficient around the horizontal X direction;

K_{rz} is the torsional stiffness coefficient around the vertical Z direction;

ν is the ground Poisson's ratio.



Key

B Width of the foundation

Figure B.4 — Definition of the degrees of freedom

Annex C
(informative)

Piled foundations

C.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to Clause 6 regarding piled foundations.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

C.2 Scope and field of application

(1) This Informative Annex covers:

- examples of pile types in different classes;
- method for the determination of the coefficient of variation;
- calculation model for pile bearing capacity based on ground parameters;
- calculation model for pile bearing capacity based on CPT profiles;
- calculation model for pile bearing capacity based on PMT profiles;
- calculation model for pile bearing capacity based on empirical tables;
- calculation model for downdrag (vertical ground movements);
- calculation model for a pile block subject to axial tension loads;
- calculation model for single pile settlement using load transfer functions;
- calculation model for single pile lateral displacement using load transfer functions;
- calculation for model for buckling and second order effects.

C.3 Examples of pile types

NOTE Table C.1 give examples of pile types classified according to Table 6.1 .

Table C.1 — Examples of pile types in different classes

Pile type	Class	Example pile types
Displacement piles	Full displacement	Driven cast-in-place concrete piles; Solid section precast concrete piles; Driven closed-ended tubular steel piles; Driven closed-ended tubular precast concrete piles; Driven open-ended tubular steel piles (plugged); Driven open-ended tubular precast concrete piles (plugged) Driven steel H-section piles (plugged); Driven micropiles; Driven timber piles; Cast-in-place concrete screw piles.
	Partial displacement	Driven open-ended tubular steel piles (unplugged); Driven steel H-section piles (unplugged); Driven and grouted steel H-section piles; Driven steel sheet piles; Cast-in-place concrete screw piles; Continuous (flight auger) helical displacement piles; Displacement auger piles; Drilled or bored pressure-grouted micropiles.
Replacement piles	Replacement	Bored cast-in-place piles installed using continuous flight auger; Cased continuous flight auger piles; Bored cast-in-place concrete piles with permanent casing; Bored cast-in-place concrete piles with temporary casing; Bored cast-in-place concrete piles with slurry or polymer support; Bored cast-in-place concrete piles excavated without support; Bored or drilled steel tubular piles; Bored ribbed piles; Drilled or bored micropiles; Caissons excavated by hand or by machine; Barrettes; Diaphragm walls; Grouted piles or battetts.
Piles not listed above		Steel helical piles; Compressed-air driven piles

C.4 Pile shaft resistance based on ground parameters.

(1) For total stress analysis, the representative value of unit shaft friction, $q_{s,rep}$ in fine soils and fills may be derived from Formula (C.1):

$$q_{s,rep} = \alpha c_{u,rep} \tag{C.1}$$

where:

$c_{u,rep}$ is the representative undrained shear strength of the ground;

α is an adhesion factor for piles in soil.

NOTE 1 The adhesion factor α is an empirical coefficient that depends on the strength of the soil, effective overburden pressure, pile type, and method of execution.

NOTE 2 The value of α typically ranges between 0.15 and 1.0 for low strength normally consolidated fine soils, and between 0.4 and 0.75 for high-strength over-consolidated fine soils.

(2) The value of $q_{s,rep}$ in weak and medium strong rock masses may be derived from Formula (C.2):

$$\frac{q_{s,rep}}{p_{ref}} = k_1 \left(\frac{q_{u,rep}}{p_{ref}}\right)^{k_2} \tag{C.2}$$

where:

$q_{u,rep}$ is the representative unconfined compressive strength of the rock mass;

p_{ref} is a reference pressure (= 100 kPa);

k_1, k_2 are empirical coefficients.

NOTE 1 The value of k_1 typically varies between 0.7 and 2.1 for cemented rocks and 1.0-1.29 for soft rocks.

NOTE 2 The value of k_2 typically varies between 0.57 and 0.61 but is commonly taken as 0.5.

(3) Under effective stress conditions, the value of $q_{s,rep}$ in fine soils, fills, and rock mass may be derived from Formula (C.3):

$$\overline{q_{s,rep}} = K_s \overline{\sigma'_v} tan\delta_{rep} = \beta \overline{\sigma'_v} \tag{C.3}$$

where:

σ'_v is the vertical effective stress at the depth being considered;

K_s is an earth pressure coefficient;

δ_{rep} is the representative angle of interface friction between the pile and the ground;

β is an empirical coefficient (= $K_s \tan\delta_{rep}$);

– denotes the average value along the pile shaft.

NOTE 1 The earth pressure coefficient depends on the strength of the soil, pile type, method of execution, and distance above the pile base.

NOTE 2 The value of K_s typically ranges between 0.5 and 0.9 for replacement piles and between 0.8 and 1.2 (or higher) for displacement piles.

NOTE 3 The value of δ_{rep} is typically taken as φ_{rep} for cast-in-place concrete piles and between 0.67φ_{rep} and 0.75φ_{rep} for precast concrete and steel piles, where φ_{rep} is the representative value of the so'l's angle of internal friction.

NOTE 4 For fine soils or fills, β is typically between 0.2 and 0.3. For coarse soils and fills, β increases with density index and is typically between 0.5 and 2.0.

C.5 Pile base resistance based on ground parameters

(1) For total stress analysis, the representative value of unit base resistance $q_{b,rep}$ in fine and coarse soils, and fills may be derived from Formula (C.4):

$$q_{b,rep} = N_c c_{ub,rep} + \sigma_{vb} \tag{C.4}$$

where:

$c_{ub,rep}$ is the representative undrained shear strength of the ground at the pile base;

N_c is a bearing factor;

σ_{vb} is the total overburden pressure at the depth of the pile base.

NOTE The value of N_c typically ranges between 6 and 10, although N_c = 9 is commonly used.

(2) When the self-weight of the pile is not included as a separate action, the term σ_{vb} in Formula (C.4) should be omitted.

(3) The value of $q_{b,rep}$ in very weak and weak fine-grained rock masses may be derived from Formula (C.5):

$$\frac{q_{b,rep}}{p_{ref}} = k_3 \left(\frac{q_{u,rep}}{p_{ref}}\right)^{k_4} \tag{C.5}$$

where:

$q_{u,rep}$ is the representative unconfined compressive strength of the rock mass;

p_{ref} is a reference pressure (= 100 kPa);

k_3, k_4 are empirical coefficients.

NOTE 1 The value of k_3 typically about 15 for cemented rocks.

NOTE 2 The value of k_4 typically varies between 0.4 and 0.6 but is commonly taken as 0.5.

(4) For effective stress analysis, the value of $q_{b,rep}$ i may be derived from Formula (C.6):

$$q_{b,rep} = q'_{b,rep} + u_b = N_q \sigma'_{vb} + (\sigma'_{vb} + u_b) \tag{C.6}$$

where:

σ'_{vb} is the vertical effective stress at the depth of the pile base;

N_q is a bearing factor;

u_b is the pore water pressure at the depth of the pile base.

NOTE The bearing factor depends on the angle of internal friction of the ground, density index, and vertical effective stress at the pile base.

(5) When the self-weight of the pile is not included as a separate action, the term ($\sigma'_{vb} + u_b$) in Formula (C.6) should be omitted.

C.6 Axial pile resistance based on CPT profiles

(1) The representative value of unit shaft $q_{s,rep}$ in coarse soils and fills may be derived from Formula (C.7):

$$q_{s,rep} = c_s q_c \tag{C.7}$$

where:

q_c is the measured cone resistance (Mpa);

c_s is an empirical cone factor for shaft resistance.

NOTE 1 If $q_c \geq 12$ Mpa over a continuous depth interval ≥ 1 m, then q_c is limited to 15 Mpa over this interval. If $q_c \geq 12$ Mpa over an interval < 1 m, then it is limited to 15 Mpa.

NOTE 2 The empirical factor c_s depends on ground and pile types (see Table C.2 and Table C.3).

Table C.2 — Typical values of cs and cb for sands

Pile type	c_b	c_s
Driven precast concrete pile or closed ended steel pipe pile	0.70	0.010[a]
Cast in place piles made by driving a steel tube with a closed end, with the steel tube being reclaimed during concreting	0.70	0.014[a]
Driven open ended steel tube or H-pile	0.70	0.006[a]
Cast-in-place with temporary casing on top of a screw pile-tip, with the casing being removed and the screw tip remaining in the ground	0.63	0.009[a]
Continuous flight auger pile	0.56	0.006[a]
Bored pile	0.35	0.006[a]
[a] Values given for fine to coarse sands. For very coarse sands, reduce the values by 25 % and for gravels by 50 %		

Table C.3 — Typical values of cs for piles in clays, silts, and peats

Soil type	Cone resistance q_c (Mpa)	c_s
Clay	≥ 2.5	0.03
	2.0-2.5	0.02 (q_c – 1.0)[a]
	< 2.0	0.02
Silt	---	min(f_r, 0.025)[b]
Peat	---	0

[a] q_c entered in Mpa

[b] f_r = measured (uncorrected) friction ratio

(2) The representative value of unit base resistance $q_{b,rep}$ in coarse soils and fills may be derived from Formula (C.8):

$$q_{\text{b,rep}} = 0.5 c_{\text{b}} k_{\text{shape}} \left(\frac{q_{\text{c,I,mean}} + q_{\text{c,II,mean}}}{2} + q_{\text{c,III,mean}} \right) < 15 MPa \quad \text{(C.8)}$$

where:

$q_{c,X,mean}$ is the mean measured cone resistance in zone X (= I, II, or III), as defined in Figure C.1;

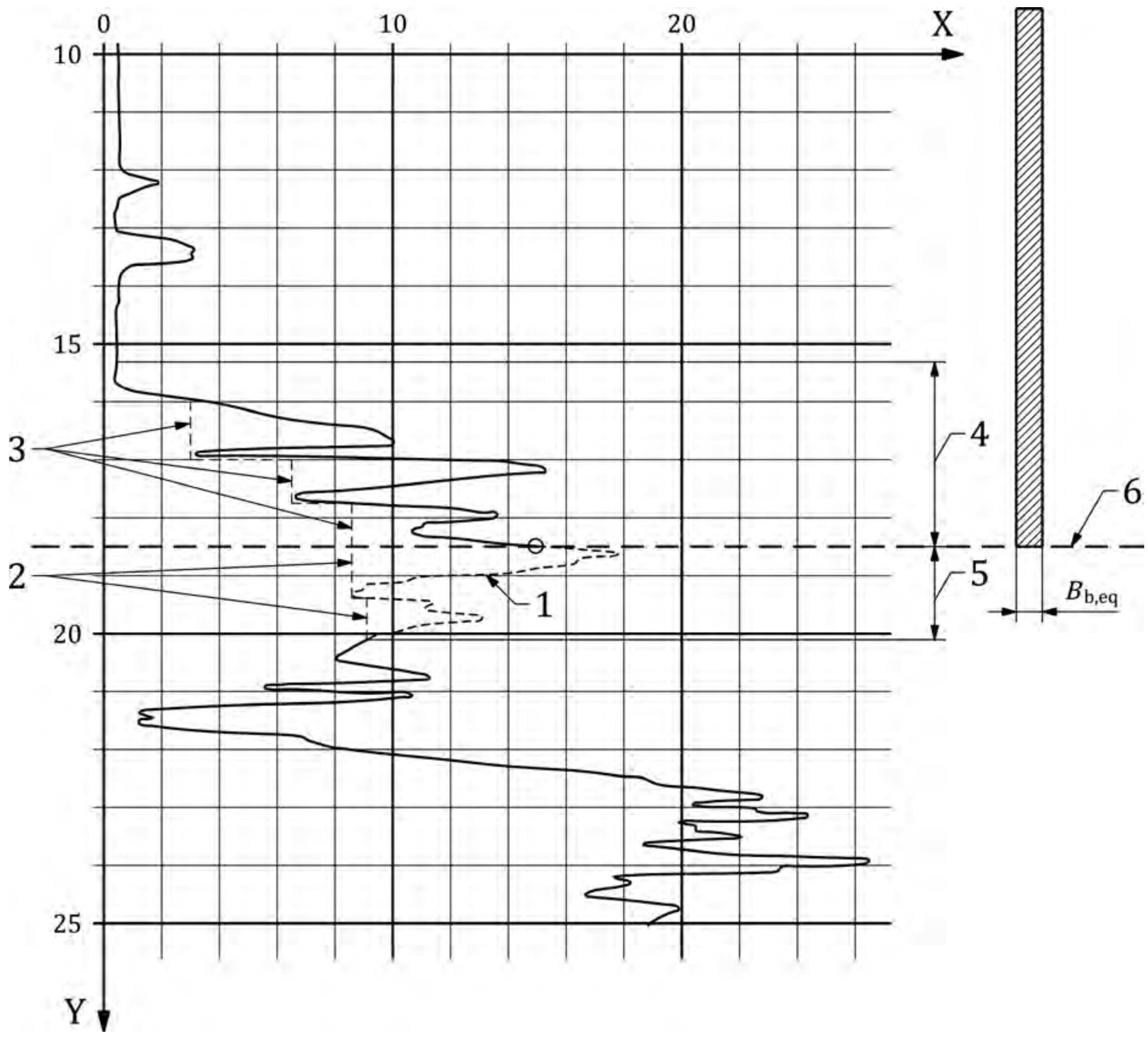
c_b is an empirical cone factor for base resistance;

k_{shape} is a factor (see Figure C.2) that accounts for the relative size of the pile base $B_{b,eq}$ and shaft $B_{s,eq}$ and the thickness h of any base plate (see Figure C.3.)

NOTE 1 The empirical factor c_b depends on ground and pile types (see Table C.2).

NOTE 2 Figure C.1 gives the definition for zones I, II, and III and Figure C.2 a chart to determine k_{shape}.

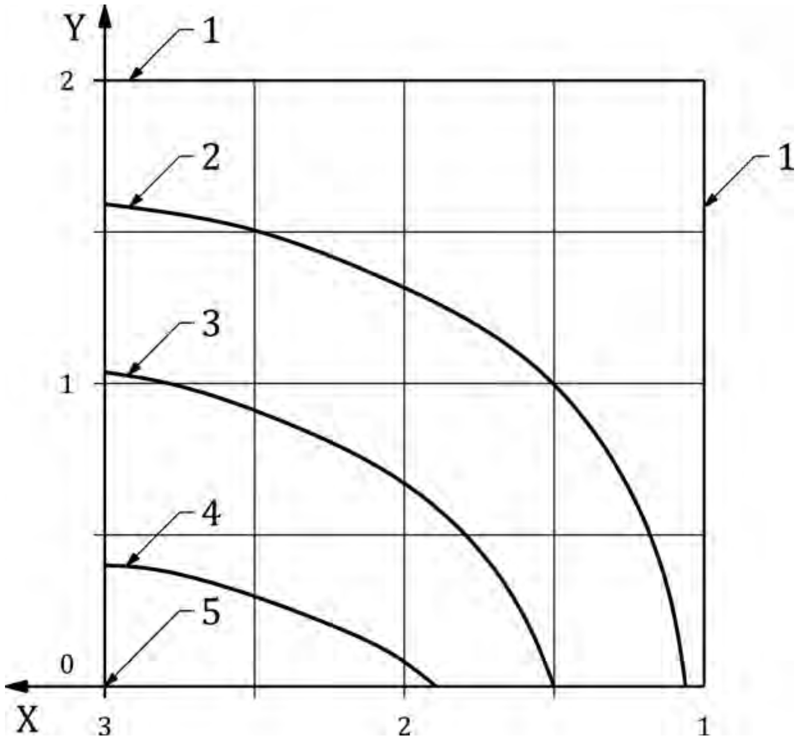
NOTE 3 In Figure C.3 a chart to determine h is given.



Key

X	q_c (Mpa)
Y	z (m)
1	zone I
2	zone II
3	Zone III
4	$8B_{b,eq}$
5	0.7 to $4B_{b,eq}$
6	pile base level
$B_{b,eq}$	equivalent pile diameter

Figure C.1 — Definition of zones I, II, and III



Key

X	$B_{b,eq}{}^2/B_{s,eq}{}^2$	3	k_{shape}=0.8
Y	$h/B_{b,eq}$	4	k_{shape}=0.7
1	k_{shape}=1.0	5	k_{shape}=0.6
2	k_{shape}=0.9		

Figure C.2 — Chart to determine k_{shape}

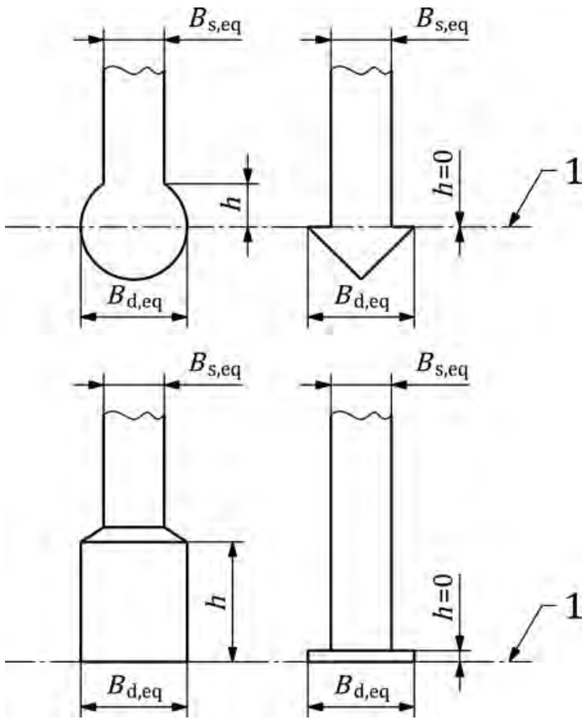


Figure C.3 — Chart to determine *h*

(3) For piles installed by driving or vibration into over-consolidated soils, the value of q_c in Formulae (C.7) and (C.8) should be multiplied by $\sqrt{(1/OCR)}$, where OCR is the overconsolidation ratio of the soil.

(4) For piles installed from an excavated depth that is deeper than that from which the cone penetration tests were executed, the value of q_c in in Formulae (C.7) and (C.8) should be reduced accordingly.

C.7 Axial pile resistance from PMT profiles

(1) The representative value of unit shaft friction $q_{s,rep}$ may be derived from Formula (C.9):

$$q_{s,rep} = \min\left(k_{s,PMT}(a_{PMT}p_{LM}^{*} + b_{PMT})(1 - e^{-c_{PMT}p_l}); q_{s,max}\right) \quad (C.9)$$

where:

$k_{s,PMT}$	is a dimensionless parameter that depends on pile type and ground type;
p_{LM}^{*}	is the PMT net limit pressure (Mpa) at a depth z; and
a_{PMT}, b_{PMT}, c_{PMT}	are parameters that depend on ground type.

NOTE 1 Values of $k_{s,PMT}$ are given in Table C.4 for selected pile types.

NOTE 2 Values of a_{PMT}, b_{PMT}, and c_{PMT} are given in Table C.5 for selected pile types.

NOTE 3 Values of $q_{s,max}$ are given in Table C.6 for selected pile types.

Table C.4 — Values of $k_{s,PMT}$ for selected pile types

Class	Installation technique	Ground type				
		Fine soil	Coarse soil	Chalk	Marl/marly limestone	Weathered rock masses
1	Mud bored piles/barrettes	1.25	1.4	1.8	1.5	1.6
	Bored (temporary casing)	1.25	1.4	1.7	1.4	—
2	Continuous flight auger bored	1.5	1.8	2.1	1.6	1.6
3	Cast in situ screwed	1.9	2.1	1.7	1.7	—
4	Driven precast or prestressed concrete	1.1	1.4	1	0.9	—
	Closed-ended driven steel	0.8	1.2	0.4	0.9	—
5	Open-ended driven steel	1.2	0.7	0.5	1	1
6	Driven H-shaped	1.1	1	0.4	1	0.9
7	Driven sheet piles	0.9	0.8	0.4	1.2	1.2
8	Injected pile/micro-pile III	2.7	2.9	2.4	2.4	2.4

Table C.5 — Values of a_{PMT}, b_{PMT}, and c_{PMT} for selected pile types

Parameter	Ground type				
	Fine soil	Coarse soil	Chalk	Marl/marly limestone	Weathered rock masses
a_{PMT}	0.003	0.010	0.007	0.008	0.010
b_{PMT}	0.04	0.06	0.07	0.08	0.08
c_{PMT}	3.5	1.2	1.3	3.0	3.0

Table C.6 — Values of $q_{s,max}$ (in kPa) for selected pile types

Class	Installation technique/ parameter	Ground type				
		Fine soil	Coarse soil	Chalk	Marl/marly limestone	Weathered rock masses
1	Mud bored piles/barrettes	90	90	200	170	200
	Bored (temporary casing)	90	90	170	170	-
2	Continuous flight auger bored	90	170	200	200	200
3	Cast in situ screwed	130	200	170	170	-
4	Driven precast or prestressed concrete	130	130	90	90	-
	Closed-ended driven steel	90	90	50	90	-
5	Open-ended driven steel	90	50	50	90	90
6	Driven H-shaped	90	130	50	90	90
7	Driven sheet piles	90	50	50	90	90
8	Injected pile/micro-pile	200	380	320	320	320

(2) The representative value of unit base resistance $q_{b,rep}$ may be derived from Formula (C.10):

$$q_{b,rep} = k_{b,PMT}\frac{1}{z_1 + 3z_2}\int_{-z_1}^{3z_2} p^*_{LM}(z)dz \tag{C.10}$$

where:

$k_{b,PMT}$ is a dimensionless parameter that depends on pile type and ground type;

$p^*_{LM}(z)$ is the netPMT limit pressure at a depth z;

z_1 is a depth equal to min(z_2, h);

z_2 is a depth equal to min(D_b/2, 0.5 m);

D_b is the base diameter of the pile;

h is the embedment depth of the pile in the bearing geotechnical unit.

NOTE Values of $k_{b,PMT}$ are given in Table C.7 for selected pile types.

Table C.7 — Values of $k_{b,PMT}$ for selected pile types

Class	Installation technique	Ground type				
		Fine soil	Coarse soil	Chalk	Marl/marly limestone	Weathered rock masses
1	Bored	1.15	1.1	1.45	1.45	1.45
2	Continuous flight auger	1.3	1.65	1.6	1.6	2.0
3	Cast-in-place screwed	1.55	3.2	2.35	2.10	2.10
4	Closed-ended driven	1.35	3.1	2.30	2.30	2.30
5	Open-ended driven	1.0	1.9	1.4	1.4	1.2
6	Driven (H-shaped)	1.20	3.10	1.7	2.2	1.5
7	Driven (sheet)	1.0	1.0	1.0	1.0	1.2
8	Micropile[a]	1.15	1.1	1.45	1.45	1.45

[a] For micropiles, base resistance is usually not taken into account

C.8 Axial pile resistance based on empirical tables

(1) The representative value of unit shaft resistance $q_{s,rep}$ for bored piles in soils may be determined from Table C.8.

NOTE The values of $q_{s,rep}$ and $q_{b,rep}$ given in this sub-clause are based on an empirical database of results from predominantly static pile load tests. The lower bound of the ranges specified is a 10 % quantile whereas the upper bound is a 50 % quantile.

(2) The 10 % quantile values given in Table C.8 should be used, unless site-specific pile load testing confirms the use of the 50 % quantile values.

Table C.8 — Representative values of unit shaft resistance $q_{s,rep}$ for bored piles in soils

Fine soils		Coarse soils	
Undrained shear strength c_u (kPa)	$q_{s,rep}$ (kPa)[a, b]	Mean cone resistance q (Mpa)	$q_{s,rep}$ (kPa)[a,b]
60	30-40	7.5	55-80
150	50-65	15	105-140
≥ 250	65-85	≥ 25	130-170

[a] The lower value represents the 10 % quantile and the upper value the 50 % quantile

[b] Intermediate values can be obtained by linear interpolation

(3) The values given in Table C.9 should be reduced by 25 % for bored piles with enlarged bases.

Table C.9 — Representative values of unit base resistance $q_{b,rep}$ for bored piles in soils

Fine soils				**Coarse soils**			
c_u **(kPa)**	$q_{b,rep}$ **(kPa)**[a,b] **for s/D equal to** [c] **…**			q_c **(Mpa)**	$q_{b,rep}$ **(kPa)**[a,b] **for s/D equal to** [c] **…**		
	2 %	**3 %**	**10 %**		**2 %**	**3 %**	**10 %**
100	350-450	450-550	800-1000	7.5	550-800	700-1050	1600-2300
150	600-750	700-900	1200-1500	15	1050-1400	1350-1800	3000-4000
≥ 250	950-1200	1200-1450	1600-2000	≥ 25	1750-2300	2250-2950	4000-5300

a The lower value represents the 10 % quantile and the upper value the 50 % quantile.
B Intermediate values can be obtained by linear interpolation
c *s* = pile head settlement; *D* = pile diameter

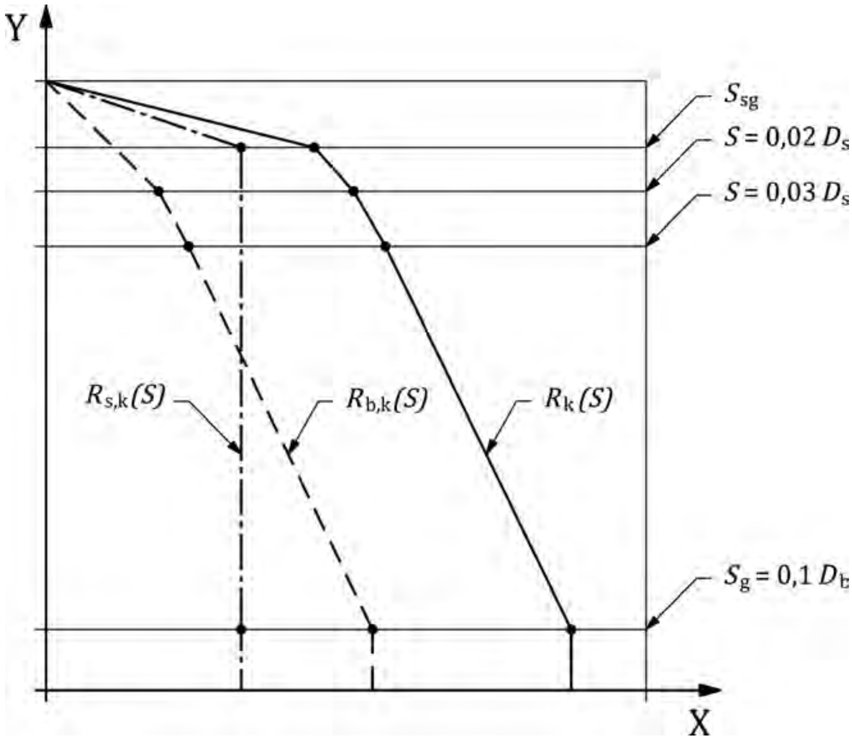
(4) The load-settlement curve for bored piles in soils may be determined from Figure C.4, with the settlement s_{sg} given by Formula (C.11):

$$s_{sg} = k_{sg}R_{sk} + 5mm \leq 30mm \tag{C.11}$$

R_{sk} is the shaft resistance calculated from Table C.8;

k_{sg} is a factor equal to 5 mm/MN.

NOTE Figure C.4 gives Load-displacement curves for bored piles



Key

X Pile capacity

Y Pile head settlement s

Figure C.4 — Load-displacement curves for bored piles

C.9 Downdrag due to vertical ground movements

C.9.1 General

(1) The drag force caused by downdrag should be classified as a permanent action.

NOTE 1 'Downdrag' is the term used to describe relative movement between settling ground and the pile shaft. A drag force occurs where the ground settlement exceeds the pile settlement.

NOTE 2 Pile settlement due to downdrag continues until the combination of imposed actions from the structure and the drag force come into equilibrium with the mobilised pile resistance.

(2) Potential downdrag should be included in the verification of serviceability limit states.

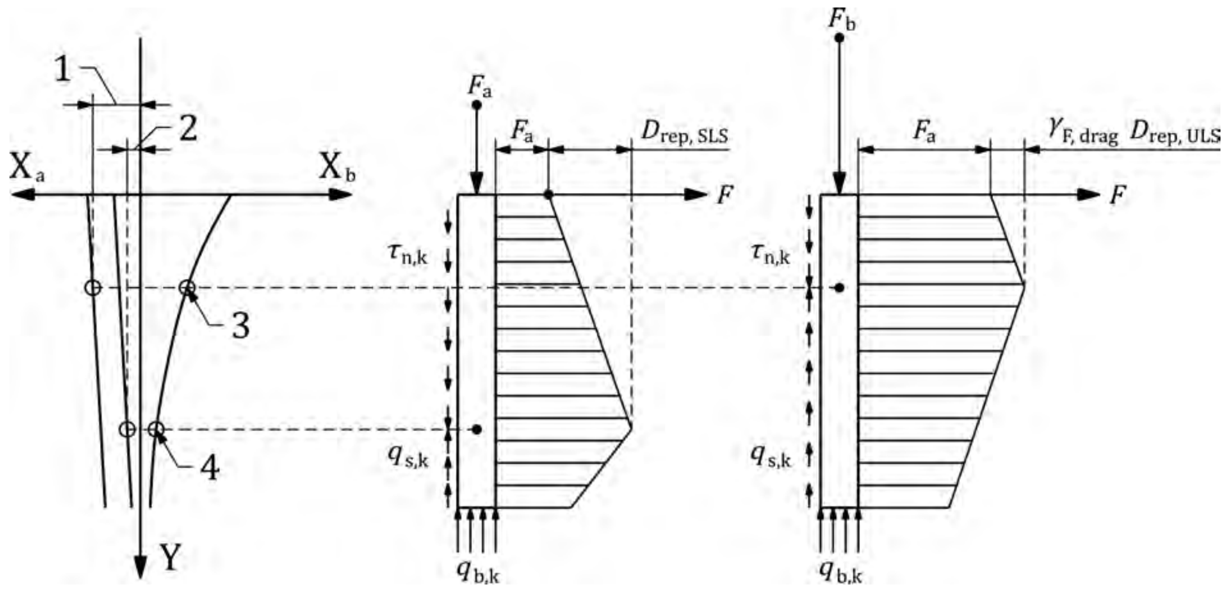
(3) Potential downdrag should be included in the verification of ultimate limit states when the drag force exceeds any variable compressive actions applied to the pile.

C.9.2 Rigorous interaction model for downdrag

(1) The calculation model shown in Figure C.5 may be used to calculate the drag force owing to potential downdrag.

NOTE 1 In this model, the neutral point marks the boundary between forces that act downwards and upwards acting along the pile shaft. The neutral point differs between ULS and SLS conditions.

NOTE 2 Figure C.5 illustrated the force distribution for assessment of dragforce on a pile subjected to downdrag.



Key

X s_{pile}

Y s_{ground}

1 $s_{pile,SLS} = f\left(\sum_{i\geq1} G_{k,i} + \sum_{j\geq1} \psi_{2,j} Q_{k,j}\right)$

2 $s_{pile,ULS} = f\left(\sum_{i\geq1} \gamma_{G,i} G_{k,i} + \sum_{j\geq1} \gamma_{Q,j} \psi_{0,j} Q_{k,j}\right)$

3 neutral point (ULS)

4 neutral point (SLS)

Figure C.5 — Force distribution for assessment of drag force on a pile subject to downdrag

NOTE 3 The neutral point will be at a different level for SLS or ULS conditions, but in both cases, corresponds to the level at which the settlement of the pile s_{pile} and the surrounding ground s_{ground} are equal. For the ULS case, the neutral point will be at a higher level compared to that for the SLS case.

(2) The settlement of the ground at any particular time s_{ground} should be estimated from anticipated changes in effective stress, ground stiffness, and depth of compressible ground.

(3) The ground settlement of should include immediate and primary consolidation, together with potential secondary consolidation (creep).

(4) The settlement of the pile s_{pile} may be estimated using analytical models, empirical relationships, numerical analysis, or other suitable method that take account of the stress distribution.

(5) As an alternative to (2) and (4), the values of s_{ground} and s_{pile} may be determined by an interaction analysis to find the depth of the neutral point Ldd where $s_{pile} = s_{ground}$.

(6) In addition to prEN 1990-1:2021, 8.4.3.4, the design value of the compressive action applied to the pile at the serviceability limit state should be determined from Formula (C.12):

$$F_{\mathrm{cd,SLS}} = \max \begin{cases} \sum_{i\geq 1} G_{\mathrm{k,i}} + Q_{\mathrm{k,1}} + \sum_{j>1} \psi_{2,j} Q_{\mathrm{k,j}} \\ \sum_{i\geq 1} G_{\mathrm{k,i}} + D_{\mathrm{rep,SLS}} + \sum_{j\geq 1} \psi_{2,j} Q_{\mathrm{k,j}} \end{cases} \tag{C.12}$$

where:

$G_{\mathrm{k,i}}$ is the *i*-th characteristic permanent action;

$Q_{\mathrm{k,1}}$ is the leading characteristic variable action;

$Q_{\mathrm{k,j}}$ is the *j*-th accompanying characteristic variable action;

$D_{\mathrm{rep,SLS}}$ is the representative drag force at the serviceability limit state;

$\psi_{2,j}$ is a combination value for accompanying variable actions.

NOTE Formula (C.12) is a modification of the quasi-permanent combination of actions given in prEN 1990-1.

(7) In addition to prEN 1990-1:2021, 8.4.3.2, the design value of the compressive action applied to the pile at the ultimate limit state should be determined from Formula (C.13):

$$F_{\mathrm{cd,ULS}} = \max \begin{cases} \sum_{i\geq 1} \gamma_{\mathrm{G,i}} G_{\mathrm{k,i}} + \gamma_{\mathrm{Q}} Q_{\mathrm{k,1}} + \sum_{j>1} \gamma_{\mathrm{Q,j}} \psi_{0,j} Q_{\mathrm{k,j}} \\ \sum_{i\geq 1} \gamma_{\mathrm{G,i}} G_{\mathrm{k,i}} + \gamma_{\mathrm{F,drag}} D_{\mathrm{rep,ULS}} + \sum_{j\geq 1} \gamma_{\mathrm{Q,j}} \psi_{0,j} Q_{\mathrm{k,j}} \end{cases} \tag{C.13}$$

$D_{\mathrm{rep,ULS}}$ is the representative drag force over the depth of ground above the neutral plane under ultimate conditions;

$\gamma_{\mathrm{G,i}}$, $\gamma_{\mathrm{Q,j}}$ are partial factors applied to permanent and variable actions, respectively;

$\psi_{0,j}$ is a combination factor for accompanying variable actions;

$\gamma_{\mathrm{F,drag}}$ is a partial factor dependent on the assumptions regarding ground parameters and the particular method of analysis used to determine $D_{\mathrm{rep,ULS}}$.

C.9.3 Simplified approach for calculating downdrag

(1) For simple cases, approximate approaches may be used.

(2) If the pile settlement s_{pile} at the ultimate limit state is greater than the settlement of the surrounding soil or fill s_{ground}, the neutral point may be assumed to be located at the ground surface.

(3) In this case of (2) the drag force may be disregarded for the verification of the ultimate limit state.

(4) If the pile settlement s_{pile} at the ultimate limit state is much smaller than the settlement of the surrounding soil or fill s_{ground}, the neutral point may be assumed to be located at the base of the settling soil or fill layer.

(5) For (4) the representative value of the drag force D_{rep} may be taken as an upper (superior) value determined for the full thickness of the settling soil or fill.

(6) For SLS conditions, the neutral plane may be assumed to be located at the base of the settling fill or soil layer.

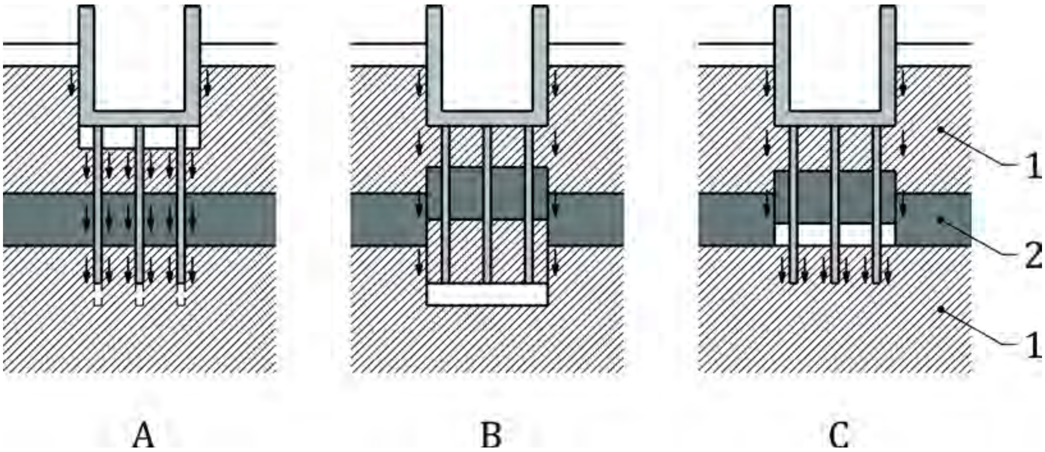
(7) Representative values for the drag force D_{rep} should be determined for the full thickness of the settling soil or fill.

C.9.4 Representative downdrag

(1) The representative value of downdrag within the settling ground may be determined from C.4, using upper (superior) values of ground strength properties.

C.10 Pile groups subject to axial tension

NOTE Possible mechanisms for groups of tension piles in layered soils are illustrated in Figure C.6.



Key

A Pull-out from ground

B Lift-off a block of soil

C Combined pull-out and lift-off

Figure C.6 — Possible mechanisms for groups of tension piles in layered soils

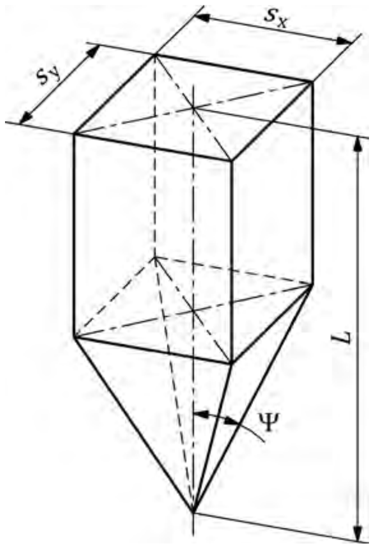
(1) For the evaluation of the block failure, the representative weight of the soil block surrounding an individual pile $W_{block,rep}$ (see Figure C.7) may be determined from Formula (C.14):

$$W_{\mathrm{block,rep}} = n_{\mathrm{z}} \left[s_{\mathrm{x}} s_{\mathrm{y}} \left(L - \frac{1}{3} \sqrt{\left({s_{\mathrm{x}}}^2 + {s_{\mathrm{y}}}^2 \right)} \cot \varphi_{\mathrm{rep}} \right) \right] \eta_z \gamma \qquad \text{(C.14)}$$

where:

L	is the embedded depth of the pile;
s_x, s_y	are the grid spacings of the piles in the group;
n_z	is the number of piles in the group;
φ	is the representative value of the internal friction angle of the soil block;
η_z	is a coefficient commonly taken as 0.8;
γ	is the weight density of the soil block.

NOTE Figure C.7 illustrates block failure of single pile.



Key

s_x, s_y are the grid spacings of the piles in the group;

φ is the representative value of the internal friction angle of the soil block;

Figure C.7 — Block failure of a single pile under tension as part of a pile group

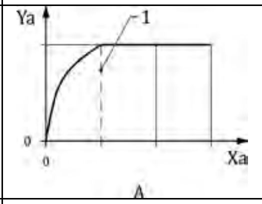
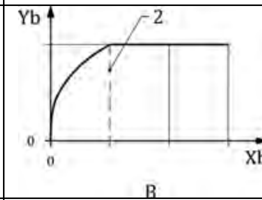
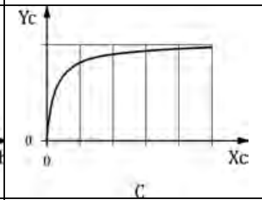
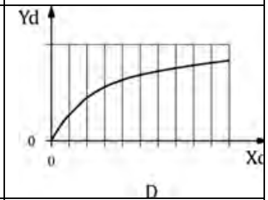
C.11 Calculation model for single pile settlement using load transfer functions

(1) Settlement of single piles may be determined using load transfer functions.

NOTE Examples of load transfer functions are given in Table C.10.

(2) Load transfer functions used for the assessment of pile settlement should be calibrated with comparable experience.

Table C.10 — Example load transfer functions

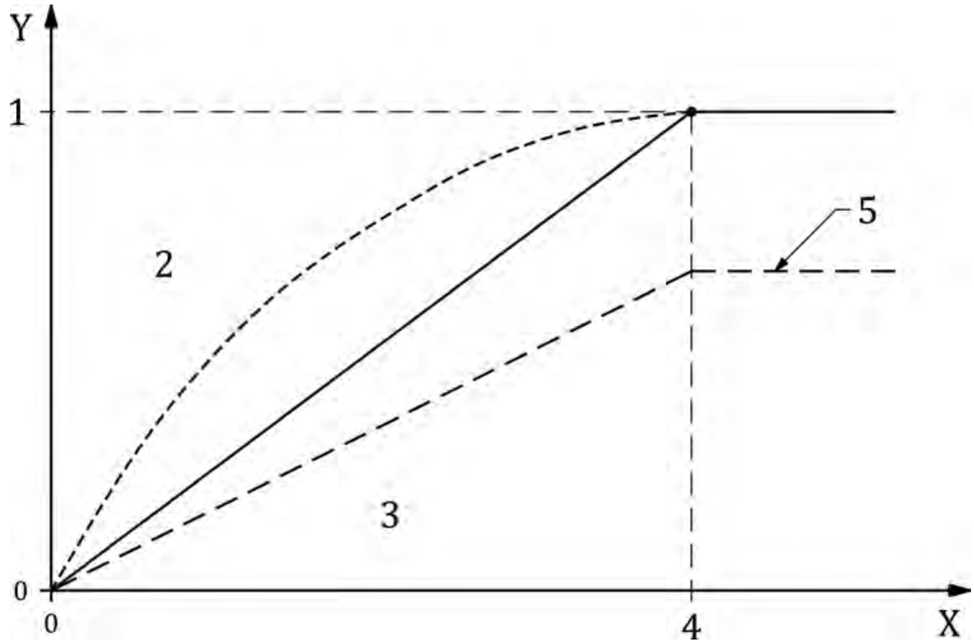
Curve	Cubic root		Hyperbolic	
	Shaft	**Base**	**Shaft**	**Base**
Shape				
	$Y_a = q_s$ $X_a = S_s$ $1 = S_{s,max}$	$Y_b = q_b$ $X_b = S_b$ $2 = S_{b,lim}$	$Y_c = q_s$ $X_c = S_s$	$Y_d = q_b$ $X_d = S_b$ 4
q/qult	$\sqrt[3]{\frac{s_s}{s_{s,max}}}$	$\sqrt[3]{\frac{s_b}{s_{b,max}}}$	$\frac{s_s}{M_s B + s_s}$	$\frac{s_b}{M_b B + s_b}$
Deformation parameter	$s_{s,max}$	$s_{b,max}$, depending on diameter	M_s	M_b
Initial slope	∞	∞	$q_{s,ult}/M_s B$	$q_{b,ult}/M_b B$

C.12 Calculation model for single pile lateral displacement using load transfer functions

C.12.1 General

(1) The behaviour of transversally loaded piles may be considered by a bilinear model, representing the non-linear soil resistance as shown in Figure C.8.

NOTE Figure C.8 illustration of the bilinear model for transversally loaded piles.



Key

X y, transversal deflection;

Y p, lateral pressure:

1 p_f, lateral pressure of the ground at failure

2 p_{fd}, design value of the lateral pressure of the ground at failure

3 y_f, transversal deflection of the pile at failure

4 Dashed line – soil resistance defined by Formula (C.15)

5 Dashed curve – actual soil resistance

Figure C.8 — Model of soil resistance as a function of the transversal deflection of a pile

(2) The lateral pressure may be determined by Formula (C.15)

$$p = min\left(\frac{p_f}{y_f} \cdot y; p_f\right) \tag{C.15}$$

where

p_f is the lateral pressure of the ground at failure;

p is the lateral pressure;

y_f is the transversal deflection off the pile.

(3) Specific non-linear soil models may be used for buckling.

NOTE A non-linear soil model is given in prEN 1990-1 and provides information about the soil resistance p at small transversal deflections y.

(4) For design situations where seismic loading potentially results in loss of shear strength in soils susceptible to liquefaction, p_f should be assumed to be equal to zero.

NOTE Examples of design situation in (4) is e.g. saturated sand of loose density and collapsible fine-grained soils.

C.12.2 P-y curves from undrained soil properties

(1) The design value of the ultimate transversal ground resistance during short-term loading in undrained situations may be expressed by $p_{fd} = 9 \cdot c_{ud}$.

(2) To account for long-term deformations resulting from creep of a highly viscous soil, $p_{fd} = 6 \cdot c_{ud}$. May be applied.

NOTE Examples of highly viscous soils is low strength clay or organic clay.

(3) A weighted average of the undrained soil response may be applied in the case of combined long-term and short-term loads.

(4) To account for limited soil resistance to close the ground surface p_{fd} may be determined using formula (C.16):

$$p_{f,d} = c_{ud} \cdot \left(2 + \frac{2}{3} \cdot \frac{z}{B}\right) + \sigma'_z \quad \text{(C.16)}$$

where

$p_{f,d}$	is the design lateral pressure of the ground at failure;
c_{ud}	is the design undrained shear strength of the ground;
σ'_z	is the effective vertical stress of the soil at the depth z;
B	is the pile diameter
z	is the depth below the ground surface.

C.12.3 P-y curves from drained soil properties

(C) For drained soil conditions the ultimate transversal ground resistance may be determined using formula C.17

$$p_{fd} = K_{qd} \cdot \sigma'_z + K_{cd} \cdot c'_d \quad \text{(C.17)}$$

where

$p_{f,d}$	is the design lateral pressure of the ground at failure;
c'_d	is the design effective cohesion of the ground;
σ'_z	is the effective vertical stress of the soil at the depth z;
K_{qd}, K_{cd}	is coefficients for calculation the ultimate drained soil resistance.

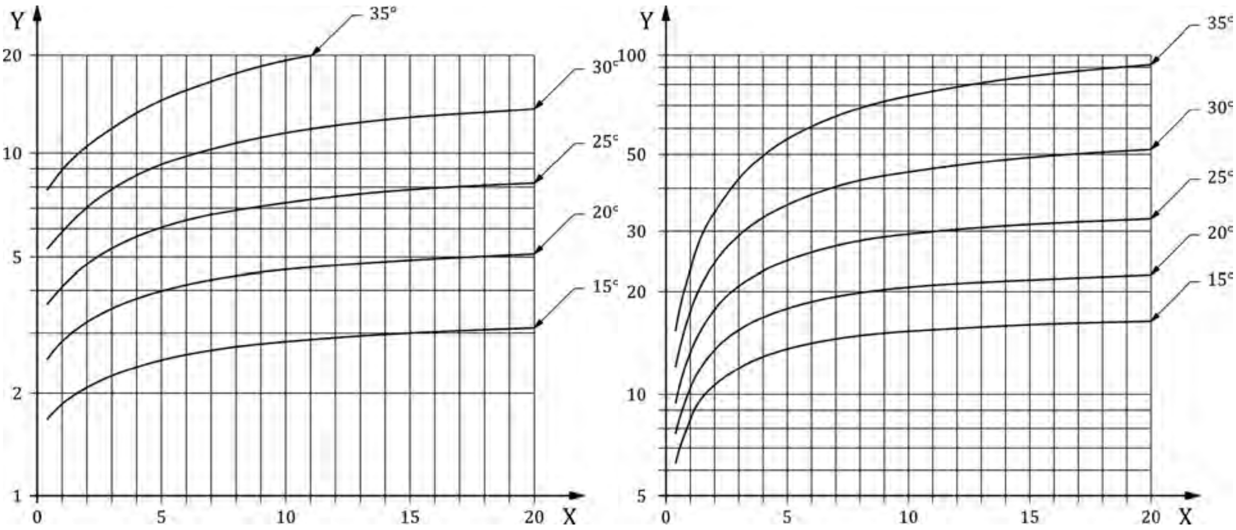
NOTE In Key

X *z/D [-]*

Y1 K_{qd}

Y2 K_{cd}

Figure C.9 gives the graphs for calculating the ultimate drained soil resistance according to Brinch Hansen (1961).



Key

X *z/D [-]*

Y1 K_{qd}

Y2 K_{cd}

Figure C.9 — Coefficients K_{qd} and K_{cd} for calculating the ultimate drained soil resistance

C.12.4 P-y curves from drained soil properties

(1) If a bilinear ground model according to formula (C.15) is used for the soil resistance, the necessary transversal displacement $y_{f,}$ resulting from the flexural buckling of the pile to mobilize p_f, may be assumed according to Table C.11

Table C.11 — Values of transversal displacement y_f.

Soil conditions	y_f
Coarse soils	0,1 B
Fine soils, long-term loading	0,12 B
Fine soils, short-term loading	0,05 B

(2) The buckling resistance, C.13, may also be determined for $y>y_f$ provided it can be verified that the soil does not undergo strain softening and that the necessary reduction is made to the overall transversal ground resistance.

NOTE A reduction to the ultimate ground resistance pf when $y>y_f$ can be calculated assuming equivalent overall ground pressure along the buckling length.

C.12.5 P-y curves from other field tests

(1) If a bilinear ground model as shown in Figure C.8 may be used derived from cone penetration test or pressuremeter test measurements.

C.13 Buckling and second order effects

C.13.1 General

(1) For piles subjected to compression, the structural resistance shall be verified by second order theory if the slenderness ratio is higher than the limits described in section C13.5.

(2) The buckling resistance of a slender pile under compression and embedded in soil should be determined by a validated model, either analytic or numerical, according to second order theory considering the support of the soil.

NOTE 1 The mobilisation of the ground resistance is dependent on the transversal deflection of the pile (see Figure C.11). The ground resistance is limited by different failure mechanisms which are dependent on the subsoil conditions as well as on the foundation geometry.

NOTE 2 The differential equation in Formula (C.18) is a validated calculation model for buckling of a uniform pile in uniform soil:

$$EI \cdot \frac{d^4 y}{dx^4} + C \cdot y + F \cdot \frac{d^2 y}{dx^2} = 0 \tag{C.18}$$

where

x	is the distance along the pile axis;
y	is the transversal deflection of the pile;
EI	is the flexural stiffness product of the pile;
C	is the subgrade reaction modulus;
F	is the axial force applied to the pile

(3) The structural resistance (ULS) and the deformation of piles (SLS) shall be verified in accordance with the structural design codes for concrete structures (prEN 1992 all parts), steel structures (prEN 1993 all parts), composite steel and concrete structures (prEN 1994 all parts) and timber structures (prEN 1995 all parts).

(4) For closely placed piles, where the centre to centre distance is less than 3D, a reduction in the transversal resistance shall be considered.

C.13.2 Buckling resistance by numerical methods

(1) The numerical method shall consider the second order moment caused by the transversal deformation during the axial loading of the pile.

NOTE 1 Numerical methods can be used for heterogeneous ground conditions and for piles with non-uniform cross section along the pile length.

NOTE 2 Numerical methods are usually based on Formula (C.18) for which the eigenvalues corresponds to the buckling forces.

(2) An initial deformation of the pile according to C13.2 should be applied, using values that are proportional to the buckling eigenmodes.

C.13.3 Buckling resistance by analytical methods

C.13.3.1 Buckling resistance

(1) The design value of buckling resistance N_{bd} for a fully embedded pile may be determined using Formula (C.19):

$$N_{bd} = \frac{y_f \cdot E_d I \cdot \left(\frac{\pi}{L_{bd}}\right)^2 + p_{fd} \cdot B \cdot \left(\frac{L_{bd}}{\pi}\right)^2}{y_f + e_{0d}} \tag{C.19}$$

where:

y_f is the relative deformation between the pile and the supporting soil where p_f is obtained

$E_d I$ is the flexural stiffness of the pile, design value according to the structural Eurocodes

L_{bd} is the buckling length, design value

p_{fd} is the design value of the ultimate transversal ground resistance [force/unit area] which may be reached with the deflection $y = y_f$ at $z^* = L_{bd}/2$, see Figure C.8 and Figure C.11

B is the shaft diameter or width of the pile in contact with the ground

e_{0d} Is the maximum transversal deformation of the initial curvature over the buckling length, design value

C.13.3.2 Buckling length

(1) The design value of the buckling length L_{bd} for a fully embedded pile should be determined using Formula (C.20):

$$L_{bd} = \pi \cdot \sqrt[4]{\frac{E_d I \cdot y_f}{p_{fd} \cdot B}} \tag{C.20}$$

where symbols are defined in Formula (C.19)

NOTE 1 For layered soils and soils with variable undrained strength over the buckling length L_{bd}, a combined average value of p_f *and* y_f, can be used.

NOTE 2 For a pile with a length $L < L_{bd}$ and where the pile top and base are pinned but free to rotate, $L_{bd} = L$ can be assumed.

C.13.3.3 Initial curvature

(1) An initial curvature of the pile shall be applied, considering production imperfections, installation effects and angular distortion of joints.

(2) With a given initial curvature, the parameter e_{0d} may be determined using Formula (C.21):

$$e_{0d} = \frac{(L_{bd})^2}{8 \cdot R_{0d}} \tag{C.21}$$

where

e_{0d} is the maximum transversal deformation of the initial curvature over the buckling length, design value;

R_{0d} is the curvature;

L_{bd} is the buckling length, design value.

(3) If no information about geometrical imperfections for a pile embedded in soil is known, the design curvature with R_{0d} within the buckling length may be assumed according to table C13.1.

NOTE Smaller values of R_{0d} are likely for piles with B<150 mm and for driven piles encountering boulders or heavily inclined bedrock.

Table C.13.1 — Design values of pile curvature.

Pile type	R_{0d} no joints	R_{0d} one joint[a]
bored steel and composite steel-concrete tube piles	300 m	150 m
driven steel and composite steel-concrete piles	200 m	100 m
precast concrete piles	200 m	100 m
cast insitu concrete piles	100 m	-
timber piles	100 m	
[a] within the buckling length		

(4) The following addition to e0d should be made to steel piles to account for manufacturing residual stresses in the pile, depending on the cross-sectional type:

- Type a_0, a: $0{,}0003 \cdot Lbd$
- Type b: $0{,}0013 \cdot Lbd$
- Type c: $0{,}0025 \cdot Lbd$
- Type d: $0{,}0045 \cdot Lbd$

NOTE Classification of cross-sectional types for buckling is found in Table 6.2 in FprEN 1993-1-1:2022.

C.13.4 Corresponding second order moment

(1) Cross-sectional checks shall be performed according to the structural Eurocodes taking into to account the corresponding second order moment during axial loading.

(2) For a pile of length equal or greater than L_{bd} according to Formula (C.20), the corresponding second order moment during axial loading may be accounted for by using Formula (C.22) and Formula (C.23):

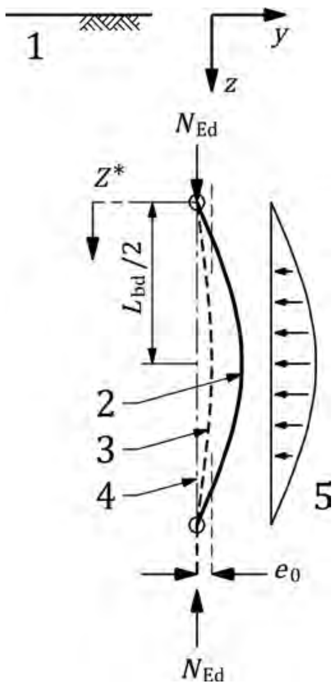
$$M_{2Ed} = N_{Ed} \cdot \frac{e_{0d} + y}{2} \quad \text{(C.22)}$$

$$y = \frac{N_{Ed} \cdot e_{0d}}{2 \cdot \left(\sqrt{B \left({}^{p_{fd}}/_{y_f}\right) E_d\, I}\right) - N_{Ed}} \quad \text{(C.23)}$$

where:

N_{Ed}	is the applied axial load, $N_{Ed} \leq N_{bd}$
M_{2Ed}	is the corresponding moment with second order effects
y	is the transverse deflection caused by the axial force ($y \leq y_f$), see Figure C.10.

NOTE Figure C.10 illustrates the transverse deflection of a pile caused by a compressive force.



Key

y transversal displacement

z depth

1 surface

2 buckling mode $N_{Ed} \leq N_{bd}$

3 Axis of imperfect pile for N_{Ed}

Figure C.10 — Transverse deflection of a pile caused by a compressive force.

C.13.5 Slenderness of piles

C.13.5.1 General

(1) The slenderness ratio λ of a fully embedded pile should be calculated by Formula (C.24):

$$\lambda = \frac{L_{\mathrm{bd}}}{\sqrt{2} \cdot i} = \frac{L_{bd}}{\sqrt{2 \cdot I/A}} \tag{C.24}$$

where

i	is the radius of gyration;
L_{bd}	is the buckling length calculated according to Formula (C.20);

A is the cross-sectional area of the pile.

C.13.5.2 Concrete piles

(1) Second order effects should be calculated for precast or cast insitu concrete piles if the slenderness ratio λ of the pile is greater than the limiting value λ_{lim} given in prEN 1992-1-1:2021, 5.8.3.1.

(2) At least half of the cross-sectional area of an unreinforced pile should be subjected to compression.

C.13.5.3 Steel piles

(1) Second order effects should be calculated for steel piles if the slenderness ratio λ is large, or the axial force N_{Ed} is large compared to the ideal critical elastic force N_{cr}.

NOTE 1 A large slenderness ratio is $\lambda \geq 0.2$, and a large axial force is $N_{Ed}/N_{cr} \geq 0.04$, according to prEN 1993-1-1:2022, 6.3.1.2(4). For piles fully embedded in the ground a large axial force is $N_{Ed}/N_{cr} \geq 0.10$ according to EN 1993-5:2007, 5.3.3(3).

NOTE 2 For a fully embedded straight pile the critical buckling load is determined according to Formula (C.25)

$$N_{cr} = 2 \cdot \sqrt{EI \cdot \frac{p_f \cdot B}{y_f}} \tag{C.25}$$

where

EI	is the flexural stiffness of the pile, design value according to the structural Eurocodes;
N_{cr}	is the critical elastic force;
y_f	is the relative deformation between the pile and the supporting soil where p_f is obtained;
B	is the cross-sectional area of the pile;
p_f	is the value of the ultimate transversal ground resistance.

C.13.5.4 Composite steel-concrete piles

(1) Second order effects should be calculated for composite steel-concrete piles if $N_{Ed}/N_{cr} \geq 0.10$.

NOTE N_{cr} is calculated using Formula (C.25) with the effective flexural stiffness (EI)eff according to EN 1994-1-1:2004, 6.7.3.3.

C.13.5.5 Timber piles

(1) Second order effects for timber piles should be calculated if the relative slenderness ratio λ_{rel} of the pile is greater than 0.3 as specified in prEN 1995-1-1:2004, 6.3.2.

(2) The relative slenderness may be determined by Formula (C.26)

$$\lambda_{rel} = \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \tag{C.26}$$

where

λ_{rel} is the relative slenderness ratio;

λ is slenderness ratio;

$f_{c,0,k}$

$E_{0,05}$

C.13.6 Partial factors

(1) Superior or inferior representative values should be adopted for the ground stiffness and ground strength depending on which is critical.

NOTE High values are sometimes critical when transversal loads, e.g. from settling soil, are present.

(2) Partial factors on the ultimate transversal ground resistance pf derived from ground strength parameters shall be in accordance to set M2 in prEN 1997-1:2022, Annex A.

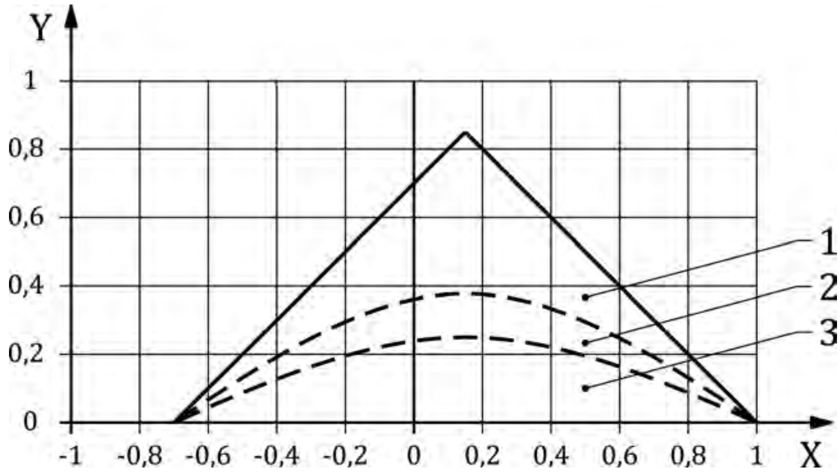
(3) A partial factor of γ_{pf} = 1,4·K_M should be applied to a measured value of ultimate transversal ground resistance, p_f.

C.14 Cyclic effects

C.14.1 Pile stability diagrams

(1) The concept of stability diagram may be used to determine whether the axial loads applied at the pile head can induce some cyclic effects.

NOTE Figure C.11 gives an example of a stability diagram.



Key

X	$G_{ave,rep}/R_c$
Y	$\Delta Q_{rep}/R_c$
R_c	Axial compressive resistance;
$G_{ave,rep}$	Representative value of the average load applied on the pile;
ΔQ_{rep}	Representative value of the half amplitude variable load
A	Stable domain: no cyclic effects
B	Metastable domain: Limited cyclic effects inducing low reduction of the pile bearing capacity with limited displacements
C	Unstable domain: significant cyclic effects inducing strong reduction of the pile bearing capacity until failure

Figure C.11 — Principle of cyclic stability diagram for axially loaded piles.

(2) Stability diagram should be developed considering specific ground conditions and pile types.

NOTE Examples of stability diagrams can be found in the literature.

(3) When a representative cyclic stability diagram leads to identify a metastable domain or an unstable domain for specific ground conditions and pile types, more detailed verifications should be conducted to assess the impact of the cyclic loads for both the SLS (cumulative pile head displacements) and ULS (degradation of ultimate resistance).

NOTE Detailed cyclic pile design procedures have been developed by the offshore industry (EN ISO 19901-4).

Annex D
(informative)

Retaining structures

D.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to that given in Clause 7, retaining structures.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

D.2 Scope and field of application

(1) This Informative Annex covers:

- limit values of earth pressures;
- at rest values of earth pressures;
- compaction effects;
- additional earth pressures induced by thermal effects for integral bridges;
- general principles and application of calculation models: limit equilibrium, beam on springs, numerical models;
- vertical equilibrium of embedded walls;
- basal heave; and
- interaction between anchors and retaining structures.

D.3 Calculation model to determine limit values of earth pressures on vertical walls

(1) In addition to 7.5.4, the values of the active earth pressure coefficients $K_{a\gamma}$, K_{aq}, and K_{ac} may be determined according to (3), (5), (8), and (9) of this sub-clause.

(2) In addition to 7.5.5, the values of the passive earth pressure coefficients $K_{p\gamma}$, K_{pq}, and K_{pc} may be determined according to (4), (6), (8), and (9) of this sub-clause.

(3) Selected values of $K_{a\gamma}$ and $K_{p\gamma}$ may be determined from Figure D.2 and Figure D.3.

NOTE Values are also given in tabular form by Kérisel and Absi (1990).

(4) The value of K_{aq} may be determined from Formula (D.1):

$$K_{aq} = k_{aq} \cos \delta \tag{D.1}$$

where:

k_{aq} is the inclined active earth pressure coefficient;

K_{aq} is the component of k_{aq} normal to the wall face.

(5) The value of K_{pq} may be determined from Formula (D.2):

$$K_{\text{pq}} = k_{\text{pq}} \cos \delta \tag{D.2}$$

where:

k_{pq} is the inclined passive earth pressure coefficient; and

K_{pq} is the component of k_{pq} normal to the wall face

(6) The values of k_{aq} and k_{pq} may be determined from Formulae (D.3)-(D.8):

$$k_{\text{aq}} = \left(\frac{\cos \delta - \sin \varphi \cos \omega_{\delta}}{\cos \alpha + \sin \varphi \cos \omega_{\alpha}}\right) e^{-2\varepsilon_{\text{a}} \tan \phi} \tag{D.3}$$

$$k_{\text{pq}} = \left(\frac{\cos \delta + \sin \varphi \cos \omega_{\delta}}{\cos \alpha - \sin \varphi \cos \omega_{\alpha}}\right) e^{2\varepsilon_{\text{p}} \tan \phi} \tag{D.4}$$

$$\sin \omega_{\delta} = \frac{\sin \delta}{\sin \varphi} \tag{D.5}$$

$$\sin \omega_{\alpha} = \frac{\sin \alpha}{\sin \varphi} \tag{D.6}$$

$$\varepsilon_{\text{a}} = \frac{(\omega_{\text{a}} + a)}{2} + \frac{(\omega_{\delta} - \delta)}{2} + \beta - \lambda \tag{D.7}$$

$$\varepsilon_{\text{p}} = \frac{(-\omega_{\text{a}} + a)}{2} - \frac{(\omega_{\delta} + \delta)}{2} + \beta - \lambda \tag{D.8}$$

where:

φ is the angle of internal friction of the soil;

δ is the angle of inclination of the earth pressure;

α is the angle of inclination of the surcharge;

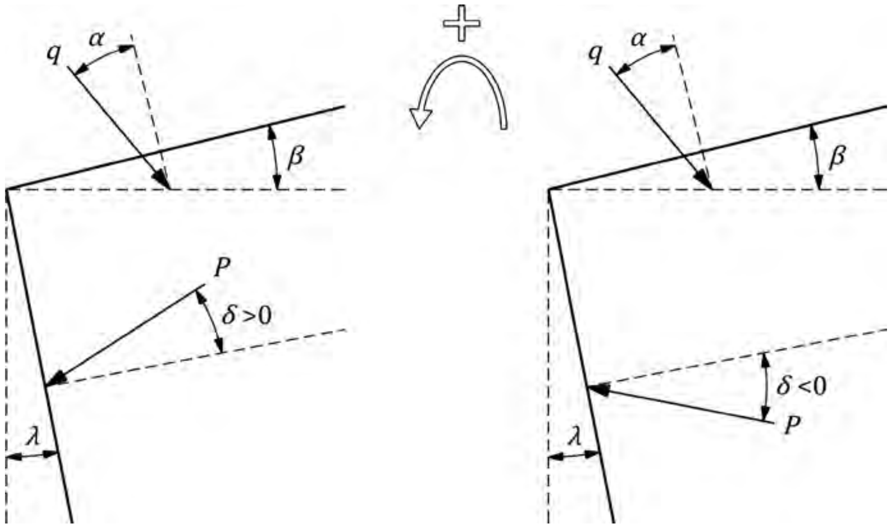
β is the inclination of the ground surface;

λ is the inclination of the wall.

NOTE 1 Positive orientations of these angles are indicated in Figure D.1.

NOTE 2 When $\delta = \alpha = \beta = \lambda = 0$, $K_{a\gamma} = K_{aq} = \tan^2(\pi/4 - \varphi/2)$ and $K_{p\gamma} = K_{pq} = \tan^2(\pi/4 + \varphi/2)$.

NOTE 3 When $\alpha = \beta = \lambda = 0$, K_{aq} is approximately equal to $K_{a\gamma}$ and K_{pq} to $K_{p\gamma}$.



Key

X definition for X

Y definition for Y

α is the angle of inclination of the surcharge;

β is the inclination of the ground surface;

δ is the angle of inclination of the earth pressure;

λ is the inclination of the wall.

Figure D.1 — Orientation for angles α, β, δ, and λ (left: active earth pressure; right: passive)

(7) When $\varphi > 0$, the values of K_{ac} and K_{pc} may be determined from Formulae (D.9)-(D.12):

$$K_{\mathrm{ac}} = \frac{1 - \left(\frac{\cos\delta - \sin\varphi\cos\omega_{\delta}}{1+\sin\varphi}\right) e^{-2\varepsilon_{\mathrm{a}}\tan\phi}\cos\delta}{\tan\varphi} \quad \text{(D.9)}$$

$$K_{\mathrm{pc}} = \frac{\left(\frac{\cos\delta + \sin\varphi\cos\omega_{\delta}}{1-\sin\varphi}\right) e^{-2\varepsilon_{\mathrm{p}}\tan\phi}\cos\delta - 1}{\tan\varphi} \quad \text{(D.10)}$$

$$\varepsilon_{\mathrm{a}} = \frac{(\omega_{\delta} - \delta)}{2} + \beta - \lambda \quad \text{(D.11)}$$

$$\varepsilon_{\mathrm{p}} = \frac{(\omega_{\delta} + \delta)}{2} - \beta + \lambda \quad \text{(D.12)}$$

where ω_{δ} and ω_{α} are given in Formula (D.3)-(D.8) and the other symbols are as defined in (6).

NOTE These expressions are based on the assumption that a/c = (tan δ)/(tan φ), where a is the adhesion between the ground and wall.

(8) When $\varphi = 0$ and $\lambda = \beta = 0$, the values of K_{ac} (= $k_{ac,u}$) and K_{pc} (= $k_{pc,u}$) may be determined from Formula (D.13):

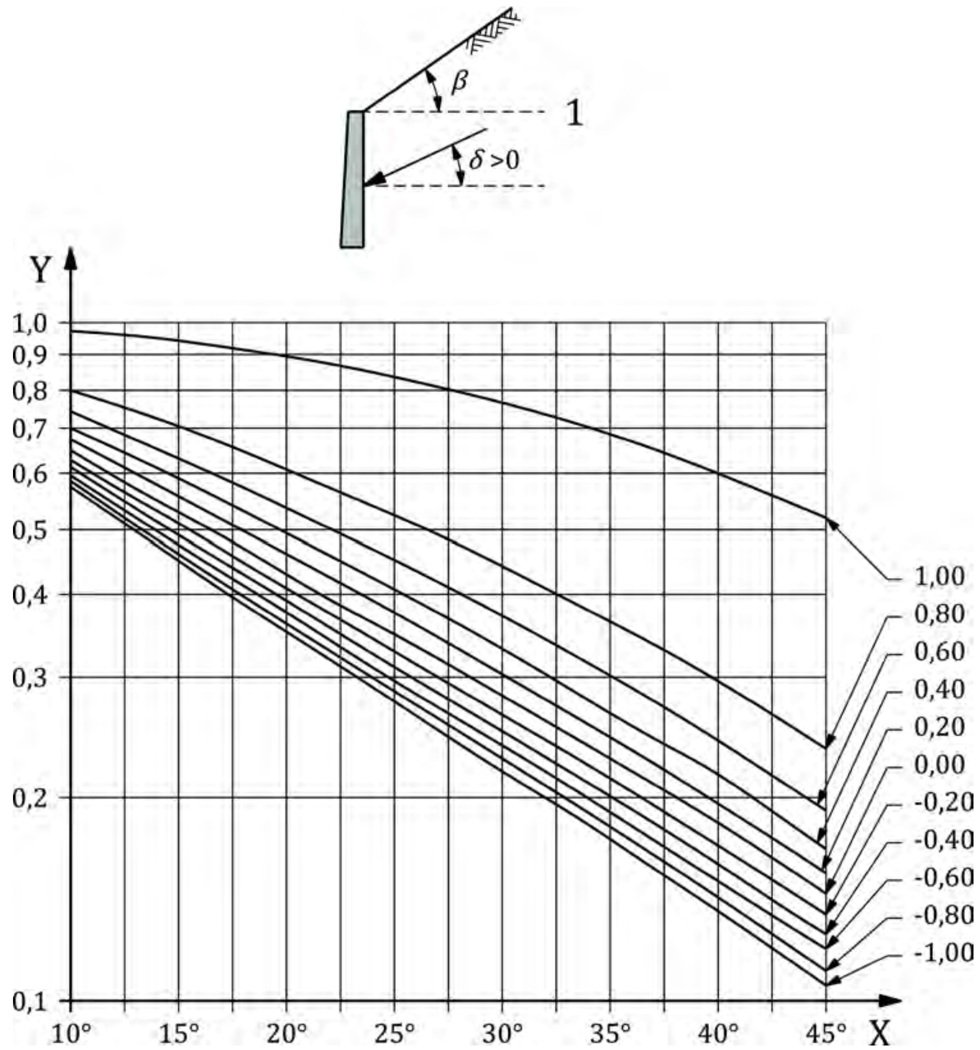
$$K_{\mathrm{ac,u}} = K_{\mathrm{pc,u}} = 1 + \sin^{-1}\left(\frac{a}{c}\right) + cos\left(\sin^{-1}\left(\frac{a}{c}\right)\right) \tag{D.13}$$

where

- a is the adhesion between the ground and wall
- c is the cohesion

NOTE 1 Figure D.2 give the coefficients of effective active earth pressure with inclined retained surface.

NOTE 2 Figure D.3 give the coefficients of effective passive earth pressure with inclined retained surface.

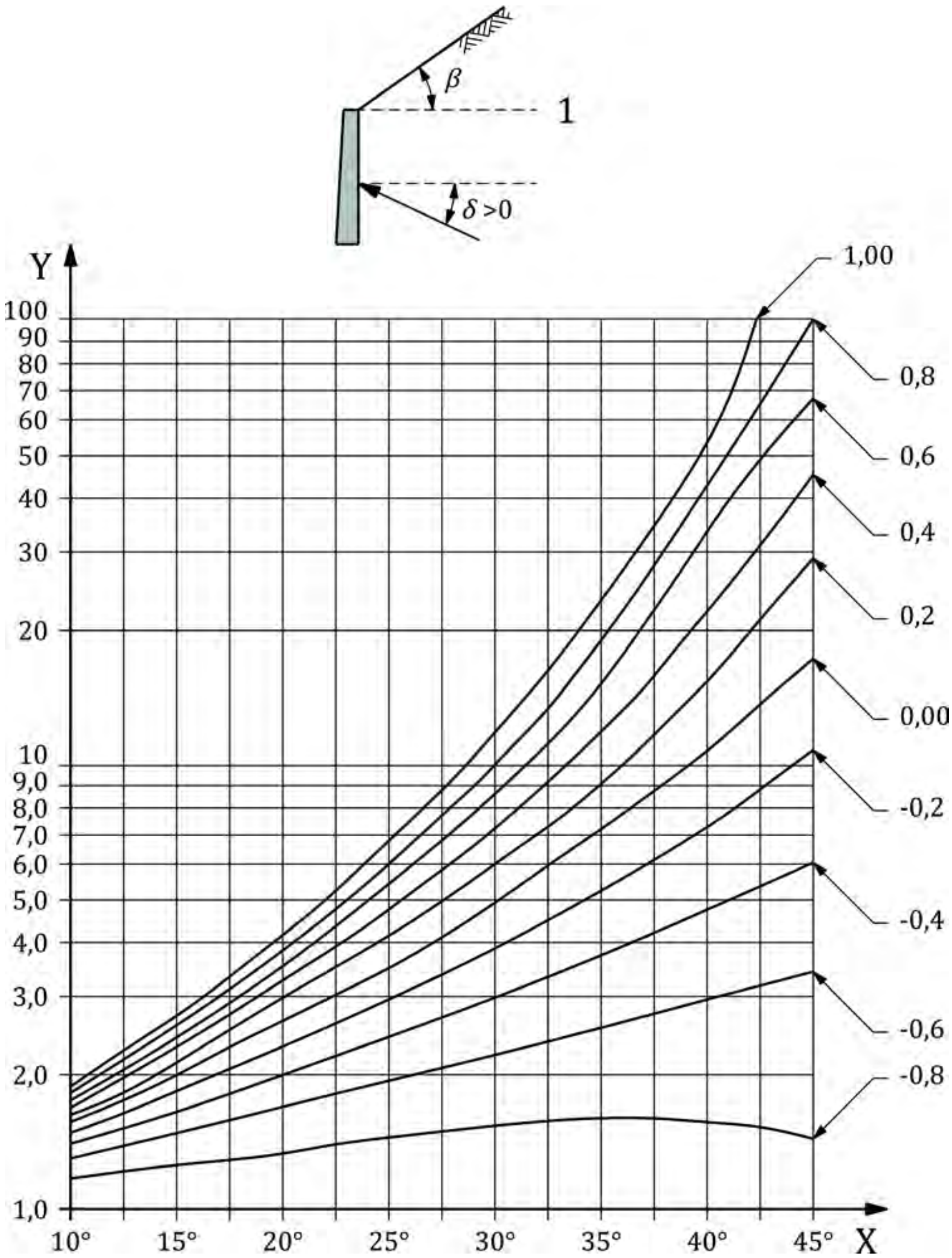


Key

X angle of friction

Y Ka effective active earth pressure (horizontal component)

Figure D.2 — Coefficients of effective active earth pressure K_a (horizontal component) with inclined retained surface (δ/φ' = 0,66)



Key

X angle of friction

Y Kp effective passive earth pressure (horizontal component)

Figure D.3 — Coefficients of effective passive earth pressure K_p (horizontal component) with inclined retained surface (δ/φ' = 0,66)

D.4 Calculation model to determine at-rest values of earth pressure

(1) In addition to 7.5.6, the at-rest earth pressure coefficient K_0 in soils may be determined only for unloading stress paths from Formula (D.14):

$$K_0 = (1 - \sin\varphi)\sqrt{R_\mathrm{o}} \times (1 + \sin\beta) \leq K_{\mathrm{p}\gamma} \quad \text{(D.14)}$$

where:

φ	is the soil's internal angle of shearing resistance;
R_o	is the over-consolidation ratio at depth z_0 (equal to $\sigma'_{v,max} / \sigma'_v$);
$\sigma'_{v,max}$	is the maximum effective overburden pressure at depth z_0;
σ'_v	is the current effective overburden pressure at depth z_0; and
β	is the inclination of the ground surface above the horizontal;
$K_{p\gamma}$	is the passive earth pressure coefficient.

(2) Formula (D.14) should not be used for very high values of R_o or in circumstances involving geological reloading.

NOTE Formula (D.14) can lead to unrealistic values of K_0 close to the ground surface, where the vertical stress is low.

(3) The direction of the resulting force should be assumed to be parallel to the ground surface.

(4) A distinction may be made between:

- K_0, the earth pressure coefficient in the initial stage before the works begin;
- K_i, the earth pressure coefficient in the initial stage after completion of the retaining wall but before the start of excavation; and
- K_d, the ratio between variations in horizontal and vertical stresses during excavation assuming at-rest conditions, that is without horizontal displacement of the retaining wall

NOTE 1 Assuming linear elastic behaviour and considering reloading stress paths, where υ is Poisson's ratio of the soil, Kd can be determined from Formula (D.15)

$$K_d = \upsilon/(1-\upsilon) \tag{D.15}$$

NOTE 2 In practice, due to the poor knowledge about reliable values for K_i and K_d, it is typically assumed that $K_0 = K_i = K_d$.

NOTE 3 For overconsolidated cohesive soils, in which excavation may lead to a significant stress relief, $K_i < K_0$.

D.5 Earth pressures due to compaction

(1) The effective compaction earth pressure normal to the wall face (p'c) at a depth (z) below ground surface may be determined from Formulae (D.16)-(D.18):

NOTE Measurements indicate that additional pressures depend on the applied compaction energy, the soil moisture content, the thickness of the compacted layers and the travel pattern of the compaction machinery. Horizontal pressure normal to the wall in a layer can be reduced when the next layer is placed and compacted. When backfilling is complete, the additional pressure normally acts only on the upper part of the wall.

(1)

$$p'_{\mathrm{c}} = \begin{cases} K_{\mathrm{p}\gamma}\overline{\gamma}_{\mathrm{c}}z \text{ for } z \le z_{\mathrm{c,min}} \\ p'_{\mathrm{c,max}} \text{ for } z_{\mathrm{c,min}} \le z \le z_{\mathrm{c,max}} \\ K_0\overline{\gamma}_{\mathrm{c}}z \text{ for } z \ge z_{\mathrm{c,max}} \end{cases} \tag{D.16}$$

$$z_{\mathrm{c,min}} = \frac{p'_{\mathrm{c,max}}}{\overline{\gamma}_{\mathrm{c}} K_{\mathrm{p\gamma}}} \tag{D.17}$$

$$z_{\mathrm{c,max}} = \frac{p'_{\mathrm{c,max}}}{\overline{\gamma}_{\mathrm{c}} K_{0}} \tag{D.18}$$

where:

$p'_{\mathrm{c,max}}$ is the maximum horizontal earth pressure due to compaction;

$\overline{\gamma}_{\mathrm{c}}$ is the average weight density of the ground over depth $z_{\mathrm{c,max}}$;

$K_{\mathrm{p\gamma,0}}$ is the passive earth pressure coefficient (with wall friction equal to zero);

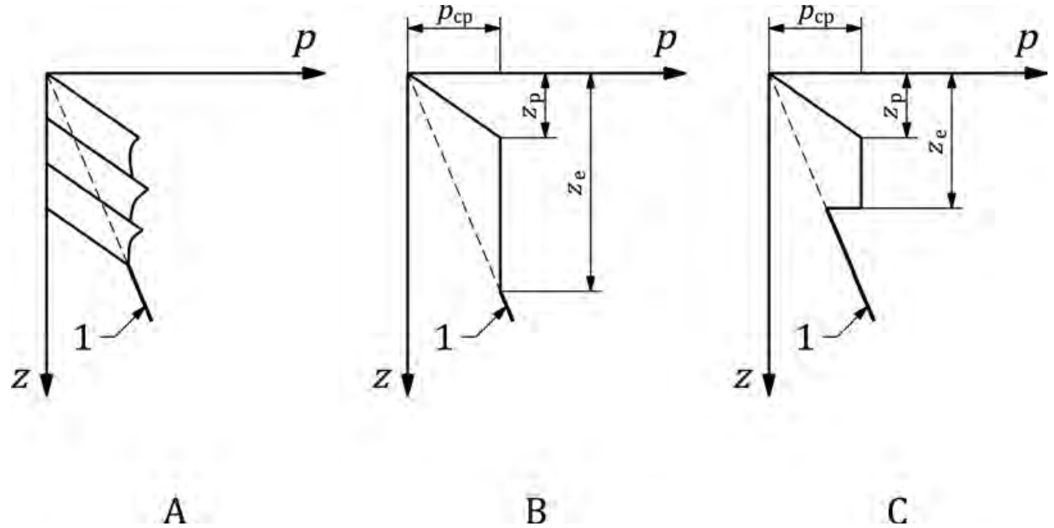
K_0 is the at-rest earth pressure coefficient;

$z_{\mathrm{c,min}}$ is the minimum depth at which p'_{c} applies;

$z_{\mathrm{c,max}}$ is the maximum depth at which p'_{c} applies.

(2) For non-yielding walls, compaction pressure may be represented by the bi-linear profile shown in Figure D.4(b).

NOTE Compaction pressures from soil placement in layers, more realistically produces a distribution similar to that shown in Figure D.4(a).



Key

A compaction earth pressure

B simplified profile for non-yielding

C yielding wall

1 K_o line

Figure D.4 — Distribution of compaction earth pressure (a); simplified profile for non-yielding wall (b) and yielding wall (c)

(3) The value of the maximum compaction earth pressure $p'_{\mathrm{c,max}}$ may be taken from Table D.1.

(4) For yielding walls, the simplified depth profile shown in Figure D.4c may be adopted.

(5) In case the wall displacement is associated with earth pressures between active and at-rest conditions, interpolated values may be used.

Table D.1 — Values of the maximum compaction earth pressure $p'_{c,max}$ (kPa)

<table>
<tr><th rowspan="2">Wall</th><th colspan="2">Intensive compaction
Width b of backfilled space</th><th rowspan="2">Light compaction (vibratory compactor mass ≤ 250 kg)</th></tr>
<tr><th>b ≤ 1.0 m</th><th>b ≥ 2.5 m</th></tr>
<tr><td>Non-yielding</td><td>40</td><td>25</td><td>15</td></tr>
<tr><td>Yielding</td><td colspan="2">25 (z = 2.0 m)</td><td>15 (z = 2.0 m)</td></tr>
<tr><td colspan="4">NOTE Use interpolation for intermediate values of b</td></tr>
</table>

D.6 Earth pressures caused by cyclic thermal movement for integral bridges

(1) The earth pressure on a structural element subjected to cyclic thermal movements should be calculated based on the thermal movement range as well as the direction (expansion or contraction) and actual amount of the relative movements.

(2) Earth pressures caused by cyclic thermal movements may be assessed by soil-structure interaction methods calibrated against comparable experience, laboratory modelling and/or case history data experience.

(3) Maximum and minimum values of the earth pressure applicable to structural design should be considered coincident with the values of the effects (temperature, creep, shrinkage) causing the expansion or contraction, respectively.

(4) The value of the enhanced pressure coefficient K* for a given value of the maximum expansion should be determined based on a recognized method.

NOTE The enhanced pressure is bounded by the earth pressure mobilised by the maximum thermal expansion (lower limit) and the full passive earth pressure (upper limit).

D.7 Basal heave

(1) Mechanical heave due to excavation is generally associated with settlements outside and should be considered as part of overall stability mechanisms.

(2) Specific models may be used to deal with the following situations:

– conventional models for overall stability calculation;

NOTE 1 These models do not take account of specific geometry (narrow and deep excavation for instance).

– concentration of vertical hydraulic gradients along the embedded part of the retaining wall;

NOTE 2 These models can locally initiate an instability process for which rigid block mechanisms cannot be considered as realistic enough.

– mechanical extrusion of soft clay that occurs simultaneously with excavation at depth.

NOTE 3 These models cannot be realistically compensated by external shear resistance, as conventional rigid block mechanisms would assume.

(3) Shear resistance may be considered.

NOTE Figure D.5 illustrates verification against basal heave.

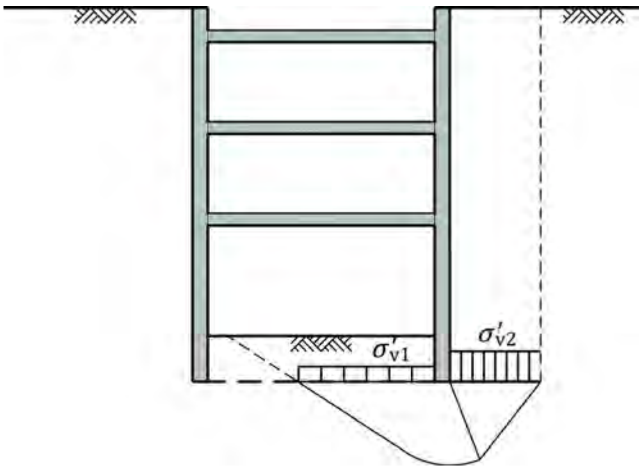


Figure D.5 — Verification against basal heave

(4) Simplified models may be used for fine or coarse soils in which the external and internal shear resistance above the toe level of the retaining wall is neglected and the same mechanisms as for bearing capacity of shallow foundations are considered.

(5) In such conditions, the limit value of the effective vertical stress that can be applied at toe level outside the excavation σ'_{v1} may be determined from Formula (D.19)

$$\sigma'_{v1} = \frac{\gamma B}{2} N_\gamma + \sigma'_{v2} N_q + c N_c \tag{D.19}$$

where:

N_γ, N_q, and N_c	are bearing capacity factors (see Clause 5);
γ	is the unit weight of soil under the wall;
B	is the width to consider outside the excavation;
c	is the cohesion;
σ'_{v2}	is the effective vertical stress at toe level inside the excavation.

(6) Mechanical heave during excavation in fine soils may be analysed assuming undrained conditions and total stress analysis, using $N_\gamma = 0$.

(7) Mechanical heave in coarse soils may be analysed assuming hydraulic gradients are concentrated within a narrow area very close to the wall, allowing the width B to be neglected.

(8) Verification of resistance to mechanical heave caused by hydraulic gradients in coarse soils should be based on an effective stress analysis, considering effective cohesion c', as well as effective stresses σ'_{v1} and σ'_{v2}.

(9) The values of σ'_{v1} and σ'_{v2} in Formula (D.19) should consider weight densities ($\gamma' + i_1\gamma_w$) and ($\gamma' - i_2\gamma_w$), where i_1 is the average gradient along the retained side of the wall and i_2 the average gradient along the wall on the excavated side.

(10) In addition to (9), hydraulic gradients and unit weights also shall be evaluated and considered for the calculation of the retaining wall itself.

(11) Verification of resistance to mechanical heave during excavation in fine soils should be based on a total stress analysis based on Bjerrum and Eide approach in Formula (D.20)

$$\gamma H_e + q_s \leq N_c c_u = \\ with\ N_c = 5 * \left(1 + 0.2\frac{H_e}{B}\right) * \left(1 + 0.2\frac{B}{L}\right)\ if\ \frac{H_e}{B} \leq 2.5,\ \ N_c = 7.5\left(1 + 0.2 * \frac{B}{L}\right) \tag{D.20}$$

where:

H_e is the depth of the excavation;

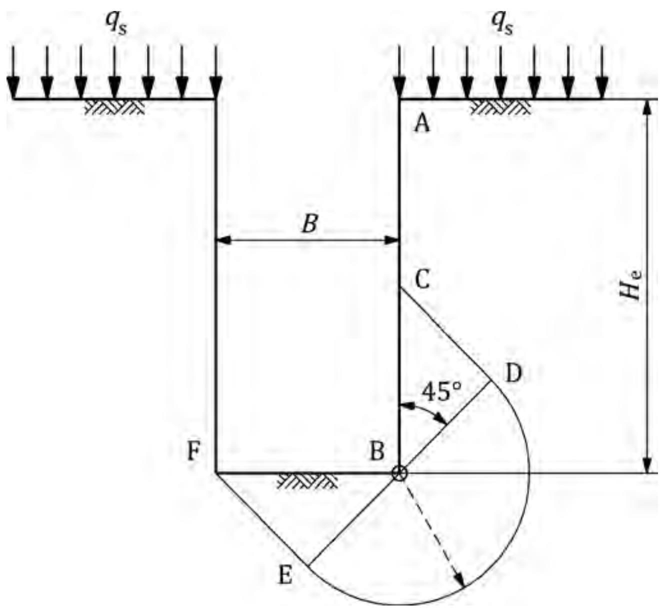
q_s Is the surface load;

c_u is the undrained shear strength;

N_c is a shape factor depending on the length and the width of the excavation.

NOTE 1 For more details, see, Bjerrum and Eide, (1956).

NOTE 2 Figure D.6 illustrate basal heave in fine soils.



Key

He	Depth of excavation
B	Width of excavation
qs	Surface load
A, B, C, D, E, F,	Volume of the ground subjected to the basal heave mechanism

Figure D.6 — Basal heave in fine soils (Bjerrum and Eide, 1956)

D.8 Limit equilibrium models

(1) Limit equilibrium models may be used both for:

– for gravity walls;
– for retaining walls to estimate the minimum embedded length and support reactions that are necessary to prevent rotational resistance (see 7.6.4.1).

NOTE 1 Limit equilibrium models consist of analysing horizontal stability of embedded retaining walls by assuming that limiting values of earth pressures are reached on both sides of the wall.

NOTE 2 Earth pressure envelopes, which can be used for walls with multiple supports, can be found in the literature. For only partially compliant walls a weighted average of active pressure and earth pressure at rest is commonly assumed.

NOTE 3 Limit equilibrium models are simplified models that do not provide information relative to displacements; they are generally used for the design of flexible embedded walls and stiff single propped walls. These models ignore construction sequences, and structural stiffness or prestressing effects.

(2) When limit equilibrium models are used to justify plastic hinges in metallic structures accordingly with EN 1993-5, limit displacements associated with limit earth pressures may be estimated based on conventional order of magnitude, traditionally expressed as a proportion λ_a of the wall height on the retained side, and λ_p of the embedded depth on the excavated side.

NOTE The values of λ_a and λ_p are 0.1-0.3 % and 1-5 %, respectively, unless different values are given in the National Annex.

D.9 Beam-on-spring models

(1) Beam-on-springs models may be used to check the following limit states, in accordance with 7.6 and 7.7:

– serviceability limit states involving horizontal displacements, within the limits given in D.7;
– structural limit states;
– rotational resistance(see 7.6.4.1).

(2) Unless additional effects are introduced into the calculation, limit equilibrium and beam-on-springs models should not be used to determine: slope instability, interaction between the retaining structure and rear anchors, or interaction between front and rear quay walls.

NOTE Wall displacements are usually calculated relative to the ground surface, ignoring any displacement of the ground surface.

(3) Intermediate values of earth horizontal pressures may be determined by use of the subgrade reaction coefficient, $k = \Delta\sigma / \Delta y$, where $\Delta\sigma$ is the variation of earth pressure associated with a variation of horizontal wall displacement Δy.

NOTE 1 This is a simplification that assimilates the ground to independent springs.

NOTE 2 Due to its empirical nature, values of the coefficient of subgrade reaction should always be determined from comparable experience in similar conditions. Guidance is provided in D.8.

NOTE 3 Spring stiffness values are very software specific.

(4) When redistribution of earth pressure due to arching effects caused by the compliance of the earth retaining structure is likely to occur, limit and intermediate values of earth pressure on the retained side should be determined from methods that take account of such redistribution.

NOTE 1 Such methods include empirical (see D.6) and continuum numerical models.

NOTE 2 Relative movements within the retained ground can cause redistribution, for example when rigidities of different support layers significantly differ from each other or when high spans exist between adjacent rigid supports.

NOTE 3 Beam-on-springs models are able to consider increased earth pressures behind rigid supports when they are prestressed.

(5) Empirical relationships based on past experience may be used to derive soil settlements behind the wall from its horizontal displacement.

NOTE Ratios between maximum vertical and maximum horizontal displacements usually lie between 0.5 and 1.

D.10 Calculation model to determine intermediate values of earth pressure

(1) The value of the subgrade reaction coefficient k may be estimated from the approximate Formula (D.21):

$$k = \frac{E_s}{d} \tag{D.21}$$

where:

E_s is the secant soil's modulus of elasticity; and

d is the interaction length.

(2) When determining the interaction length d, the following should be considered:

- the interaction length cannot be larger than the total embedment length D of the wall;
- in practice, it generally is considered that $d < 2/3\ D$;
- during intermediate excavation stages, for which passive earth pressure is only mobilized along a limited part of the embedded height, an order of magnitude, consistent with the theory of beams resting on elastic supports and confirmed by a large series of monitoring results, is $d = 1.5\ l_0$, where $l_0 = (4EI / k)^{1/4}$, and EI is the bending stiffness of the wall per linear metre;
- in specific circumstances where the embedded length is determined by hydraulic considerations rather than by the mechanical mobilization of passive earth pressure due to excavation, the interaction length is no longer depending on the bending stiffness, as high differential water pressures affect the total height.

NOTE 1 Example of hydraulic considerations are pumping phases without excavation, tidal effects on quay walls, high water head and increased embedded length in order to reach an impervious layer.

NOTE 2 In current situations for which the interaction height is dependent on the bending stiffness, an estimate determined from the relationships above is $k = 0.4\ E_s^{4/3}/(EI)^{1/3}$.

NOTE 3 The soil modulus E_s to consider is intermediate between the initial loading modulus and the unload-reload modulus.

(3) As an alternative to (1) and (2), other methods may be used for structures that mobilize passive pressure in backfill.

NOTE For example, bridge abutments.

(4) Backfill soil reaction forces on bridge abutments should consider the increase in passive earth pressure with wall movement.

NOTE For temperature induced seasonal wall movements, the predominant pattern is a combination of horizontal translation and rotation about the wall base.

(5) The horizontal component of the mobilised passive earth pressure coefficient $K_{\mathrm{ph,mob}}$ along the wall height may be determined from Formula (D.22):

$$K_{\mathrm{ph,mob}}(z) = K_0 + \left(K_{\mathrm{ph}} - K_0\right)\frac{v(z)/z}{a + v(z)/z} \tag{D.22}$$

where:

K_0 is the coefficient of earth pressure at rest;

K_{ph} is the horizontal component of the coefficient of passive earth pressure;

z is the depth;

v(z) horizontal displacement at depth z (positive towards the backfill); for a rigid wall rotating about its base, $v(z) = s_{\mathrm{h}}(1 - z/h)$;

s_{h} horizontal displacement at the wall top;

h Is the height of the retaining wall;

a is a backfill-dependent coefficient.

(6) In the absence of detailed specifications, the value $a = 0.02$ may be used.

D.11 Numerical continuum models

(1) The most critical geotechnical failure mechanism or combination of failure mechanisms may be determined by numerical continuum models using shear strength reduction approach.

NOTE Examples of combination of failure mechanisms are overall or bottom instability, rotational failure, foundation failure.

(2) Information relative to settlements should be considered carefully when simplified linear elastic models are used, since such models cannot take account of different soil behaviours during a primary loading and an excavation.

NOTE 1 In the case of retaining structures, only non-linear models provide relevant information with respect to both horizontal and vertical displacements within the ground mass.

NOTE 2 Current soil models rarely take account of the anisotropic behaviour of alluvial soils, which is likely to influence the relationship between horizontal and vertical displacements around a retaining structure.

(3) In undrained conditions, when calculation is performed in terms of effective stresses, attention should be paid to the decrease of groundwater pressures induced by the dilatancy generated with an inappropriate constitutive law.

D.12 **Vertical wall stability**

(1) According to 7.6.4.2, the skin friction needed to ensure vertical equilibrium of an embedded wall, and the vertical components of active and passive earth pressures needed to ensure its horizontal equilibrium should be consistent with each other.

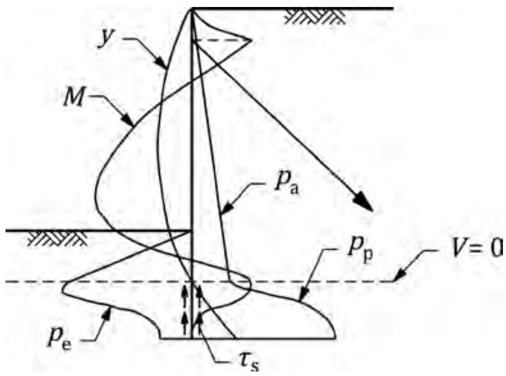
(2) Consistency between skin friction (in bearing capacity calculations) and vertical components of earth pressure (used to justify horizontal equilibrium) should be checked above the depth at which the shear force applied to the embedded part of the wall is equal to 0 (see Figure D.7).

NOTE 1 This level can be considered as a rotation axis above which it is essential that earth pressures are not underestimated on the retained side and are overestimated on the excavated side; beneath this level, such eventualities become on the safe side.

NOTE 2 Mobilising skin friction to equilibrate vertical forces changes the inclination of earth pressures δ, that tends to increase the active earth pressure earth side if structural forces are exerted downwards, or decrease the passive earth pressure on the excavated side if structural forces are exerted upwards (e.g. inclined struts resting on the excavated surface).

NOTE 3 Despite using a negative value of the inclination δ to derive earth pressure on the retained side, the vertical component can be significantly lower than the friction that could be mobilised without stress relief and, for this reason, it is often neglected in bearing capacity calculations.

NOTE 4 Figure D.7 illustrates the depth at which shear force applied to embedded wall is zero.



Key

X definition for X

Y is the horizontal displacement of the retaining structure;

M is the bending moment;

V is the shear force;

p_a is the active earth pressure applied to the wall;

p_p is the passive earth pressure applied to the wall

τ_s is the shaft friction mobilized to equilibrate the vertical anchor force

Figure D.7 — Depth at which shear force applied to embedded wall is zero

D.13 Determination of the anchor length to prevent interaction between anchors and retaining structures

(1) Potential interaction between a retaining structure and any deadman anchors used to stabilize it may be ignored when the passive wedge mobilized by the anchor does not intersect with the active wedge acting on the structure.

(2) The model illustrated in Figure D.8 may be used to ensure that grouted anchors do not interfere with a retaining structure:

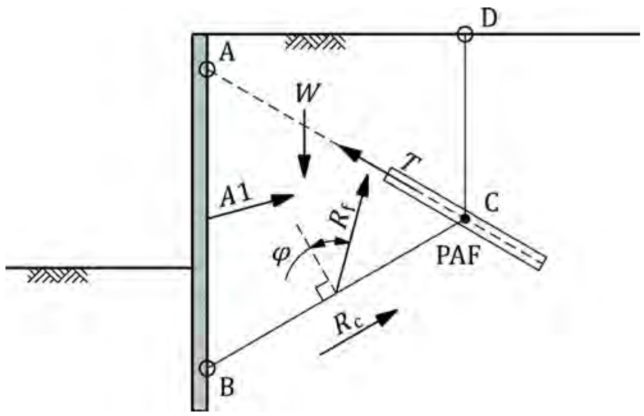
– the anchor's reaction is assumed to be balanced by the shear resistance that is mobilised along the conventional failure surface shown in Figure D.8, so not to increase earth pressures directly acting on the wall;
– equilibrium of forces acting on the ground between the retaining wall and the anchors provide the maximum anchor force that can be equilibrated without increasing earth pressures on the wall;
– interaction is neglected when the ratio between this maximum anchor force, and the applied anchor force based on previous calculations of the retaining wall, is higher than 1.5.

NOTE 1 If this condition in Figure D.8 is not met, the shear resistance that the soil mobilizes along the conventional failure surface is insufficient to dissipate the force applied by the anchor. Consequently, the retaining structure has to provide more reaction to ensure overall equilibrium of the soil mass that needs to be considered in the calculation model, or the free length of the anchor has to be increased until it is justified that interaction can be neglected.

NOTE 2 The stabilizing reaction A1 to introduce in the calculation is equal and opposite to the resulting effective earth pressure considered for the design of the retaining structure itself.

NOTE 3 The consequence is that the equilibrium of forces applied to the volume ABCD provides a value of the anchor force, F, that is the maximum one that the anchor can apply within the soil mass without increasing the resulting earth pressure, A1, that has been considered in the design of the retaining structure.

NOTE 4 Figure D.8 illustrates a model used to determine anchor length to prevent interaction with retaining structure.



Key

ABCD	is the volume of soil comprised between the rear face of the retaining wall, AB, the conventional failure surface, BC, and the vertical surface intercepting the point C where the resulting anchor force is applied, CD;
W	is the effective weight of the volume ABCD;
F	is the destabilising force applied by the anchor on the volume ABCD;
A2	is the destabilising earth pressure applied on CD;
A1	is the stabilizing reaction applied by the retaining structure;
R	is the frictional component of the shear resistance of the soil on the failure surface BC;
C	is the additional shear resistance due to the cohesion.

Figure D.8 — Determination of anchor length to prevent interaction with retaining structure

(3) For grouted anchors, the resulting force exerted in the ground may be assumed to act in the middle of the fixed anchor length.

NOTE This assumption is relevant in standard ground conditions for which friction may be considered as uniformly distributed along the anchored length.

(4) If micropiles or other anchoring elements without a free length are used, an equivalent free length shall be determined before applying (2) and (3).

(5) The equivalent free length shall be consistent with the fixed anchor length along which friction is considered when verifying the bearing capacity of the micropiles according to 6.

Annex E
(informative)

Anchors

E.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to that given in Clause 8 regarding anchors.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

E.2 Scope and field of application

(1) This Informative Annex covers layout of anchors

E.3 Example for anchor design models

(1) The free anchor length should be determined during the design of the anchored structure.

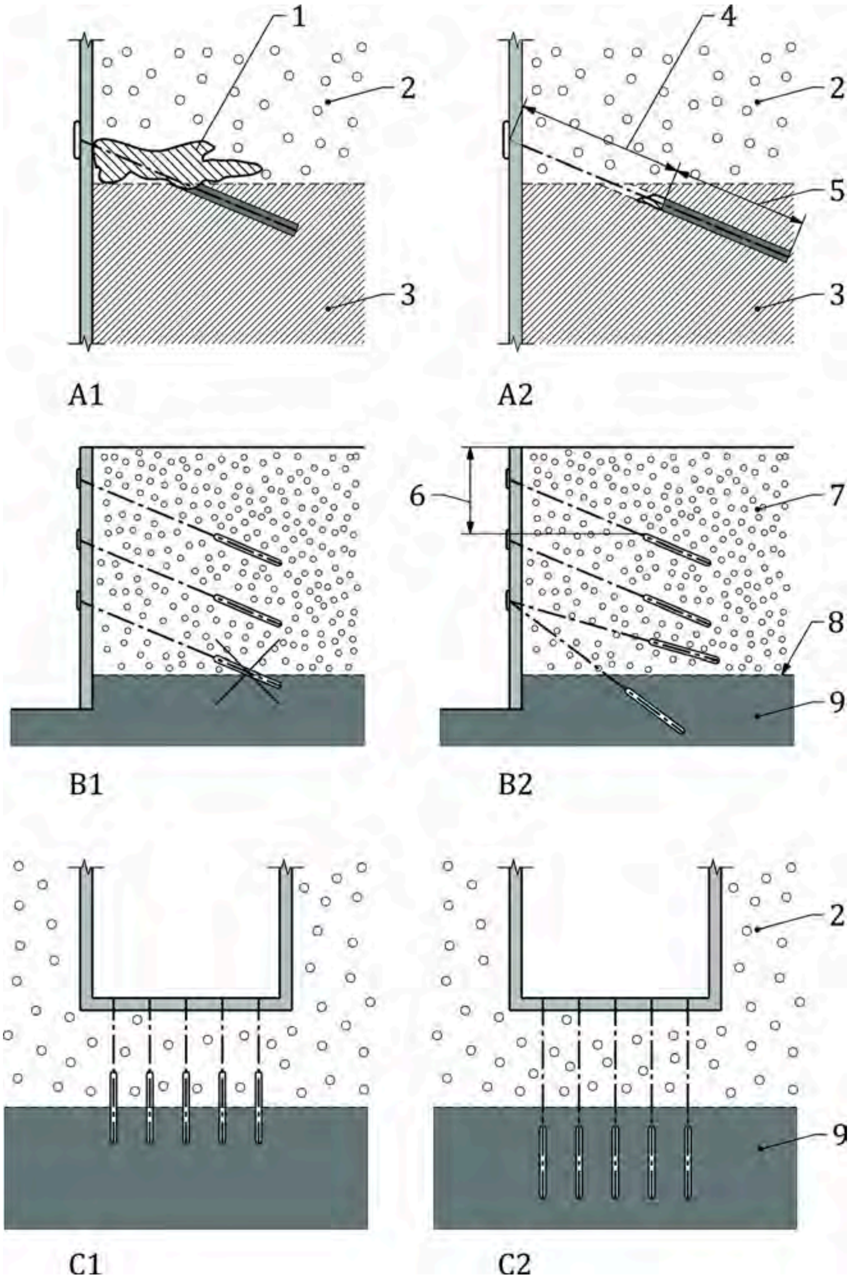
NOTE Examples of design models for anchored structures are given in Annexes A and D.

E.4 Layout of anchors

(1) The layout of anchors should consider the proximity of the load-bearing stratum and the execution.

NOTE 1 Examples of the configuration of anchors are given in Figure E.1, Figure E.2, and Figure E.3.

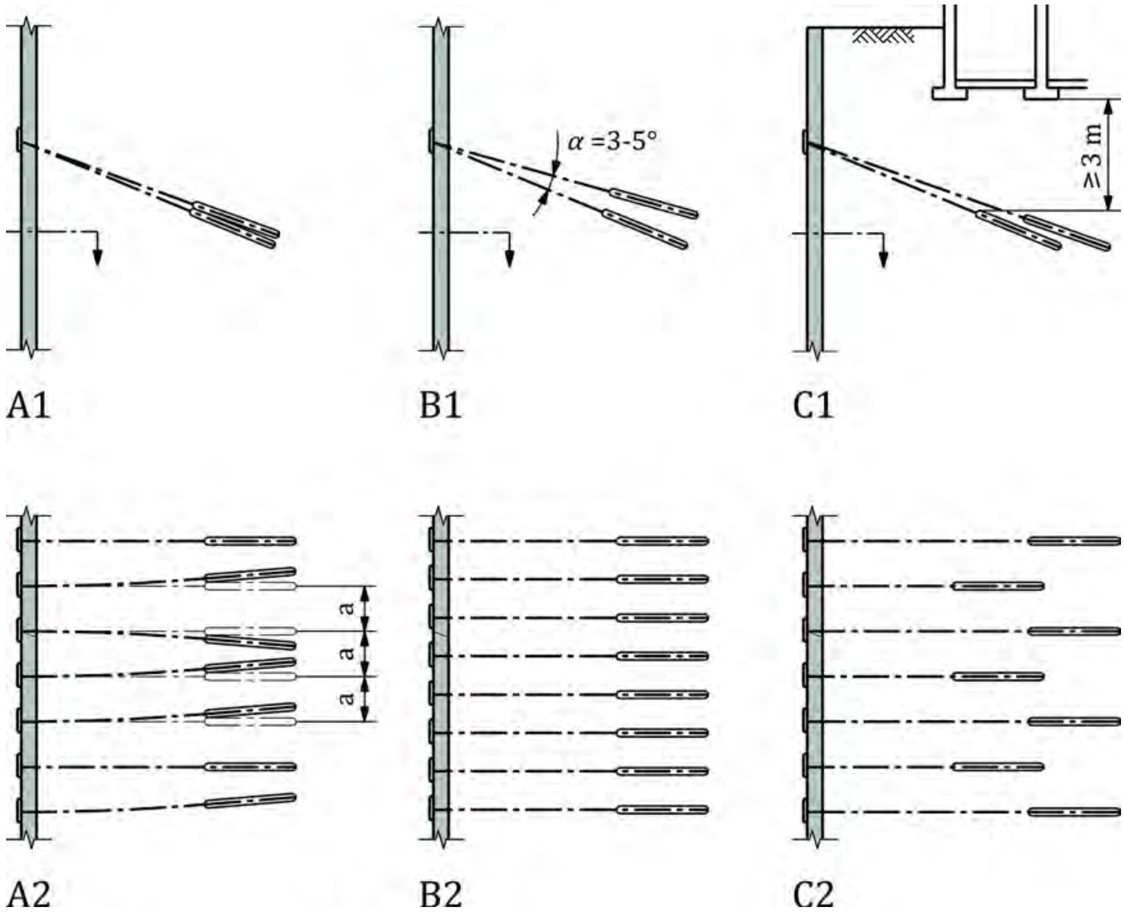
NOTE 2 In Figure E.3(a), all the grout bodies are outside the active earth pressure wedge. There is no additional earth pressure to the retaining wall. If the grout bodies are very close to the support (see Figure E.3(b)), additional earth pressure act.



Key

1	grout input info borehole and gravel	6	>4 m
2	gravel	7	sand
3	silt	8	transition zone
4	L_{free}>5 m	9	clay
5	L_{fixed}		

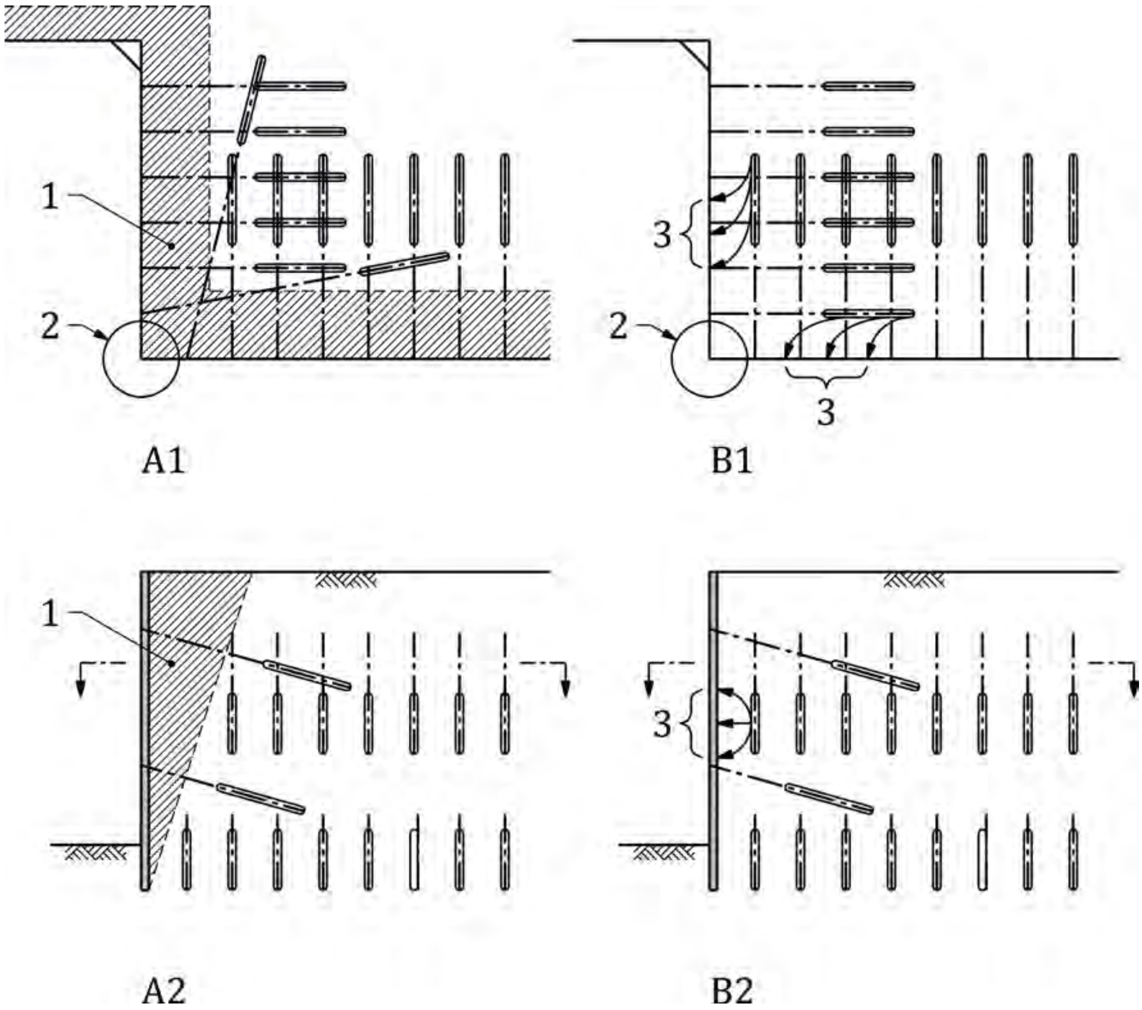
Figure E.1 — Examples of good (right side) and bad (left side) anchor configurations in stratified ground



Key

A1	PLAN: wrong	B1	PLAN: right	C1	PLAN: right
A2	SECTION: wrong	B2	SECTION: right	C2	SECTION: right

Figure E.2 — Examples of good and bad spreading and staggering of anchors



Key

A1	PLAN: $E=E_a$	B1	PLAN: $E>E_a$	1	Active earth pressure wedge
A2	SECTION: $E=E_a$	B2	SECTION: $E>E_a$	2	Corner designed to transfer tension
				3	Additional arth pressure

Figure E.3 — Examples of anchoring a protruding wall corner

Annex F
(informative)

Reinforced fill structures

F.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to that given in Clause 9 for reinforced fill structures.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

F.2 Scope and field of application

(1) This Annex covers calculation models for reinforced fill structures.

F.3 Calculation models for reinforced fill structures

F.3.1 Method of slices for slip surface analysis

(1) Slip surface analysis using the method of slices may be used for verifying internal and compound stability.

(2) In the case of reinforced slopes, the horizontal interslice forces may be ignored only if (3) is applied as well.

(3) It may be assumed that reinforcement elements are only considered where they intersect the assumed failure surface on a particular slice only if (2) is applied as well.

(4) The force applied in slip surface analysis to account for reinforcement elements should be limited to the resistance of the reinforcement element (see Figure F.1(a)).

(5) The force change due to its distribution within the particular slice should be added to the forces acting on that particular slice (see Figure F.1(b)).

NOTE Figure F.1 illustrates implementation of forces from reinforcing element into the method of slices.

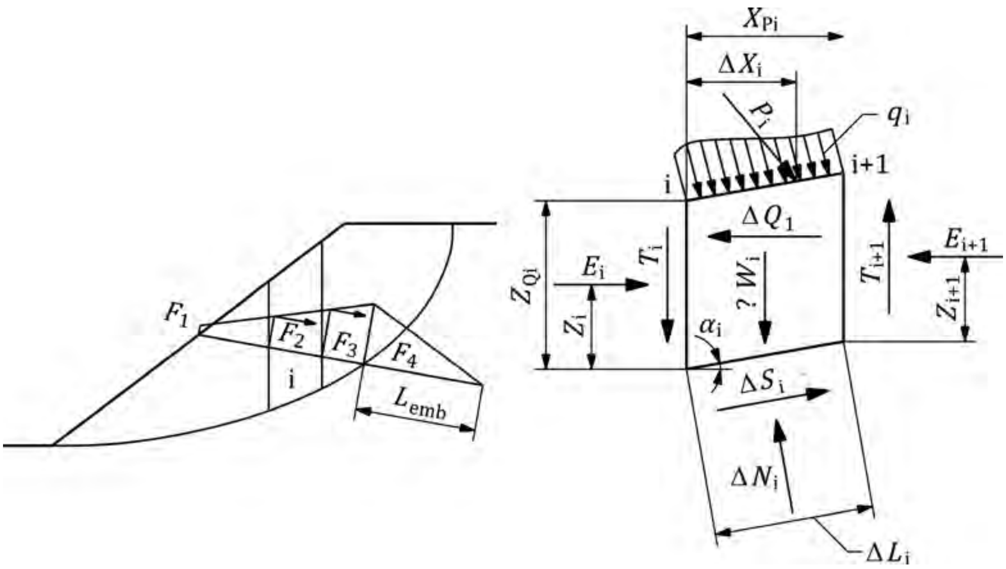


Figure F.1 — Forces from reinforcing element – implementation into method of slices

F.3.2 Coherent gravity method

(1) The coherent gravity method may be used for direct calculation of the load in each layer of soil reinforcements for internal stability check.

(2) The coherent gravity method may be used for non-extensible reinforcement that develops its tensile design strength at a strain < 1 %.

NOTE Figure F.2 illustrates the coherent gravity method.

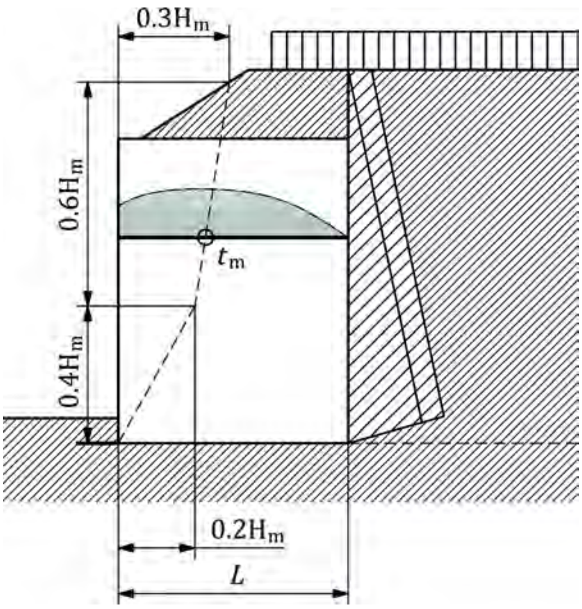


Figure F.2 — Coherent gravity method

(3) The coherent gravity method may be used.

(4) The stress state within the reinforced soil block should be taken to be proportional to K_0 at the effective ground surface reducing to K_a at a depth of 6 m.

(5) The maximum tensile force T_j to be resisted by the jth layer of reinforcement (at a depth of h_j from the top of the wall) should be determined from Formula (F.1F.1):

$$T_j = T_{p,j} + T_{s,j} + T_{f,j} = K\sigma_{v,j}S_{v,j} + T_{s,j} + T_{f,j} = K\left(\frac{R_{v,j}}{L_j - 2e_j}\right)S_{v,j} + T_{s,j} + T_{f,j} \tag{F.1}$$

(6) The line of maximum tension in the reinforcement should be assumed as indicated on Figure F.2.

$T_{p,j}$ is the tensile force per metre width due to the vertical loads of self-weight and UDL surcharge;

$T_{s,j}$ is the tensile force per metre width due to any strip loading;

$T_{f,j}$ is the tensile force per meter width due to any horizontal loads;

K is the earth pressure coefficient within the reinforced soil block at the depth of the jth layer of reinforcement;

$\sigma_{v,j}$ is the vertical stress on the jth layer of reinforcements;

$S_{v,j}$ is the vertical spacing of the reinforcements at the jth level in the wall; = $|h_{j+1} - h_{j-1}|/2$

$R_{v,j}$ is the resultant vertical load excluding external strip loads on the jth layer of reinforcement

L_j is the length of the j-th layer of reinforcement

e_j is the eccentricity of the resultant vertical load at the level of the jth layer of reinforcement.

(7) The tensile resistance of a reinforcing element at the line of maximum tension in the j-th layer shall be greater than the maximum tensile force T_j.

NOTE Detailed calculation procedure of coherent gravity method can be found in NF P 94 270.

F.3.3 Tie-back wedge method

(1) The tie-back wedge method may be used for direct calculation of the load in each layer of soil reinforcements for internal stability check.

(2) The tie-back wedge method may be used for extensible reinforcement that develops its tensile design strength at a strain > 1 %.

(3) The stress state within the reinforced soil block should be taken to be proportional to K_a for all reinforcement layers.

(4) The verification of tensile resistance of a reinforcing element should comply with F.3.2 and Formula (F.1) with K equal to K_a, where the influence of the eccentricity of the resultant vertical load is not considered.

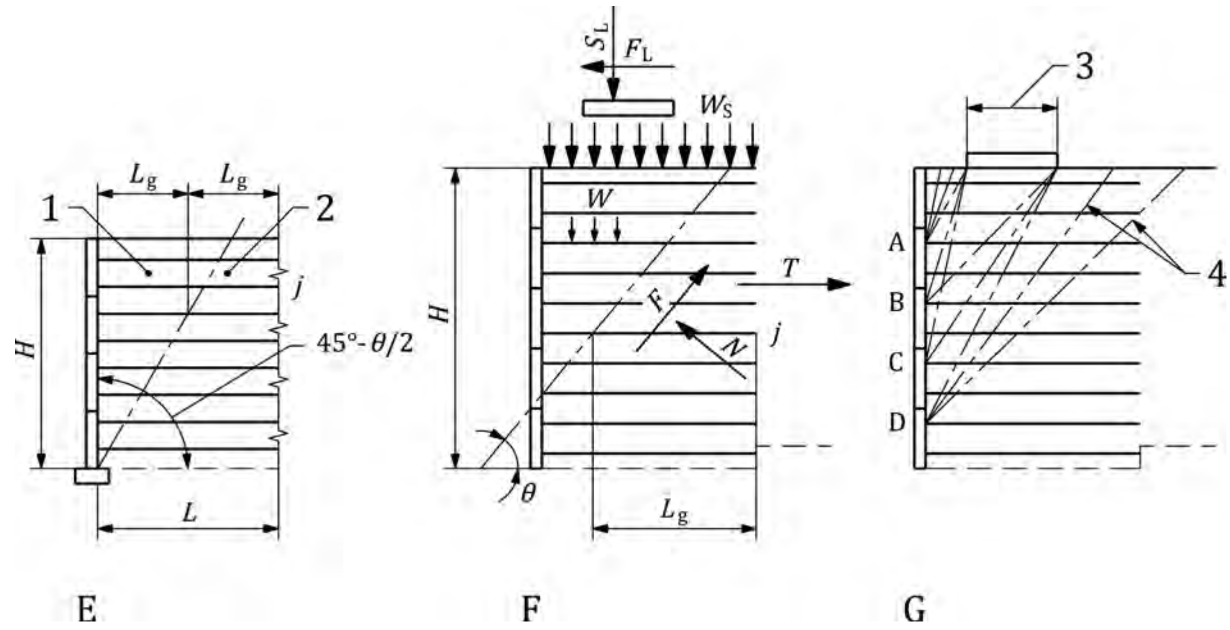
(5) The stability of a series of potential straight line failure planes forming wedges through the reinforced soil block should also be checked considering beneficial effect from the tensile resistance within each reinforcement layer that crosses the failure plane (see Figure F.3).

(6) The tensile resistance of each reinforcing element shall comply with 9.6.2.

NOTE 1 Detailed calculation procedure can be found in BS 8006-1.

NOTE 2 Figure F.3 illustrated the Tie-back wedge method.

(7) The tie-back wedge method may be used for internal and compound stability check.



Key

E …

F ..

G …

1 Active zone

2 Passive zone

3 Foundation width

4 Potential failure surface

Figure F.3 — Tie-back wedge method

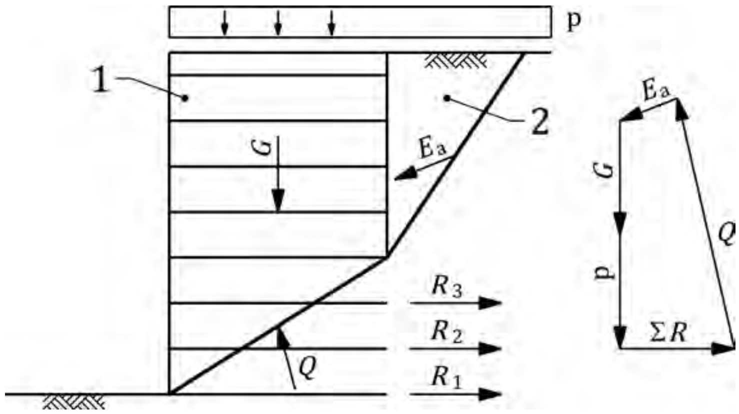
F.3.4 Multi-part wedge method

(1) The multi-part wedge method may be used for internal and compound stability check.

(2) If the potential failure mechanism is assumed to be a two-part wedge, the lower part of the wedge (Prism 1) should pass through the reinforced soil structure and the upper part of the wedge (Prism 2) through the retained (unreinforced material) behind it (see Figure F.4).

(3) The stability of any combination of wedges should be checked accounting for beneficial effect from the reinforcing elements in each layer cut by the failure plane of any wedge.

NOTE Figure F.4 illustrates the two-part wedge method.



Key

1 Wedge 1

2 Wedge 2

Figure F.4 — Two-part wedge method

F.4 Calculation models for reinforced embankment bases

F.4.1 Resistance to transverse sliding

(1) The lateral sliding stability of the embankment should be determined by examining any preferential slip surfaces that pass above the basal reinforcement layers.

(2) The lateral thrust F_{lt} from the embankment fill should be determined from Formula (F.2):

$$F_{\mathrm{lt}} = 0.5K_a H(\gamma H + 2W_{\mathrm{s}}) \tag{F.2}$$

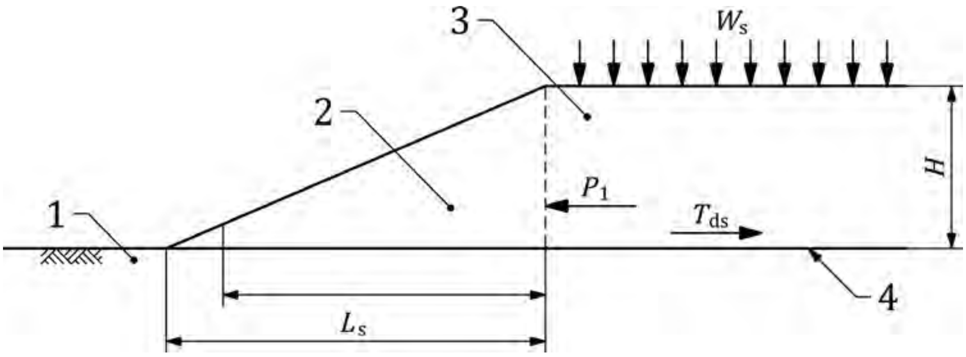
where:

- K_a is an active pressure coefficient;
- γ is the weight density of the fill;
- H is the height of the embankment; and
- W_s is the surcharge load.

(3) The tensile resistance of the reinforcing elements shall be greater than the lateral thrust.

(4) The sliding resistance along the top of the reinforcement layers beneath the embankment side slope shall be greater than the lateral thrust below the embankment crest from (2) (see Figure F.5).

NOTE Figure F.5 illustrates a calculation model to determine resistance to sliding.



Key

1 Soil foundation

2 Embankment

3 Fill

4 Reinforcement

P1 Lateral thrust from formula (F.2)

T_{ds} Is $R_{t,el}$

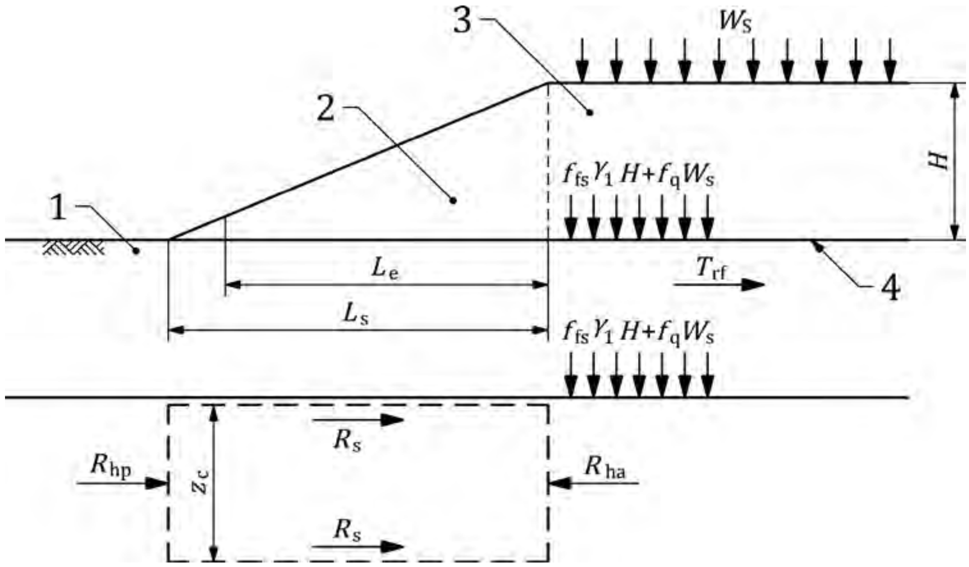
Figure F.5 — Calculation model to determine resistance to sliding

F.4.2 Resistance to foundation extrusion

(1) Where the thickness of low strength fine foundation soil is relatively small compared to the embankment width (thickness ≤ 0.25 embankment width) foundation extrusion, squeezing, should be determined.

(2) The side slope of the embankment should be long enough to develop resistance to prevent the mobilization of the outward shear stresses in the foundation soils (see Figure F.6).

NOTE Figure F.6 illustrates a calculation model to determine resistance to extrusion.



Key

1 Soil foundation

2 Embankment

3 Fill

4 Reinforcement

Figure F.6 — Calculation model to determine resistance to extrusion

(3) The minimum side slope length required should be determined using Formula (F.3):

$$L_e = \frac{(\gamma H + W_s - 4c_u)z_c}{(1 + \alpha'_{ds})c_u} \tag{F.3}$$

where:

γ	is the unit weight of the embankment fill;
H	is the maximum height of the embankment;
W_s	is the surcharge load;
c_u	is the undrained shear strength of the soft foundation soil;
z_c	is the depth of the foundation soil when the depth is limited and c_u is constant throughout;
α'_{ds}	is a soil/reinforcement interaction coefficient relating to c_u.

F.5 Calculation models for load transfer platform over rigid inclusions

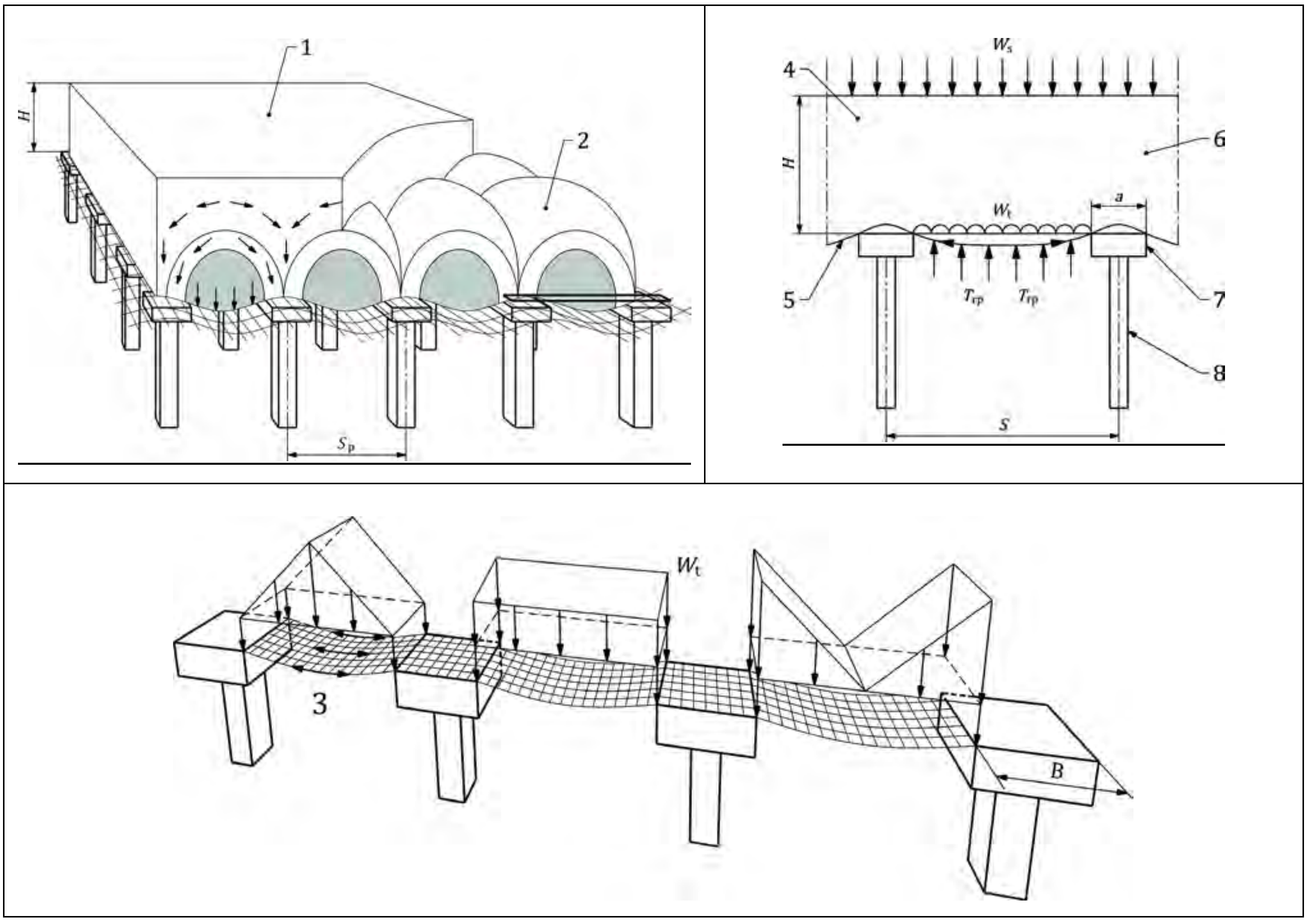
F.5.1 General

(1) Basal reinforcement should be designed to transfer the load from the embankment onto the discrete inclusions.

NOTE Figure F.7 give a schematic concept of load transfer platform over discrete inclusions.

(2) The part of the load from the embankment weight γH and surface surcharge w_s that acts on the reinforcement should be determined by different calculation methods.

(3) The tensile force FLTP shall be smaller than tensile resistance in the reinforcement determined from isochronous creep curves for specified limiting strain for an analysed limit state.



Key

1	Fill	5	Reinforcement
2	Soil arching	6	Embankment
3	ε, strain	7	Pile cap
4	Fill	8	Pile

Figure F.7 — Schematic concept of a load transfer platform over discrete inclusions

F.5.2 Hewlett and Randolph method

(1) In the Hewlett and Randolph method, the surcharge on the load transfer platform strips between adjacent inclusion caps should be assumed to be uniform.

NOTE In Figure F.7 the surcharge on the load transfer platform is W_T.

(2) For geosynthetic reinforcement that allows some deformation, the tensile force F_{LTP} in a reinforcing element should be determined from Formula (F.4):

$$F_{\mathrm{LTP}} = \frac{W_T(s_{\mathrm{p}} - B)}{2B}\sqrt{1 + \frac{1}{6\varepsilon}} \quad \text{(F.4)}$$

where:

W_T is the vertical uniformly distributed load on the reinforcement;

s_p is the centre to centre spacing of the inclusions;

B is the breadth of the inclusion cap or inclusion diameter;

ε is the limiting strain in the reinforcement.

NOTE 1 This formula assumes that there is no support from underlying low bearing strata.

NOTE 2 Detailed information about the Hewlett and Randolph method can be found in BS 8006-1.

F.5.3 EBGEO method

(1) In the EBGEO method, the surcharge on the load transfer platform shall be assumed to be triangular.

NOTE In Figure F.7 the surcharge on the load transfer platform is W_T.

(2) The determination of surcharges and resistances of individual system elements should be determined by iterative calculation procedure.

NOTE Details of the calculation procedure can be found in EBGEO.

F.5.4 Concentric Arches method

(1) In the Concentric Arches method, the surcharge on the load transfer platform shall be assumed to have a shape of inverse triangle or uniform load with respect to embankment height and subsoil resistance support.

NOTE In Figure F.7 the surcharge on the load transfer platform is W_T.

(2) Surcharges and resistances of individual system elements should be determined by an iterative calculation procedure.

NOTE Details of the calculation procedure can be found in CUR 226.

F.6 Calculation models for embankments over voids

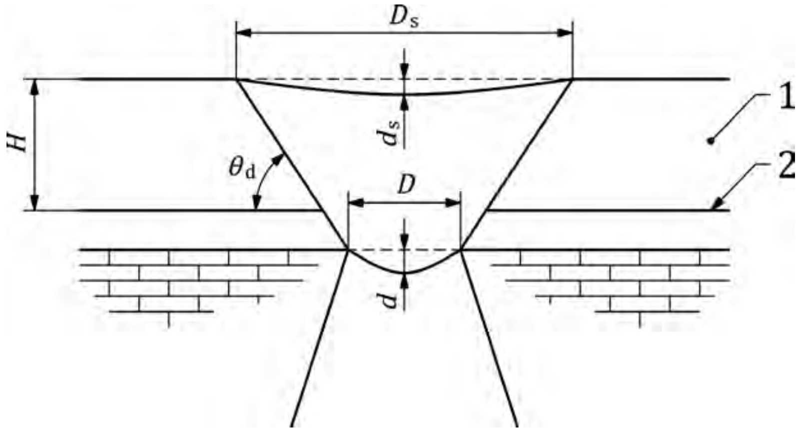
(1) In areas prone to the development of voids or deep depressions soil reinforcement may be used to provide a short term indicating function or a long term permanent solution.

(2) The design void diameter should be assumed based on comparable experience.

(3) The maximum differential settlement of the ground surface above a void should be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE The maximum differential settlement is typically 1-7 % for roads, depending on the class of road. For railways, it is typically < 0.5 %, depending on the permitted speed of the trains.

NOTE Figure F.8 illustrates the parameters required for using Formula F.6)



Key

1 Embankment

2 Reinforcement

ds Depression at surface

d Depression at reinforcement

Figure F.8 — Parameters required for Formula (F.6)

(4) Provided the deformed shape of the geosynthetic reinforcement is parabolic, the strain in the reinforcement layer ε shall be determined from Formula (F.5):

$$\varepsilon = \frac{8}{3}\left(\frac{d_s}{D_s}\right)^2 \tag{F.5}$$

where:

d_s is the deformation at the surface; and

D_s is the diameter of the depression at the surface,

(5) The tensile force F_{vo} in the geosynthetic reinforcement for a circular void and for case a of Figure F.8 shall be determined from Formula (F.6):

$$F_{vo} = 0.5(\gamma H + w_s)D\sqrt{1 + 1/6\varepsilon} \tag{F.6}$$

where:

H is the height of material above the geosynthetic layer;

w_s is the surcharge;

D is the diameter of the void at the level of the geosynthetic layer;

γ is the weight density of the embankment fill;

ε is the reinforcement strain given in Formula (F. 5).

(6) For cases b and c shown in Figure F.8, more complex calculation procedures should be followed to determine the force F_{vo}.

NOTE For further details, see EBGEO.

(7) As an alternative to (6), cases b and c may be analysed using a different method provided it has been calibrated and validated against comparable experience.

(8) The tensile force F_{vo} shall be smaller than tensile resistance in the reinforcement determined from isochronous creep curves for specified limiting strain for an analysed limit state.

F.7 Veneer reinforcement

(1) The stability of a soil veneer above a potential sliding plane should be determined by assuming a tension crack at the top of the slope and a resistant passive wedge at the toe.

(2) The contribution of friction down the slope should take the value of the lowest frictional interaction between the multiple layers that form the veneer system.

NOTE Veneer systems can be made up of multiple synthetic and mineral layers with different frictional characteristics.

(3) The tensile force T_{ven} required to hold the veneer system on the slope without water should be determined from Formula (F.7):

$$T_{ven} = W_A \sin\beta - W_A \cos\beta \tan\delta - C_A - \frac{C_P + W_P \tan\varphi}{\cos\beta - \sin\beta \tan\varphi} \tag{F.7}$$

where:

T_{ven}	is the tensile force to hold the veneer system on the slope without water	C_P	is the cohesion along the passive wedge
W_A	is the weight of the active wedge	β	is the inclination of the ground surface
W_P	is the weight of the passive wedge	δ	is the ground-structure interface friction angle
C_A	is the cohesion along the active wedge	φ	is the soil's angle of friction

(4) The tensile force F_{ven} shall be smaller than tensile resistance in reinforcing element for an analysed limit state.

(5) The stability of the horizontal anchorage at the top of the veneer without water should be verified using Formula (F.8):

$$T_{\text{ven}} \leq \left(\frac{T_{\text{ven}} \sin\beta}{L_{\text{ds}}} + \gamma_{\text{cs}} d_{\text{cs}}\right) f_{\text{ds}} L_{\text{ds}} \tag{F.8}$$

where:

T_{ven}	is the tensile force to hold the veneer system on the slope without water	d_{cs}	is the depth of the cover soil

L_{ds}	is the pullout (fixed) length of the veneer reinforcement	β	is the inclination of the ground surface
γ_{cs}	is the weight density of the coversoil	f_{ds}	is the direct shear factor

(6) When water is present or a different shape of anchorage is used, Formulae (F.8) and (F.9) should be amended accordingly.

NOTE Additional details on calculation procedure are given by Rimoldi (2018).

F.8 Durability, reduction factor for tensile strength

F.8.1 Reduction factors for geosynthetic reinforcing element

NOTE See 9.3.3

(1) The value of the reduction factor for tensile strength of geosynthetic reinforcement, η_{gs} shall be determined from Formula (F.9):

$$\eta_{gs} = \eta_{cr} \cdot \eta_{dmg} \cdot \eta_{w} \cdot \eta_{ch} \cdot \eta_{dyn} \cdot \eta_{con} \tag{F.9}$$

where:

η_{cr} is a factor accounting for the adverse effect of tensile creep due to sustained static load over the design service life of the structure at the design temperature;

η_{dmg} is a factor accounting for the adverse effects of mechanical damage during transportation, installation and execution;

η_{w} is a factor accounting for the adverse effects of weathering;

η_{ch} is a factor accounting for the adverse effects of chemical and biological degradation of the reinforcing element over the design service life of the structure at the design temperature;

η_{dyn} is a factor accounting for the adverse effects of intense and repeated loading over the design service life of the structure (fatigue);

η_{con} is a factor accounting for the adverse effects of joints and seams for geosynthetic reinforcing elements and polymeric coated steel woven wire mesh.

NOTE 1 The values of η_{cr}, η_{dmg}, η_{w}, and η_{ch} are the reciprocals of the reduction factors specified in ISO TR 20432, as RFCR, RFID, RFW, and RFCH, respectively.

NOTE 2 The value of η_{dyn} is the reciprocal of the reduction factor specified in EBGEO as A_5.

NOTE 3 The value of η_{con} is the reciprocal of the reduction factor specified in EBGEO as A3, based on test complying with EN ISO 10321.

NOTE 4 Values of η_{cr}, η_{dmg}, η_{w}, and η_{ch} is given in ISO TR 20432 and values of η_{dyn} is given in EBGEO, unless the national annex gives different values.

NOTE 5 For short term or rapid loading η_{cr} can be modified in accordance with ISO TR 20432 to allow for the nature of the applied load.

NOTE 6 η_{cr} include creep strain based on isochronous curves, to allow for creep and limiting elongation.

NOTE 7 The factor η_w can have a value less than 1,0 if the reinforcement is not covered by soil within one day of installation.

F.8.2 Reduction factors for steel woven wire meshes

NOTE See 9.3.5

(1) The value of the reduction factor for tensile strength of steel woven wire meshes, η_{pwm} shall be determined from Formula (F.10):

$$\eta_{\text{pwm}} = \eta_{\text{dmg}} \cdot \eta_{\text{cor}} \tag{F.10}$$

where:

η_{dmg} is a reduction factor accounting for the adverse effects of mechanical damage during transportation, installation and execution;

η_{cor} is a reduction factor accounting for the adverse effects of degradation of the element by corrosion over the design service life of the structure, corrosion being triggered by the local loss of watertightness due to chemical degradation of the polymeric coating and/or the loss of the Zinc or Zinc/Aluminium layer by corrosion.

NOTE 1 The value of η_{cor} is determined by testing standard to be developed.

NOTE 2 The values of η_{dmg} is the reciprocal of the reduction factor specified in ISO TR 20432, as RF_{ID}.

NOTE 3 The value of η_{dmg} can have a value lower than 1.0 only if the steel wires get damaged during execution, while damage to the coating is irrelevant for the decrease of tensile strength at short term and is accounted in the determination of η_{cor}. The damage of the coating is considering in η_{cor} as it will induce corrosion of the exposed wire.

F.9 Typical grades of steel used for soil reinforcement elements

F.9.1 General

(1) This clause provides complementary guidance to 9.3.4 for typical grades of steel used for tension elements in reinforced fill structures and applies to tension elements for reinforced structures only.

F.9.2 Grades of steel used for tension elements

(1) Tension elements may be made using any of the steel grades given in Table F.9.1.

(2) Other grades of steel may be used, provided they comply with the provisions of 9.3.4.

Table F.9.1 — Typical grades of steels used for tension elements

Type of Steel	Relevant Standard	Steel Name	Yield strength[a]		Tensile Strength[b]		Strength distribution across section
			symbol	N/mm²	symbol	N/mm²	
Hot-rolled strips	EN 10025-2	S235	f_y	235	f_u	360-510	uniform
		S355	f_y	355	f_u	470-630	
		S460	f_y	460	f_u	550-720	
Reinforcing steel	EN 10080	B400B[c]	$f_{0.2k}$	400	f_{tk}	≥432	non-uniform (unless otherwise demonstrated by testing)
		B450B[c]	$f_{0.2k}$	450	f_{tk}	≥486	
		B500B[c]	$f_{0.2k}$	500	f_{tk}	≥542	
		B550B[c]	$f_{0.2k}$	550	f_{tk}	≥594	
		B600B[c]	$f_{0.2k}$	600	f_{tk}	≥648	

a Values stated are minimum where $f_{0.2k}$ = $R_{p0.2}$ (specified proof strength at 0.2 % strain) and f_{tk} = R_m (specified tensile strength) in accordance with EN 10080

b The grades shown are common, commercially available, grades. Consult with manufacturers for available diameters.

c Minimum ductility Class B according to Table 5.5 of prEN 1992-1-1:2021

Annex G
(informative)

Ground improvement

G.1 Use of this Informative Annex

(1) This Informative Annex provides complementary guidance to that given in Clause 11 for ground improvement.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

G.2 Scope and field of application

(1) This Informative Annex:

- gives examples of diffused ground improvement techniques in Table G.1;
- gives examples of discrete ground improvement techniques in Table G.2;
- indicates which European execution standards (if any) apply to each technique.

G.3 Examples of ground improvement techniques

NOTE Table G.1 and Table G.2 give typical families and classes used for design.

Table G.1 — Examples of diffused ground improvement techniques

Method	**Technique**	**Family and Class**	**Description**	**Execution Standard**
Grouting Methods	Permeation grouting	AII	Replacement of interstitial water or gas of a porous medium with a grout, also known as “impregnation” grouting. Suitable for a wide range of soils to considerable depths.	EN 12715
	Jet grouting	AII	Hydraulic disaggregation of soils using high velocity jets.	EN 12716
	Compaction grouting	AI	Displacement grouting method which is the injection of a medium/low slump mortar into the soil to compact/densify it by expansion alone. Suitable for a wide range of soils to considerable depths.	EN 12715
Compactive Methods	Deep vibration	AI	Densification of generally granular soil by the insertion of a vibrating poker. Significant depths of suitable soils can be treated and marine operation is possible.	EN 14731
	Dynamic compaction	AI	Densification of soil by the impact of heavy weights from significant heights. Significant depths of suitable soils can be treated and marine operation is possible.	None
	Impact roller compaction	AI	Compactive effort provided by a non-circular roller, usually three or four sided. Only shallow depths of suitable soils can be treated.	None

Method	Technique	Family and Class	Description	Execution Standard
	Rapid impact compaction	AI	Compactive effort provided by weight dropping with a rapid control mechanism usually mounted on a vertical arm. Shallow/medium depths of suitable soils can be treated.	None
	Micro-blasting	AI	Compactive effort provided by detonating small charges of explosive at depths below ground level. The weight and arrangement of explosive charge is tailored to the depth and type of soil present. It can be used over water and can treat considerable depths.	None
	Compaction grouting	AII	Injection of grout into a host medium or ground in such a manner as to deform, compress, or displace the ground.	None
Soil Replacement	Soil replacement	I	Replacement of unsuitable soil with engineered materials with or without georeinforcement. Depth limited by excavation stability.	None
Thermal Methods	Ground freezing	AII	Freezing of interstitial water within soils to create hardened bodies of significant strength and very low hydraulic conductivity. More suitable for granular soils but can be used in cohesive soils with care due to potential soil expansion.	None
	Ground heating	AI AII	The use of thermal methods to generally remove water from fine grained soils with a resultant increase in strength. Ultimately with very high temperatures, soil can be fused in a rock like structure.	None
Consolidation Methods	Surcharge	AI	Use of additional load in advance of construction, generally on soft clays, to force consolidation and reduce long term residual settlements	None
	Vertical drains & surcharge	AI	Use of sand or prefabricated geotextile drains in combination with surcharge to reduce drainage paths within soft cohesive soils to force accelerated consolidation and accelerated groundwater pressure dissipation during construction in order to reduce overall programme and to reduce residual long-term settlements. Land and marine based rigs available to considerable depths.	EN 15237
	Dewatering	AI	Lowering of the ground water table or depressurisation of the groundwater pressure within soils to increase effective strength, force consolidation and reduce long term residual settlements.	None
	Vacuum consolidation	AI	Use of a vacuum instead of surcharge in advance of construction, generally on soft cohesive soils, to force accelerated consolidation and accelerated groundwater pressure dissipation during construction in order to reduce overall programme and to reduce residual long-term settlements.	EN 15237
Mixing Methods	Dry methods	AII	Mechanical disaggregation of soils while introducing a dry binder pneumatically and commonly cement. Most usually	EN 14679

Method	Technique	Family and Class	Description	Execution Standard
			executed highly compressible fine grained soil. Land and marine based rigs available to considerable depths.	
	Wet methods	AII	Mechanical disaggregation of soils while introducing a fluid binder. Generally more powerful system than the dry system and can be executed in various type of soils. Land and marine based rigs available to considerable depths.	EN 14679
	Jet grouting	AII	Hydraulic disaggregation of soils using high velocity jets of fluid binder combined or not with either water or water and air. Suitable for most soils and available for land or marine use to considerable depths.	EN 12716

Table G.2 — Examples of discrete ground improvement techniques

Method	Technique	Family and Class	Description	Execution Standard
Mixing Methods	Dry methods	BII	Mechanical disaggregation of soils while introducing a dry binder pneumatically and commonly cement. Most usually executed highly compressible fine grained soil. Land and marine based rigs available to considerable depths.	EN 14679
	Wet methods	BII	Mechanical disaggregation of soils while introducing a fluid binder. Generally more powerful system than the dry system and can be executed in various type of soils. Land and marine based rigs available to considerable depths.	EN 14679
	Jet grouting	BII	Hydraulic disaggregation of soils using high velocity jets of fluid binder combined or not with either water or water and air. Suitable for most soils and available for land or marine use to considerable depths.	EN 12716
Granular Inclusions	Stone columns/ Vibro-replacement	BII	Compacted stone columns are created in the ground to form a composite ground with the surrounding soil. Most often used in soft cohesive soils but in granular soils as well to improve strength and stiffness of the overall system and accelerate drainage with possible densification of the surrounding soil depending on the soil type. Land and marine based rigs available to considerable depths.	EN 14731
	Sand columns/ Sand compaction piles	BI	Compacted sand columns are created in the ground to form a composite ground with the surrounding soil. Most often used in soft cohesive soils but in granular soils as well to improve strength and stiffness of the overall system and accelerate drainage with possible densification of the surrounding soil depending on the soil type. Land and marine based rigs available to considerable depths.	EN 14731
	Dynamic replacement	BI	The use of dynamic compaction to drive bulbs of granular material into soft soils thereby both improving the soil by the dynamic compaction and the introduction of competent granular piers. Most often used in soft cohesive soils to	None

Method	Technique	Family and Class	Description	Execution Standard
			improve strength and stiffness of the overall system and accelerate drainage. Land and marine based rigs available.	
	Geosynthetics encased columns	BI	Stone or sand columns, encased in a geotextile casing, formed in very soft soils where the lateral restraint is too small to prevent very significant column bulging. The geotextile casing provides support to the columns and prevents excessive bulging under load. Land and marine based rigs available to significant depths.	None
Steel/Wood Inclusions	Vibrated	BII	Rigid columns of steel or wood are vibrated into the ground, with possible densification effort to the existing ground depending on the soil type, to form a composite ground with various type of soil and providing support to the structure above through load distribution between the soil and inclusions. Land and marine based rigs available to considerable depths.	None
	Bored	BII	Rigid columns of steel or wood are bored into the ground, sometimes with associated compactive effort, to form a composite ground with various type of soil and providing support to the structure above through load distribution between the soil and inclusions. Land and marine based rigs available to considerable depths.	None
	Driven	BII	Rigid columns of steel or wood are driven into the ground, causing some densification, to form a composite ground with various type of soil and providing support to the structure above through load distribution between the soil and inclusions. Land and marine based rigs available to considerable depths.	None
Concrete/ Grout Inclusions	Vibrated concrete columns	BII	An improvement method whereby columns of concrete or mortar are backfilled in the ground during withdrawal of a vibrating pipe or poker to form a composite ground with various type of soil, providing support to the structure above through load distribution between the soil and inclusions possible densification effort to the existing ground depending on the soil type.	None
	Bored	BII	An improvement method whereby columns of concrete or mortar are backfilled in the ground during withdrawal of a boring auger to form a composite ground with various type of soil, providing support to the structure above through load distribution between the soil and inclusions sometimes with associated compactive effort to the existing ground.	None
	Driven	BII	An improvement method whereby columns of concrete or mortar are backfilled in the ground during withdrawal of a driven pipe to form a composite ground with various type of soil, providing support to the structure above through load distribution between the soil and inclusions and possible	None

Method	Technique	Family and Class	Description	Execution Standard
			densification effort to the existing ground depending on the soil type.	
	Grouted stone columns	BII	An improvement method whereby compacted and grouted stone columns are created in ground to form a composite ground with the surrounding soil. Providing support to the structure above through load distribution between the soil and inclusions and possible densification effort to the existing ground depending on the soil type. Land and marine based rigs available to considerable depths.	None
	Compaction grouting	BII	Injection of grout into a host medium or ground in such a manner as to deform, compress, or displace the ground.	None

G.4 Use of stress envelope to determine acceptable limit states

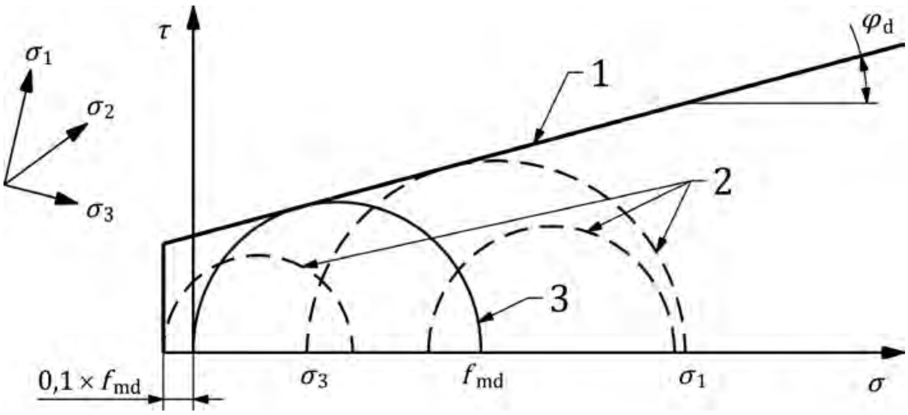
(1) When the design is based on the explicit calculation of the principal stresses it shall be verified that the design values pf the principal stresses do not exceed the states of stress defined in Figure G.1.

(2) In addition to (1) the principal tensile stress shall not exceed 10 % of $f_{m,d}$.

(3) For Class BII rigid inclusions subjected to eccentricities, resulting stresses within the cross section shall be verified to be within the stress envelope given in Figure G.1.

(4) When the design is not based on the explicit calculation of principal stresses, the design value of the normal stresses and of the shear stresses shall not exceed 0.7 $f_{m,d}$ and 0.2 $f_{m,d}$ respectively.

NOTE Figure G.1 illustrates the allowable stresses in rigid ground improvement material.



Key

1 Envelope for allowed states of stress

2 Examples for states of stress σ_1, σ_3, allowed

3 State of stress in a uniaxial compression test: $\sigma_3 = 0$, $\sigma_1 = f_{m,d}$

Figure G.1 — Allowable stresses in rigid ground improvement material

φ_d (strengthened soil) = φ'_d (unimproved soil)

$\tan \varphi_d = \tan \varphi_k / \gamma_\varphi$

Bibliography

References given in recommendations (i.e. "should" clauses)

The following documents are referred to in the text in such a way that some or all of their content constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative documents could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 206, *Concrete — Specification, performance, production and conformity*

EN 771-3, *Specification for masonry units — Part 3: Aggregate concrete masonry units (Dense and lightweight aggregates)*

EN 1536, *Execution of special geotechnical work — Bored piles*

EN 1538, *Execution of special geotechnical work — Diaphragm walls*

EN 1563, *Founding — Spheroidal graphite cast irons*

prEN 1992-1-1:2021, *Eurocode 2: Design of concrete structures — Part 1-1: General rules — Rules for buildings, bridges and civil engineering structures*

EN 1995-1-1:2004, *Eurocode 5: Design of timber structures — Part 1-1: General — Common rules and rules for buildings*

EN 10218-2, *Steel wire and wire products — General — Part 2: Wire dimensions and tolerances*

EN 10223-3, *Steel wire and wire products for fencing and netting — Part 3: Hexagonal steel wire mesh products for civil engineering purposes*

EN 10245-1, *Steel wire and wire products — Organic coatings on steel wire — Part 1: General rules*

EN 12063, *Execution of special geotechnical work — Sheet-pile walls*

EN 12390-2, *Testing hardened concrete — Part 2: Making and curing specimens for strength tests*

EN 12504-1, *Testing concrete in structures — Part 1: Cored specimens — Taking, examining and testing in compression*

EN 12699, *Execution of special geotechnical works — Displacement piles*

EN 12715, *Execution of special geotechnical work — Grouting*

EN 12716, *Execution of special geotechnical work — Jet grouting*

EN 13251, *Geotextiles and geotextile-related products — Characteristics required for use in earthworks, foundations and retaining structures*

EN 13670, *Execution of concrete structures*

EN 14081-1, *Timber structures — Strength graded structural timber with rectangular cross section — Part 1: General requirements*

EN 14199, *Execution of special geotechnical works — Micropiles*

EN 14487 (all parts), *Sprayed concrete*

EN 14488 (all parts), *Testing sprayed concrete*

EN 14490:2010, *Execution of special geotechnical works — Soil nailing*

EN 14679, *Execution of special geotechnical works — Deep mixing*

EN 15237, *Execution of special geotechnical works — Vertical drainage*

EN 15258, *Precast concrete products — Retaining wall elements*

EN 16907-5, *Earthworks — Part 5: Quality control*

EN ISO 12957 (all parts), *Geosynthetics — Determination of friction characteristics*

EN ISO 12958-1, *Geotextiles and geotextile-related products — Determination of water flow capacity in their plane — Part 1: Index test (ISO 12958-1)*

EN ISO 12958-2, *Geotextiles and geotextile-related products — Determination of water flow capacity in their plane — Part 2: Performance test (ISO 12958-2)*

EN ISO 13793, *Thermal performance of buildings — Thermal design of foundations to avoid frost heave (ISO 13793)*

EN ISO 14689, *Geotechnical investigation and testing — Identification, description and classification of rock (ISO 14689)*

EN ISO 22282-4, *Geotechnical investigation and testing — Geohydraulic testing — Part 4: Pumping tests (ISO 22282-4)*

EN ISO 22477 (all parts), *Geotechnical investigation and testing — Testing of geotechnical structures*

References given in permissions (i.e. "may" clauses)

The following documents are referred to in the text in such a way that some or all of their content expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EBGEO, Recommendations for design and analysis of earth structures using geosynthetic reinforcements, (English Translation) Deutsche Gesellschaft für Geotechnik, Ernst & Sohn, München, Germany, 2011.

EN 12716:2018, *Execution of special geotechnical work — Jet grouting*

ISO/TS 13434, *Geosynthetics — Guidelines for the assessment of durability*

References given in possibilities (i.e. "can" clauses) and notes

The following documents are cited informatively in the document, for example in "can" clauses and in notes.

prEN 1991-2:2022, *Eurocode 1 – Action on structures – Part 2 Traffic loads on bridges and other civil engineering works*

EN 1994-1-1:2004, *Eurocode 4: Design of composite steel and concrete structures — Part 1-1: General rules and rules for buildings*

prEN 1998 (all parts), *Eurocode – Design of structures for earthquake resistance*

ISO 6707-1, *Buildings and civil engineering works — Vocabulary — Part 1: General terms*

EN 12794, *Precast concrete products. Foundation piles*

EN 13369, *Common rules for precast concrete products*

EN 14731, *Execution of special geotechnical works — Ground treatment by deep vibration*

EN 16907 (all parts), *Earthworks*

EN ISO 10321, *Geosynthetics — Tensile test for joints/seams by wide-width strip method (ISO 10321)*

EN ISO 19901-4, *Petroleum and natural gas industries — Specific requirements for offshore structures — Part 4: Geotechnical and foundation design considerations (ISO 19901-4)*

EN ISO 22477-1:2018, *Geotechnical investigation and testing — Testing of geotechnical structures — Part 1: Testing of piles: static compression load testing (ISO 22477-1:2018, Corrected version 2019-03)*

EN ISO 22477-4:2018, *Geotechnical investigation and testing — Testing of geotechnical structures — Part 4: Testing of piles: dynamic load testing (ISO 22477-4:2018)*

EN ISO 22477-5:2018, *Geotechnical investigation and testing — Testing of geotechnical structures — Part 5: Testing of grouted anchors (ISO 22477-5:2018)*

EN ISO 22477-10:2016, *Geotechnical investigation and testing — Testing of geotechnical structures — Part 10: Testing of piles: rapid load testing (ISO 22477-10:2016)*

ISO/TR 20432, *Guidelines for the determination of the long-term strength of geosynthetics for soil reinforcement*

Bishop, A. W., The use of the slip circle in the analysis of slopes. Geotechnique, Vol 5, (1965) Proc. Europ. conf. on stability of earth slopes., Stockholm, Vol. I, pp 1-14 (1954).
Bjerrum and Eide, (1956), *Stability of strutted excavations in clay*, Géotechnique, Vol. 6, No. 1. pp. 32-47.

Brinch Hansen, J., (1961): *The Ultimate Resistance of Rigid Piles Against Transversal Forces*, Geoteknisk Institut, Copenhagen, Bulletin No. 12

BS 8006-1, *Code of practice for strengthened/reinforced soils and other fills.*

Coulomb, C.A., *Essai sur une application des regles des maximis et minimis a quelquels problemesde statique relatifs, a la architecture*. Mem. Acad. Roy. Div. Sav., Vol. 7, pp 343–387 (1776).

Cullman, *Die graphiscehe statik*. Zürich: Verlag van Meyer & Zeller (1866).

CUR226 (2016). Van Eekelen, S.J.M. & Brugman, M.H.A., Editors. D*esign Guideline Basal Reinforced Piled Embankments (CUR226)*. CRC press, Delft, the Netherlands (English translation). ISBN 9789053676240.

Goodman.R.E. and Shi.G. (1985). Block theory and its application to rock engineering. Englewood Cliffs, NJ, Prentice-Hall.

DIN 4084:2009-01, *Baugrund – Geländebruchberechnungen*. Berlin: Beuth Verlag;

Fredlund D.G., Krahn J., *Comparison of slope stability methods of analysis*, Canadian Geotechnical J. Vol. 29, no 3 (1977).

Froelich, O.H., *The factor of safety with respect to sliding of soil along the arc of a logarithmic spiral*. Proc. of the 3rd ICSMFE Conference, Vol. 2, Switzerland (1953).

Janbu, N., *Application of composite slip surfaces for stability analysis*. Proc. Europ. conf. on stability of earth Slopes., 1954 Stockholm, Vol. 3, pp 43-49 (1955).

Kérisel, J., Absi, E, *Active and passive earth pressure tables*, 3rd Edition, A.A.Balkema, Rotterdam, 1990.

Morgenstern, N. R., Price, V. W., *The analysis for the stability of general slip surfaces*. Geotechnique, Vol 15, pp 79-93 (1965).

NF P 94 270 *Calcul géotechnique - Ouvrages de soutènement - Remblais renforcés et massifs en sol cloué*.

Poisel,Rainer, and Preh, Alexander (2004), Rock slope initial failure mechanisms and their mechanical models, Felsbau 22, no 2

Rimoldi, P. (2018). *Semi-probabilistic approach to the stability of veneer reinforcement*. Proc. 11th International Conference on Geosynthetics, Seoul, Korea.

Sarma S.K., *Stability analysis of embankments and slopes*, Journal of the geotechnical engineering division. ASCE, Vol. 105, no GT12 (1979).

Spencer, E., *A method of analysis of the stability of embankments assuming parallel inter-slice forces*, Geotechnique, Vol. 17, pp 11-26 (1967).

Wyllie, Duncan C. (2017), Rock Slope Engineering, Civil Applications, Fifth Edition, CRC Press, 620pp.