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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

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Contents

Drafting foreword by PTs 6

This document (prEN 1997-3:20xx) has been prepared jointly by project teams M515/SC7.T6

This document is the Final Document of prEN 1997-3 (as required under Phase 4 of Mandate M/515).

This document is a working document.

European Foreword

[DRAFTING NOTE: this version of the foreword is relevant to EN Eurocode Parts for enquiry stage]

This document (EN 1997-3) 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 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.

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>*

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

[Drafting note: This contains information formerly included in “Scope of EN 1997”]

EN 1997 is intended to be used in conjunction with EN 1990, which establishes principles and requirements for the safety, serviceability, robustness, and durability of structures, including geotechnical structures, and other construction works.

EN 1997 establishes additional principles and requirements for the safety, serviceability, robustness, and durability of geotechnical structures.

EN 1997 is intended to be used in conjunction with the other Eurocodes for the design of geotechnical structures, including temporary geotechnical structures.

Design and verification in EN 1997 are based on the partial factor method, prescriptive rules, testing, or the observational method.

0.3 Introduction to EN 1997-3

[This contains "Additional information specific to EN 19xx" taken from current Eurocode parts and revised as appropriate]

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 EN 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 EN 1997-3 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 EN 1997-3 through the following clauses:

<Drafting note> This is a preliminary list that will be updated for April draft.

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), Formual (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 EN 1997-3 on the application of the following informative annexes.

Annex A: Slopes, cutting and embankments

Annex B: Spread foundations

Annex C: Piled foundations

Annex D: Retaining structures

Annex E: Anchors

Annex F: Reinforced fill, soil nailed, and rock bolted structures

Annex G: Ground improvement

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 EN 1997-3

(1) EN 1997-3 provides specific rules to be applied for design and verification of certain types of geotechnical structures.

1.2 Assumptions

(1) EN 1997-3 is intended to be used in conjunction with EN 1997-1, which provides general rules for design and verification of all geotechnical structures.

(2) EN 1997-3 is intended to be used in conjunction with EN 1997-2, which provides requirements for assessment of ground properties from ground investigation.

2 Normative references

<Drafting note> The normative references will be further reviewed and updated for delivery for the April 2021 Final Draft.

American Wood Preservers Institute

ASTM D25-2:2017, *Standard Specification for Round Timber Piles*

BRE

BS 8417:2011+A1:2014, Preservation of wood. Code of practice.

EN 206:2013+A1:2016, *Concrete – Specification, performance, production and conformity*

EN 445:2007, *Grout for prestressing tendons. Test methods*

EN 447:2007, *Grout for prestressing tendons – Basic requirements*

EN 771-3, *Specification for masonry units. Aggregate concrete masonry units (Dense and light-weight aggregates)*

EN 1536:2010+A1:2015, *Execution of special geotechnical work. Bored piles*

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

EN 1538:2010+A1:2015, *Execution of special geotechnical works. Diaphragm walls*

EN 1563:2018, *Founding. Spheroidal graphite cast irons*

EN 1912:2012, *Structural Timber. Strength classes. Assignment of visual grades and species*

EN 1990-1, *Basis of structural and geotechnical design*

EN 1991-4

EN 1992-1-1, *Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings*

EN 1993-1-1, *Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*

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

EN 1994-1-1, *Eurocode 4. Design of composite steel and concrete structures. General rules and rules for buildings*

EN 1995-1-1, *Eurocode 5: Design of timber structures – Part 1-1: General Common rules and rules for buildings*

EN 1996-1-1, *Eurocode 6: Design of masonry structures – Part 1-1: General rules for reinforced and unreinforced masonry structures*

EN 1997-1, *Eurocode 7: Geotechnical design – Part 1: General rules*

EN 1997-2, *Eurocode 7: Geotechnical design – Part 2: Ground properties*

EN 1998-5, *Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures, and geotechnical aspects*

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

EN 10025-2, *Hot rolled products of structural steels. Technical delivery conditions for non-alloy structural steels*

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

EN 10080:2005, *Steel for the reinforcement of concrete. Weldable reinforcing steel. General*

EN 10138, *Prestressing steels*

EN 10149

EN 10210, *Hot finished structural hollow sections of non-alloy and fine grain steels*

EN10218-2, *Steel wire and wire products. General. Wire dimensions and tolerances*

EN 10219, *Cold formed welded structural hollow sections of non-alloy and fine grain steel*

EN 10223-3, *Steel wire and wire products for fencing and netting – Part 3: hexagonal steel wire mesh products for civil engineering purposes*

EN 10223-8, *Steel wire and wire products for fencing and netting. Welded mesh gabion products*

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

EN 10245-2 (all parts). *Steel wire and wire products. Organic coatings on steel wire*

EN 10248 (all parts), *Hot rolled sheet piling of non alloy steels*

EN 10249 (all parts), *Cold formed sheet piling of non alloy steels*

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

EN 10138

EN 12063:1999, *Execution of special geotechnical work. Sheet pile walls*

EN 12699:2015, *Execution of special geotechnical works. Displacement piles*

EN 12715, *Execution of special geotechnical works – Grouting*

EN 12716, *Execution of special geotechnical works – Jet grouting*

EN 12794:2005, *Precast concrete products. Foundation piles*

EN 13251, *Geotextiles and geotextile-related products. Characteristics required for use in earthworks, foundations and retaining structures*

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

EN 13670, *Execution of concrete structures*

EN 14081-1:2016+A1:2019, *Timber structures. Strength graded structural timber with rectangular cross section. General requirements*

EN 14199:2015, *Execution of special geotechnical works. Micropiles*

EN 14475, *Execution of special geotechnical works – Reinforced fill*

EN 14487-1, *Sprayed concrete. Definitions, specifications and conformity*

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

EN 14679, *Execution of special geotechnical works – Deep mixing*

EN 14731, *Execution of special geotechnical works – Ground treatment by deep vibration*

EN 15237, *Execution of special geotechnical works – Vertical drainage*

EN 15258, *Precast concrete products. Retaining wall elements*

EN 50162, *Protection against corrosion by stray current from direct current systems*

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

EN ISO 6892-1, *Metallic materials. Tensile testing. Method of test at room temperature*

EN ISO 10319, *Geosynthetics. Wide-width tensile test*

EN ISO 10321, *Geosynthetics – Tensile test for joints/seams by wide-width strip method*

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

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

EN 14688 (all parts), *Geotechnical investigation and testing - Identification and classification of soil*

EN ISO 14689:2018, *Geotechnical investigation and testing. Identification, description and classification of rock*

EN ISO 16907 (all parts), *Earthworks*

EN ISO 18674

EN ISO 17892-2, *Geotechnical investigation and testing – Laboratory testing of soil: Part 7: Unconfined compression test*

EN ISO 22282-4

EN ISO 22476-13 *Geotechnical investigation and testing – Field testing – Part 13: Plate load test (publication not confirmed)*

EN ISO 22477 (all parts), *Geotechnical investigation and testing – Testing of geotechnical structures*

3 Terms, definitions, and symbols

<Drafting note> The terms, definitions and symbols will be further reviewed, cross-checked with EN 1997-1/EN 1997-2 and updated for delivery for the April 2021 Final Draft.

3.1 Terms and definitions

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

3.1.1 Common terms used in EN 1997-3

3.1.1.1 substructure

part of a structure wholly or mainly below the level of the adjoining ground or a given level

[SOURCE: ISO 6707-1: 2017]

3.1.1.2 superstructure

part of a structure above the substructure

[SOURCE: ISO 6707-1: 2017]

3.1.1.3 foundation

construction for transmitting forces to the supporting ground

[SOURCE: ISO 6707-1:2017]

3.1.1.4 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

3.1.1.5 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 6707-1:2017]

3.1.1.6 low-rise structure

warehouse sheds, factory buildings, or residential buildings up to three storeys high

[Drafting note: text to be aligned with definition given in EN 1997-1 or EN 1990]

3.1.1.7 high-rise structure

buildings and structures greater than three storeys high, including chimneys and towers

[Drafting note: text to be aligned with definition given in EN 1997-1 or EN 1990]

3.1.1.8 frost heave

the swelling of soil due to formation of ice within it

[SOURCE: ISO 6707-1:2017]

3.1.1.9 ground heave

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

3.1.1.10 creep

increase in strain during sustained load

[SOURCE: ISO 6707-1:2017]

3.1.1.11 secondary consolidation

slow deformation of soil and rock mass because of prolonged pressure and stress; synonym for ‘creep’ in fine soils

3.1.1.12 competent rock

rock with sufficient strength and stiffness to withstand an applied static or dynamic load under given conditions 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 slope inclination

inclination of a ground or fill surface to the horizontal

3.1.2.3 cut

void that results from excavation of the ground

3.1.2.4 cutting

earth-structure created by excavation of the ground

3.1.2.5 cut slope

slope that results from excavation

3.1.2.6 embankment

earth-structure formed by the placement of fill

3.1.2.7 embankment slope

slope that results from the placement of fill

3.1.2.8 earthworks

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

3.1.2.9 excavation

result of removing material from the ground

3.1.2.10 levee

embankment for preventing flooding

3.1.2.11 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: 2017]

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: 2017]

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 6707-1:2017]

3.1.3.6 adjusted elasticity method

method to evaluate the settlement of a spread foundation using elasticity theory with a simple formula and 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 6707-1:2017]

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 6707-1:2017]

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 6707-1:2017]

3.1.4.4 driven pile

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

[SOURCE: modified from ISO 6707-1:2017]

3.1.4.5 end bearing pile

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

NOTE 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 6707-1:2017]

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 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 6707-1:2017]

3.1.4.7 tension pile

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

3.1.4.8 pile cap

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

[SOURCE: ISO 6707-1:2017]

3.1.4.9 piled foundation

foundation that incorporates one or more piles

[SOURCE: ISO 6707-1:2017]

3.1.4.10 pile group

foundation that incorporates piles arranged in a grid

3.1.4.11 piled raft

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

3.1.4.12 Ground Model Method

calculation method 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

3.1.4.13 Model Pile Method

calculation method 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

3.1.4.14 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

3.1.4.15 drag force

additional axial force acting on a pile due to downdrag

3.1.4.16 drag settlement

additional settlement of a pile due to downdrag

3.1.4.17 neutral plane

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

3.1.4.18 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

3.1.4.19 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

3.1.4.20 working pile

pile that will form part of the foundation of the structure

3.1.4.21 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

3.1.4.22 ultimate control test

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

3.1.4.23 serviceability control test

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

3.1.4.24 inspection test

test used to verify acceptance of a working pile

NOTE 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)

3.1.4.25 integrity test

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

3.1.4.26 pile load

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

3.1.4.27 pile test proof load

a maximum proposed test load required for a compression, tension, or transverse load test which takes into account the imposed action plus allowances for drag force (which may act in reverse under temporary loading conditions) or transverse ground load caused by moving ground, together with any temporary support resulting from particular conditions of the test such as variations in groundwater, pile head level or pile head restraint under service conditions

3.1.4.28 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

3.1.4.29 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]

3.1.4.30 critical creep load

critical creep load is the load beyond which the rate of axial pile displacement under constant load takes place with a notably increased increment

[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 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

force applied to the pile in a continuously increasing and then decreasing manner of a suitable duration (typically less than 1 s) relative to the natural period of the pile which causes the pile to compress over the full length and translate approximately as a unit during the full loading period

[SOURCE: EN ISO 22477-10:2016, 3.1.5]

3.1.4.35 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]

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

3.1.4.37 ultimate resistance of a pile

corresponding state in which the piled foundation displaces significantly with negligible increase of resistance

3.1.4.38 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]

3.1.4.39 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]

3.1.4.40 closed form solution

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

3.1.4.41 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]

3.1.4.42 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

3.1.4.43 pile set

permanent pile settlement after one hammer impact blow during driving

3.1.4.44 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. The weight of the wall itself, sometimes including stabilising 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. 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 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 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 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 soldier pile wall

embedded wall composed of primary elements, placed in the ground before excavation begins, and secondary elements, placed during successive steps of excavation works

NOTE to entry: Such retaining structures are also known as king post walls, or Berlin walls, more especially when primary elements are made of steel profiles.

3.1.5.6 combined wall

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

3.1.5.7 anchored wall

embedded wall supported by a tension element

3.1.5.8 waling

horizontal member of a tieback wall, which transmits the force from the tieback to the beams

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 to entry: A ‘grouted anchor’ in EN 1997-3 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

load test in which the load is applied in cycles, as specified in EN ISO 22477-5, Test Method 1

3.1.6.16 Test Method 3

load test in which the load is applied in steps, as specified in EN ISO 22477-5, Test Method 3

3.1.7 Terms relating to reinforced ground

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 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 (0.1hr, 1hr, 10hr etc.). 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 improvement

3.1.8.1 ground improvement

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

NOTE 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.

3.1.8.2 ground improvement zone

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

3.1.8.3 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

3.1.8.4 rigid inclusion

inclusions with higher stiffness and a measurable unconfined compressive strength

3.1.8.5 discrete ground improvement

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

3.1.8.6 diffused ground improvement

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

3.1.8.7 structural connection

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

3.1.8.8 contact

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

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

3.1.8.9 area ratio

ratio of the improved ground to the total area comprising improved and unimproved ground

NOTE to entry: The area ratio of diffused ground improvement is 1.

3.1.8.10 load distribution

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

NOTE 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

<Drafting note> The list of Symbols will be further updated for delivery for the April 2021 Final Draft.

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' x 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 ($A_r = A_0 - \Delta A_r$);
$A_{r,con}$	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 Bb (for square piles), Db (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) ;

D	bar diameter; and
D	embedment depth; and
D	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; and
D	diameter or width of the reinforcing element; and
D	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;
D_{add}	representative vertical or transverse temporary support force;
D_{b}	base diameter (for circular piles) 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_{rep}	representative drag force due to moving ground in pile foundations;
D_{supp}	representative vertical or transverse temporary support force;
E_{i}	initial tangent modulus in at-rest conditions;
E_{ur}	unloading-reloading modulus;
$F_{\mathrm{ad,SLS}}$	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;
$F_{\mathrm{ak,SLS}}$	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;
$F_{\mathrm{cd,SLS}}$	design axial compression applied to the pile at the serviceability limit state, including potential down drag forces;
$F_{\mathrm{d,group}}$	design action applied to the pile group or piled raft;
F_{td}	design axial tension applied to the pile;
$F_{\mathrm{tr,d}}$	design transverse force applied to the pile including an allowance for any potential transverse force due to moving ground;
K	earth pressure coefficient averaging the pressure around the whole circumference, $K = (1 + K_0)/2$;
K_0	at-rest earth pressure coefficient;
$K_{\mathrm{a}\gamma}$, K_{aq}, K_{ac}	normal active earth pressure coefficients;
$K_{\mathrm{ac,u}}$	normal active earth pressure coefficients for undrained conditions;
K_{M}	consequence factor applied to material properties;
$K_{\mathrm{p}\gamma}$, K_{pq}, K_{pc}	normal passive earth pressure coefficients;
$K_{\mathrm{pc,u}}$	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_{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_{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, M3 independent sets of material factors

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

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_c, N_q, N_γ bearing resistance factors;

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 of EN ISO 22477-5;

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;

$(R_{\mathrm{calc}})_{\mathrm{mean}}$	mean calculated pile resistance for a set of profiles of field test results;
$(R_{\mathrm{calc}})_{\mathrm{min}}$	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;
$R_{\mathrm{d,group}}$	design resistance of the pile group or piled raft;
$R_{\mathrm{d,gs,int}}$	design tensile strength of the interface with the geosynthetic reinforcing element;
$R_{\mathrm{d,st,int}}$	design tensile strength of the interface with a steel reinforcing element;
$R_{\mathrm{d,sn,int}}$	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);
$R_{\mathrm{m,sn,pul}}$	measured pull-out force;
R_{pd}	design value of the resisting force caused by earth pressure on the side of a foundation ;
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;
$R_{\mathrm{ri,i}}$	resistance of a rigid inclusion i, depending on its position within the group;
$R_{s,rep}$	pile's representative shaft resistance (in axial compression);
$R_{\mathrm{sys,rep}}$	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;
$(R_{\mathrm{test}})_{\mathrm{mean}}$	mean calculated pile resistance measured in a set of load tests;
$(R_{\mathrm{test}})_{\mathrm{min}}$	minimum calculated pile resistance measured in a set of load tests;
$R_{\mathrm{tr,d}}$	pile's design transverse resistance;
$R_{\mathrm{t,rep}}$	pile's representative axial tensile resistance;
$R_{\mathrm{t,rep,el}}$	representative tensile resistance of the reinforcing element;
$R_{\mathrm{x,d}}$	design resistance of a pile (where x = b, c, s, st, t, or tr, as above) ;
$R_{\mathrm{x,m}}$	measured resistance of a pile (where x = b, c, s, st, t, or tr, as above) ;
$R_{\mathrm{x,rep}}$	representative resistance of a pile (where x = b, c, s, st, t, or tr, as above) ;
S_{t}	sensitivity of fine soil;
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;

$T_{gs,k}$	characteristic tensile strength of geosynthetic reinforcement;
T_k	characteristic tensile strength of the reinforcing element;
$T_{k,cr}$	characteristic tensile strength of the reinforcing element allowing for creep and limiting elongation;
T_{rep}	representative value of T;
V_{norm}	coefficient of variation based on a normal distribution of strength values.

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_{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;
$c_{u,rep}$	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 ground investigation below the base of the foundation;
d_s	rock discontinuity spacing between a pair of immediately adjacent discontinuities;
e	eccentricity of the resultant action, with subscripts B and L;
e_d	design eccentricity of the resultant action, with subscripts B and 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_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 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;
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_{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);
q	overburden or surcharge pressure at the level of the foundation base; and
q	vertical surcharge applied at the surface of the ground
q'	effective overburden pressure at the level of the foundation base;
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);

$q_{s,i}$	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_{ground}	ground strata settlement profile (at any particular time);
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;
x	distance along the length of the reinforcing element;
z_a	depth of zone of influence; and
z_a	depth at the active side of the retaining wall;
z_p	depth at the passive side of the retaining wall;
z_{zoi}	depth of zone of influence.

3.2.3 Greek upper-case letters

Δa	deviation related to geometrical properties and change made to nominal geometrical properties for particular design purposes, e.g. assessment of effects of geometrical imperfections, sensitivity and uncertainty; ;
ΔA_r	maximum anticipated loss of steel area during the design service life of the structure;
Δc_{dev}	allowance in design for deviation of the concrete cover;
Δe	loss of steel thickness at one face at the considered point in time along the design service life (of the reinforcement element or structure).

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_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 EN 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_{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;

γ_{Re}	passive earth resistance factor (on retaining walls);
$\gamma_{Rd,group}$	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_{rep}	representative value of δ;
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 between the horizontal load, H and the direction L';
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);
φ_0	original diameter of the reinforcement element.
$\varphi_{cv,k}$	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;

3.2.5 Abbreviations

DC	Design Case;
CPT	Cone Penetration Test;
GC	Geotechnical Category;

GCC	Geotechnical Complexity Class;
MFA	Material Factor Approach;
NDP	National Determined Parameter
PMT	Pressure meter Test;
PWM	Polymer Steel Woven Wire Mesh
RFA	Resistance Factor Approach;
SPT	Standard Penetration Test;

4 Slopes, cuttings, and embankments

4.1 Scope and field of application

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

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

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

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

(1) <REQ> This clause shall also apply to dams and levees but excludes the verification of water retention of those structures.

NOTE 3. 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.

4.2 Basis of design

4.2.1 Design situations

(1) <REQ> EN 1997-1, 4.2.2 shall apply.

4.2.2 Geometrical properties

4.2.2.1 General

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

4.2.2.2 Zone of influence

(1) <REQ> EN 1997-1 4.2.1.1 shall apply.

4.2.3 Actions and environmental influences

4.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

4.2.3.2 Permanent and variable actions

(1) <RCM> Long-term settlement and movement in serviceability limit states should be verified using the quasi-permanent combination of actions specified in EN 1990, 8.4.3.4.

(2) <REQ> Redistribution of the initial in-situ stress due to excavation shall be considered.

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

NOTE 4. Guidance on traffic loads is given in EN 1997-1, Annex F.

4.2.3.3 Cyclic and dynamic actions

(1) <REQ> EN 1997-1, 4.3.1.3 shall apply.

4.2.3.4 Environmental influences

(1) <REQ> EN 1997-1, 4.3.1.4 shall apply.

(2) <REQ> Actions due to temperature effects that act within the zone of influence shall be included into the design verification.

(3) <RCM> The influence of shedding due to diurnal and nocturnal cycles should be included into the verification of slope and cutting stability.

(4) <RCM> Freezing and freeze-thaw cycles should be included into the verification of slope and cutting stability.

(5) <RCM> Freezing and thawing forces of groundwater in rock discontinuities should be considered as actions.

4.2.4 Limit states

4.2.4.1 Ultimate Limit States

(1) <REQ> In addition to EN 1997-1, 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;
- 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 of the failure of drains, filters or seals;
- failure in ground caused by surface or internal erosion, or scour;
- excessive movements in the ground; and
- structural failure in structures, roads, railway lines, or utilities due to movements in the ground in the zone of influence.

(2) <RCM> Ultimate limit states other than those given in (1) should be verified as necessary.

4.2.4.2 Serviceability Limit States

(1) <REQ> In addition to EN 1997-1, 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 to changes made to the groundwater conditions;
- deformation of the structure, which may cause serviceability limit states of existing nearby structures ;

- excessive movements in the ground due to shear deformations, settlement, vibration or heave; and
- accumulated ground movement or settlement due to creep.

(2) <RCM> Serviceability limit states other than those given in (3) should be verified as necessary.

4.2.5 Robustness

(1) <REQ> EN 1997-1, 4.1.4 shall apply.

4.2.6 Ground investigation

4.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply.

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

(2) <RCM> For the verification of excavation works, the ground investigation should focus on the properties of the ground to be excavated.

4.2.6.2 Minimum extent of in-situ testing

(1) <REQ>The depth and horizontal extent of the in-situ testing shall be sufficient to determine the ground conditions within the zone of influence.

(2) <RCM> For slopes and cuttings in Geotechnical Category 1, the minimum depth of investigation below the planned excavation level should be $d_{\min}$ = 2 m.

(3) <RCM> For slopes and cuttings in Geotechnical Category 2 in rock masses with a high load – rock strength ratio, the minimum depth of investigation below the planned excavation level into the rock mass should be $d_{\min}$ = 2 m.

(4) <RCM> For slopes and cuttings in Geotechnical Category 2, the minimum depth of investigation $d_{\min}$, below the planned excavation level in soils or rock masses with a low load – rock strength ratio, should comply with Formula (4.1), unless a stratum of high strength is identified:

$$d_{\min} \geq \max(0.4\, h_{\mathrm{C}}; 3m) \tag{4.1}$$

where:

h_{C} is the maximum height of the cutting or slope from the highest ground level (Figure 4.1)

(3)

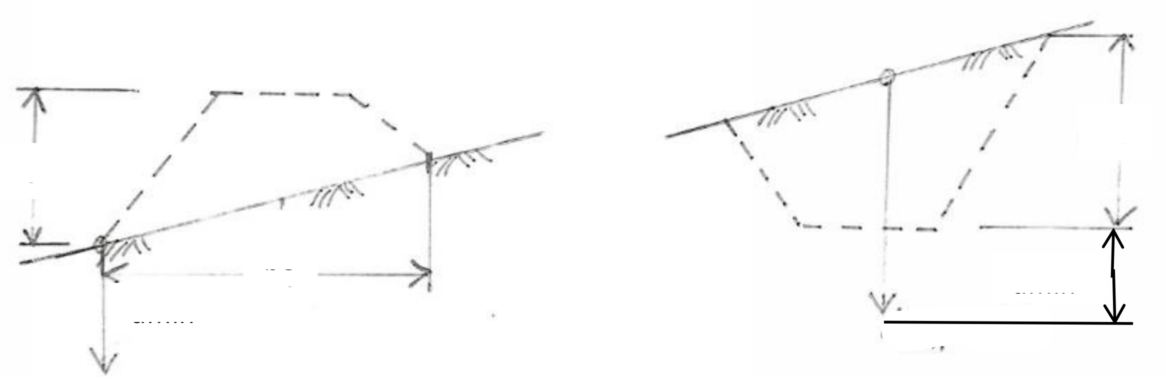
(4) 

Figure 4.1 – Minimum depth of investigation for a) embankments and b) cuttings

NOTE 6. Excavation below groundwater level can cause hydraulic heave, groundwater flow, internal erosion, piping, surface erosion or severe reduction in ground strength.

(5) <RCM> For embankments in Geotechnical Category 1, the minimum depth of investigation, below the planned base of the embankment, should be $d_{\min}$ = 2 m.

(6) <RCM> For embankments in Geotechnical Category 2 on top of rock masses with a high load – rock strength ratio, the minimum depth of investigation, below the planned base of the embankment into the rock mass, should be $d_{\min}$ = 3 m.

(7) <RCM> For embankments in Geotechnical Category 2, the minimum depth of investigation $d_{\min}$, below the planned base of the embankments, on top of soils or rock masses with a low load – rock strength ratio, should comply with Formula (4.2), unless a stratum of high strength is identified:

$$d_{\min} \geq \max(1.2h_{\mathrm{E}}; 1.0b_{\mathrm{E}}; 3m) \tag{4.2}$$

where:

b_{E} is the width of the embankment at original ground level (see Figure 4.1)

h_{E} is the maximum height of the embankment from the lowest ground level

(8) <RCM> In addition to (9), $d_{\min}$ should go, at least, down to the bottom of the deepest fine soil layer (or layer of high compressibility) that could undergo consolidation settlement.

(9) <REQ> Groundwater and piezometric levels shall be determined if they could influence the stability or settlement of the slopes, cuttings, embankments or any surrounding structures or utilities.

4.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

4.3 Materials

4.3.1 Ground properties

(1) <REQ> EN 1997-2, 7-12 shall apply.

NOTE 7. A classification of fill is given in EN 16907-2.

(2) <RCM> Anisotropy should be considered when determining ground properties that can potentially influence ground behaviour.

(3) Anisotropic ground strength is of special importance for cuttings in fine soils due to the unloading and rotation of the principal stresses.

(4) <RCM> Potential reduction in strength caused by exposure to weather conditions during or after execution (in particular, saturation of the ground and thawing of frozen ground) should be considered.

(5) <PER> In addition to EN 1997-1, 5.2.2(2), drained or undrained soil or fill parameters (or a combination of both) may be used in the design of slopes, cuttings, and embankments, depending on the soil hydraulic conductivity and the duration of any loading or unloading.

(6) <REQ> Values of discontinuities shall comply with EN 1997-2, 6.2.

4.3.2 Properties of improved ground

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

4.4 Groundwater

4.4.1 General

(1) <REQ> EN 1997-1, 6 shall apply.

(2) <REQ> Measures shall be provided to prevent the adverse effects of potential scour leading to erosion of soil under and around an earth-structure.

(3) <REQ> Water pressure at interfaces and between discontinuities shall be determined.

(4) <REQ> Water flow through interfaces and discontinuities shall be determined

(5) <REQ> If a drainage system in not provided, then the design shall be verified to withstand potential building-up of groundwater pressures.

4.4.2 Drainage systems

(1) <REQ> Clause 12 shall apply.

(2) <PER> Drainage systems may be provided to ensure the design groundwater and piezometric levels are not exceeded due to unforeseen circumstances.

NOTE 8. Examples of drainage for cuttings and embankments are given in EN 16907-1.

(3) <RCM> Where the safety and serviceability of the earth-structure depend on the successful performance of a drainage 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;
- installing a drainage system that will perform adequately without maintenance; and
- installing a secondary (“backup”) system.

4.5 Geotechnical analysis

4.5.1 General

(1) <REQ> EN 1997-1, 7 shall apply.

(2) <RCM> 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 and associated effects such as liquefaction; and
– amplification of loads or displacements owing to resonance.

4.5.2 Overall and local stability

4.5.2.1 Stability of slopes and cuttings in soils and fills

(1) <REQ> When verifying the overall stability, all potential failure mechanisms shall be verified.

(2) <RCM> At least one of the following calculation models for slope stability should be used:

– limit-equilibrium methods;
– numerical models according to EN 1997-1, 8;
– limit analysis; and
– methods for rock slopes.

NOTE 9. Calculation models for overall stability of soil and fill slopes are given in Annex A.3.

NOTE 10. Calculation models for stability of rock slopes are given in Annex A.4.

(3) <REQ> For sloping ground in layered soils with considerable differences in shear strength or subjected to high external loads, the stability of both circular and non-circular failure surfaces shall be verified, paying attention to the layers with lowest shear strength.

(4) <RCM> 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.

(5) <RCM> The weight density of a geotechnical unit should be a superior (upper) or inferior (lower) value, depending on whether the weight density has a favourable or unfavourable influence on the stability of the slope.

(6) <PER> The stabilizing effect from capillary action may be used if its effect can be verified by comparable experience, groundwater pressure measurements or by long-term site observations under the different weather conditions that can occur during the service life of the earth-structure.

NOTE 11. 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.

4.5.2.2 Stability of slopes and cuttings in rock mass

(1) <REQ> The rock excavation technique and sequence shall be integrated into the verification of the slopes or cuttings in rock.

NOTE 12. Calculation models for stability of rock slopes are given in Annex A.4.

(2) <REQ> The damaging effects of excavation by blasting on rock fracturing shall be acknowledged for in the design and verification of the slope or cutting.

(3) <REQ> The influence of rock wedges within slopes and cuttings on the local stability shall be considered.

(4) <REQ> The effect of possible local instability on the overall stability shall be considered.

(5) <REQ> Scaling of rock surfaces shall be implemented into the design as a key safety measure.

4.5.2.3 Local stability of rock

(1) <REQ> The influence of rock wedges within slopes and cuttings on the local stability shall be considered.

(2) <REQ> Local instability of sloping ground within the zone of influence of structures and foundations shall be considered.

(3) <REQ> The effect of possible local instability on the overall stability shall be considered.

4.5.3 Analysis of embankments

(5) <RCM> Analysis of embankments containing different materials should adopt strength and stiffness properties that have been determined at compatible strains in the different materials.

(6) <REQ> Where lightweight fill materials with a weight density less than that of water are used, the possibility of uplift due to buoyancy shall be considered.

(7) <PER> 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.

4.5.4 Supporting elements

(1) <REQ> 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 the stiffness of the ground and that of the supporting element.

NOTE 13. Cases include failure surfaces intersecting supporting elements such as walls, piles, anchors, discrete ground improvement, and reinforcement elements and walls.

(2) <REQ> If supporting elements are used in the design to increase the overall stability, the structural reliability shall be verified for the combined effects of action from the ground and the super-structure for all relevant design situations.

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

(4) It shall be verified that the supporting element can resist a design force effect given by Formula (4.3):

(8) $E_d = \max(\gamma_{S,d}\, F_{d,ULS};\ \gamma_F\, F_{d,SLS})$ (4.3)

(9) where

(10) $F_{ad,ULS}$ is the design value of the force that the supporting element shall provide to prevent an ultimate limit state of the slope, cutting or embankment.

(11) $F_{ad,SLS}$ is the design value of the force that the supporting element shall provide to prevent a serviceability limit state of the slope, cutting or embankment.

(12) $\gamma_{S,d}$ is a model factor to take into account the concentration of load in the supporting element.

(13) γ_F is a partial factor to convert s SLS value into an ULS value (using DC4)

NOTE 14. The value of the model factor $\gamma_{S,d}$ is 1.0 unless the National Annex gives another value.

NOTE 15. The value of the partial factor γ_F is 1.35, according to DC4, unless the National Annex gives another value.

4.5.5 Ground displacement and settlement of embankments

(1) <RCM> 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 of soil due to change in water content; and
- swelling of ground due to change in water content;

(2) presence of cavities in the ground<RCM> 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 16. Consolidation and creep can occur simultaneously, particularly in thick soil layers of low hydraulic conductivity.

(3) <RCM> 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) <RCM> 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

4.6.1.1 Overall and local stability

(1) <REQ> The overall stability of the following geotechnical structures shall be verified:

– ground retaining structures;
– cuttings and embankments, including reinforced and improved soil structures;
– structures, infrastructure and foundations on or near sloping ground; and
– existing slopes within the zone of influence of planned construction works.

(2) <REQ> Verification of the overall and local stability of anchors shall comply with Clause 8.

(3) <RCM> 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 17. A calculation model for extrusion resistance of reinforced embankment bases is given in Annex F.4

NOTE 18. Calculation models for embankments subject to punching shear are given in Annex B.5.

(4) <PER> The slope, cutting or embankment may need additional reinforcement elements to achieve the overall stability.

(5) <REQ> For design verification of the additional reinforcement elements, Clause 10 shall apply.

4.6.1.2 Stability of reinforced structures

(1) <REQ> Verification of limit states for reinforced ground structures shall comply with Clauses 9 and 10.

(2) <REQ> In addition to (1), verification of the bearing resistance of the ground beneath reinforced embankments shall comply with Clause 5.

4.6.1.3 Stability of improved ground

(1) <REQ> Verification of limit states for geotechnical structures in or on improved ground shall comply with Clause 11.

4.6.2 Verification by prescriptive rules

(1) <REQ> EN 1997-1, 4.5 shall apply.

4.6.3 Verification by testing

(1) <REQ> EN 1997-1, 4.6 shall apply.

(2) <PER> Staged construction, trial embankments and trial excavations or cuttings may be used to verify limit states.

4.6.4 Verification by the Observational Method

(1) <RCM> In addition to EN 1997-1, 4.7, the Observational Method should only be used for soil and fill if there is no possibility of sudden, brittle failure and only if it is possible to monitor the earth-structure and its zone of influence.

4.6.5 Partial factors

(14) <REQ> EN 1997-1, 4.4.1(4) shall apply:

NOTE 19. 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.

(15) <RCM> The design value of the angle of internal friction of the ground φ_d should comply with EN 1997-1, 4.4.3(5).

(16) <RCM> The design value of the angle of internal friction of soil at critical state ($\varphi_{cs,d}$) should be determined from Formula (4.4.):

$$\tan \varphi_{cs,d} = \frac{\tan \varphi_{cs,rep}}{\gamma_{\tan\varphi,cs}} \tag{4.4}$$

where:

$\varphi_{cs,rep}$ is the representative value of the angle of internal friction of soil at critical state;

$\gamma_{\tan\varphi,cv}$ is a partial material factor applied to the coefficient of internal friction of soil at critical state.

(17) <RCM> The design value of the effective cohesion of soil at critical state (c′d,cs) should be taken as zero.

(18) <RCM> The design value of the angle of friction along an existing slip surface in fine soil of high plasticity should be determined from the soil's residual angle of friction.

(19) <RCM> The design value of the angle of friction of soil along a residual slip surface (□res,d) should be determined from Formula (4.5):

$$\tan \varphi_{res,d} = \frac{\tan \varphi_{res,rep}}{\gamma_{\tan\varphi,res}} \tag{4.5}$$

where:

$\varphi_{res,rep}$ is the representative value of the angle of friction of soil along a residual slip surface;

$\gamma_{\tan\varphi,res}$ is a partial material factor applied to the coefficient of internal friction of soil along a residual slip surface.

(20) <RCM> The design value of the effective cohesion of soil along a residual slip surface ($c'_{d,res}$) should be taken as zero.

(21) <PER> Provided the conditions specified in EN 1997-1 4.4.3(10) are satisfied, the value of γ_M for transient design situations may be multiplied by a factor $K_{M,tr} \leq 1{,}0$ provided that the product $K_{M,tr} \cdot \gamma_M$ is not in itself less than 1,0.

NOTE 20. The value of KM,tr is 1.0 unless the National Annex gives a different value.

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)[1, 2]**
Overall stability	Actions and effects-of-actions	γ_F and γ_E	DC3
	Ground properties[3]	γ_M	M2[2]
Bearing resistance	see Clause 5		

[1]Values of the partial factors for Design Cases 3, (DC3) are given in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1
[3]Also includes ground properties of Class IA 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)[1]
Overall stability	Actions and effects-of-actions	γ_F and γ_E	Not factored
	Ground properties[2]	γ_M	M2
Bearing resistance	see Clause 5		

[1]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1 Annex A.
[2]Also includes ground properties of Class IA ground improvement (Clause 11).

4.7 Serviceability limit states

4.7.1 General

(1) <REQ> The verification of serviceability limit states for slopes, cuttings, and embankments shall comply with EN 1997, 9.

(2) <REQ> 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 infrastructure.

(3) <REQ> Serviceability limit states of cuttings, slopes and embankments shall be verified by calculation and monitoring.

4.7.2 Displacement of slopes and cuttings

(1) <PER> In accordance with EN 1990, 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) <REQ> 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) <RCM> When verifying the settlement of an embankment, any decrease in effective stress in the ground due to submergence should be considered.

4.7.4 Structural serviceability

(1) <REQ> Selection of limiting values of ground movement that affects the serviceability of a structure shall comply with EN 1997-1, 9.3.

4.8 Implementation of the design during execution and service life

4.8.1 General

(1) <REQ> EN 1997-1, 10 shall apply.

(2) <RCM> The execution of earthworks should comply with EN 16907-3.

4.8.2 Inspection

(1) <REQ> EN 1997-1, 10.3 shall apply.

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

4.8.3 Monitoring

4.8.3.1 General

(1) <REQ> EN 1997-1, 10.4 shall apply.

(2) <RCM> In addition to (1), a Monitoring Plan should be prepared for slopes, cuttings, and embankments in Geotechnical Category 2 for the following situations:

- when testing is used to verify limit states;
- 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 control adverse effects on structures and utilities.

4.8.3.2 Monitoring of slopes and cuttings

(1) <RCM> The Monitoring Plan for slopes and cuttings should include measurement of the following, as appropriate:

- 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 (e.g. cracking) of structures and infrastructures within the zone of influence.

4.8.3.3 Monitoring of embankments

(1) <RCM> The Monitoring Plan for an embankment should include measurement of the following, as appropriate:

– 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; and
– checks on hydraulic conductivity or grain sized distribution of fill material and of foundation soil during construction.

(2) <RCM> When an embankment on fine soil of low strength is raised in layers, 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) <REQ> EN 1997-1, 10.5 shall apply.

(2) <RCM> The Maintenance Plan should include the following, as appropriate:

– 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; and
– allowable loads and other restrictions during maintenance work.

4.9 Testing

(1) <REQ> EN 1997-1, 11 shall apply.

(2) <RCM> Testing for quality control of earthworks should comply with EN 16907-5.

4.10 Reporting

(1) <REQ> EN 1997-1, 12 shall comply.

5 Spread foundations

5.1 Scope and field of application

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

(2) <PER> 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) <REQ> EN 1997-1, 4.2.2, shall apply.

(2) <RCM>In addition to (1) design situations for spread foundations should include but not be limited to:

- the effects of soluble, expansive, and collapsible soils;
- the effects of the particular features of rock; and
- the effects of scour.

5.2.2 Geometrical properties

5.2.2.1 General

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

(2) <RCM> The width of a spread foundation should be chosen taking into account setting out tolerances, working space requirements, and the dimensions of the structural member supported by the foundation.

(3) <REQ> 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 taken into account.

NOTE 21. Influences that can affect the resistance of the bearing stratum are given in Annex B.3.

5.2.2.2 Zone of influence

(1) <REQ> EN 1997-1, 4.1.2.1 shall apply.

5.2.3 Actions and environmental influences

5.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

5.2.3.2 Permanent and variable actions

(1) Actions in design situations involving a spread foundation shall include, but not limited to:

- imposed actions from the super-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 of active clays;
- 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; and
- accidental actions.

(2) <RCM> The adverse effects of actions on a spread foundation due to planned construction of neighbouring structures and nearby excavations should be taken into account.

(3) <REQ> Hazards due to ground properties shall be explicitly identified.

NOTE 22. Note: Examples of risks are active soils, swelling, shrinking and heave.

NOTE 23. Conditions of use for active clays are given in EN 16907-3: Annex B10.

(4) <REQ> Measures shall be taken to avoid the swelling of active soils during execution of a spread foundation.

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

NOTE 24. For example, due to the presence or removal of nearby trees or other vegetation.

(6) <REQ> Water pressures not caused by the foundation load shall be included as actions.

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

(8) <RCM> For extended spread foundations, including slabs, an analysis of the interaction between the supported structure and the ground should be performed in order to determine the distribution of actions on the spread foundation.

5.2.3.3 Cyclic and dynamic actions

(1) <REQ> EN 1997-1, 4.3.1.3 shall apply.

(2) <RCM> The design of foundations subject to cyclic and dynamic loading should take into account 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;
- increase in horizontal actions leading to larger eccentricity and smaller effective foundation area and hence 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.

5.2.3.4 Environmental influences

(1) <REQ> EN 1997-1, 4.3.1.4 shall apply.

(2) <REQ> Measures shall be taken to avoid frost impact on ground during execution in frost susceptible ground.

(3) <REQ> Testing to determine the frost susceptibility of ground shall comply with EN 1997-2, 12.1.

(4) <PER> 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; and
- providing insulation to prevent frost occurring in accordance with EN ISO 13793.

(5) <REQ> The potential of ground freezing due to low temperatures passing through foundation elements causing deformations of the structure shall be taken into account in the case of frost susceptible ground.

NOTE 25. This particularly applies to thin raft foundations, including during execution.

(6) <RCM> The adverse effects of frost action caused by construction work or by ground freezing should be taken into account.

(7) <REQ> 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.

5.2.4 Limit states

5.2.4.1 Ultimate limit states

(1) <REQ> In addition to EN 1997-1, 8.1, the following ultimate limit states shall be verified for all spread foundations:

- bearing failure;
- sliding, slipping and toppling failure;
- shear and tensional 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) <RCM> Ultimate limit states other than those given in (1) should be verified as necessary.

(3) <REQ> For shear or tensional failure of possible ground-foundation reinforcement elements, Clause 10 shall apply.

5.2.4.2 Serviceability limit states

(1) <REQ> In addition to EN 1997-1, 9, the following serviceability limit states shall be verified for all spread foundations:

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

(2) <RCM> Serviceability limit states other than those given in (3) should be verified as necessary.

5.2.5 Robustness

(1) <REQ> EN 1997-1, 4.1.4 shall apply.

5.2.6 Ground investigation

5.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply.

5.2.6.2 Minimum extent of in-situ testing

(1) <REQ> The depth and horizontal extent of the ground investigation shall be sufficient to determine the ground conditions within the zone of influence.

(2) <RCM> 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) <RCM> 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):

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

where:

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

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

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

where:

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

(5) <RCM> 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.

(6) <PER> The minimum depth of investigation may be reduced within medium strong (or stronger) rock masses provided there is comparable experience to allow the properties of the rock mass to be predicted.

(7) <RCM> Greater investigation depths should be selected when:

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

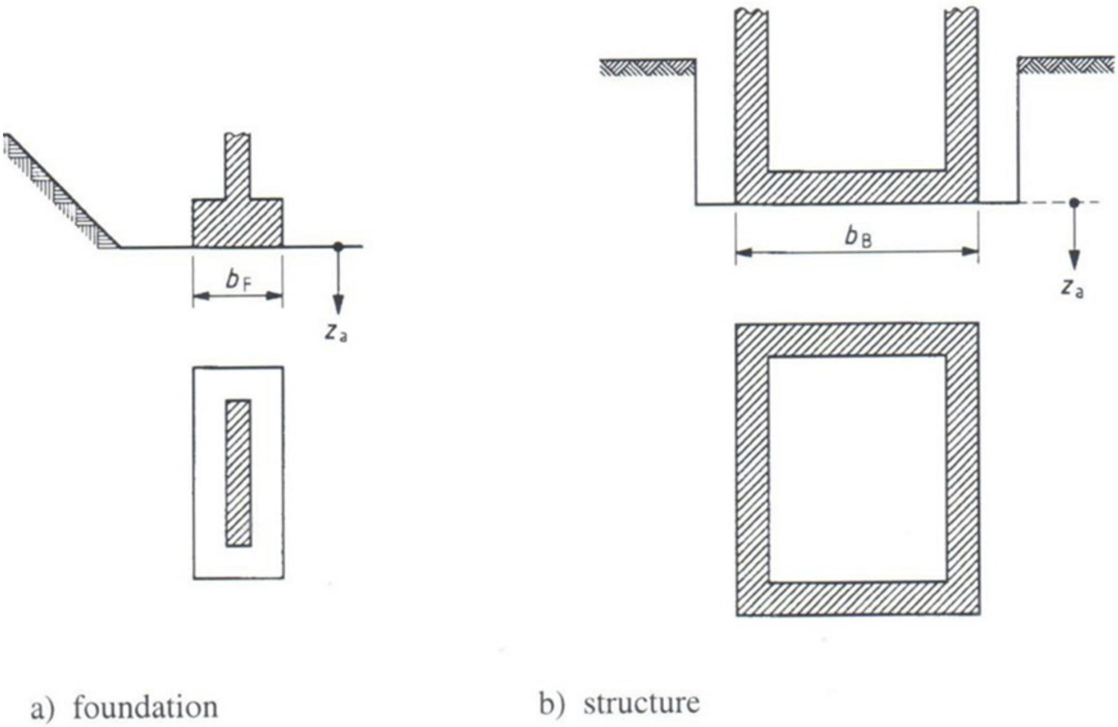


Figure 5.1 – Definition of d_{min} for spread foundations

<Drafting NOTE: figure to be redrawn, z_a to be replaced by d_{min}>

5.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

5.3 Materials

5.3.1 Ground properties

(1) <REQ> EN 1997-2, 7-12 shall apply.

(2) <REQ> For engineered fills, EN 1997-1, 5.2 shall apply.

(3) PER> In addition to EN 1997-1, 5.2.2(2), drained or undrained soil parameters may be used in the design of spread foundations, 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) <REQ> EN 1997-1, 5.5 shall apply.

5.4 Groundwater

5.4.1 General

(1) <REQ> EN 1997-1, 6 shall apply

(2) <REQ>The groundwater level and groundwater pressures and all potential changes in these 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) <PER> 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) REQ> Surface water, groundwater and piezometric levels shall comply with EN 1997-1, 6.2, and EN 1997-2, 11.

(5) <RCM> Where the groundwater level is close to the foundation level, the effects of capillary rise causing deterioration of foundation materials should be taken into account.

NOTE 26. Capillary rise can be avoided by including waterproofing membranes or a capillary break soil layer.

5.4.2 Drainage systems

(1) <REQ> Clause 12 shall apply.

(2) <RCM> If ponding of water above a spread foundation reduces its reliability 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) <RCM> Where the safety and serviceability of a spread foundation depend on the successful performance of a drainage system, one or more of the following measures should be taken:

- a Maintenance Plan should be specified;
- a drainage system should be specified that will perform adequately without maintenance; and
- a secondary (“backup”) system should be specified that will prevent any potential leakage from entering the ground beneath or next to the structure.

NOTE 27. 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) <REQ> EN 1997-1, 7 shall apply.

(2) <REQ> If a calculation is used to verify limit states for a spread foundation, separate analyses shall be carried out for each limit state.

(3) <PER> Separate analyses for each limit state may be omitted when using numerical models that verify ultimate and serviceability limit states in one analysis.

(4) <REQ> When checking ultimate limit states, the calculation model shall represent as closely as possible the anticipated failure mechanism.

(5) <RCM> When checking a spread foundation for ultimate or serviceability limit states, the interaction effect of adjacent foundations on the loading, resistance and movement of the foundation should be taken into account as well as the effect of the spread foundation on nearby foundations, structures, and services.

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

NOTE 28. Guidance is given in Annexes B.4 to B.12.

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

(8) <RCM> 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) <PER> The undrained bearing resistance (R_{Nu}) to a force acting normal to the base of a spread foundation on soil or fill may be determined from Formula (5.3):

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

where:

A'	is the effective plan area of the foundation;
B'	is the effective foundation width shown in Figure 5.2;
N_{cu}	is a non-dimensional bearing resistance factor for undrained conditions;
$N_{\gamma u}$	is a non-dimensional bearing resistance factor for the influence of the ground's weight density ($N_{\gamma u}$ is zero for undrained conditions except when the ground surface slopes downwards away from the foundation, in which case it is negative);

c_u is the soil undrained shear strength;

q is the overburden pressure applied to the ground outside the foundation;

γ is the weight density of the ground below the base of 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 29. Formulae 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 30. 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 $N_{\gamma u}$ a non-dimensional bearing resistance factor for the influence of the ground's weight density with negative value in this case.

(2) <RCM> Formula (5.3) should only be used if the undrained shear strength is assumed constant within the zone of influence for bearing resistance.

NOTE 31. In strongly overconsolidated soils, the undrained shear strength is usually constant.

(3) <RCM> 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:

$$A' = B' \times L' = (B - 2e_B)(L - 2e_L) \tag{5.4}$$

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 32. The dimensions are illustrated in Figure 5.3.

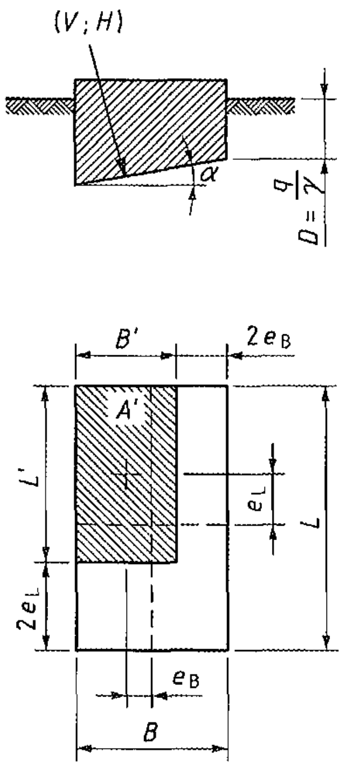


Figure 5.2 – Notation for a rectangular spread foundation with an inclined base and eccentric load

<Drafting NOTE: figure to be redrawn, V to be replaced by N, H to be replaced by T; ground to be shown sloping down at angle ω to the horizontal on one side>

(4) <RCM> 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):

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

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

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

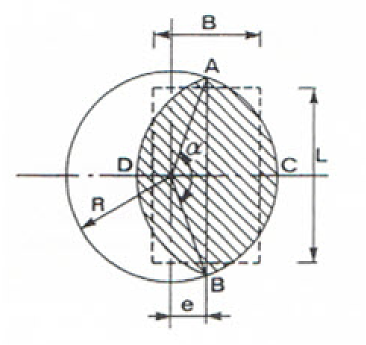


Figure 5.3 – Notation for a circular spread foundation with an inclined base and eccentric load

<Drafting NOTE: figure to be redrawn, B to be replaced by B_{eq}; L to be replaced by L_{eq}

(5) <PER> The drained bearing resistance (R_{N}) to a force acting normal to the base of a spread foundation on soil or fill may be determined from Formula (5.7):

$$R_{\mathrm{N}} = A'\left(c'N_{\mathrm{c}}b_{\mathrm{c}}d_{\mathrm{c}}g_{\mathrm{c}}i_{\mathrm{c}}s_{\mathrm{c}} + q'N_{\mathrm{q}}b_{\mathrm{q}}d_{\mathrm{q}}g_{\mathrm{q}}i_{\mathrm{q}}s_{\mathrm{q}} + 0.5\gamma'B'N_{\gamma}b_{\gamma}d_{\gamma}g_{\gamma}i_{\gamma}s_{\gamma}\right) \tag{5.7}$$

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 effective 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 34. Formulae for N_{c}, N_{q}, etc. are provided in Annex B.4(4) and (6).

NOTE 35. Guidance is given in Annex B.4(7) to account for the effect of groundwater level on groundwater pressure and effective weight density.

(6) <RCM> 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.

(7) <PER> 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.

(8) <REQ> The q term in Formulae (5.3) and (5.7) shall be reduced in case of potential removal of overburden during the design service life of the foundation.

(9) <RCM> 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.

(10)<REQ> 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 take into account the characteristics of the layering and discontinuities.

(11)<RCM> Where a weaker geotechnical unit underlies a stronger unit, including a granular layer forming a working platform foundation, the rupture mechanisms that should be taken into account 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 36. Calculation models for punching failure of a spread foundation on a stronger geotechnical unit over a weaker unit are given in Annex B.5.

(12)<PER> 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.

NOTE 37. Guidance on the design of working platforms is given in TWf (2019).

(13)<REQ> 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.

NOTE 38. Guidance on the design of a spread foundation on reinforced ground close to a slope is given in EBGEO and CIRIA SP123 (1996).

(14)<RCM> When analytical models cannot accommodate or do not adequately represent the design situations described in (16) and (17), numerical models should be used instead to determine the most unfavourable failure mechanism (see EN 1997-1, 8.2).

5.5.2.2 Bearing resistance and settlement from empirical models

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

(2) <PER> The bearing resistance and settlement of a spread foundation on soil may be determined from the results of pressuremeter tests and calculation models.

NOTE 39. Empirical calculation models for the bearing resistance and settlement of a spread foundation based on the results of Ménard Pressuremeter Tests are given in Annexes B.6 and B12.

5.5.2.3 Bearing resistance of rocks

(1) <PER> The bearing resistance of spread foundations of rock may be determined using the following models presented in Figures 5.4and 5.5.

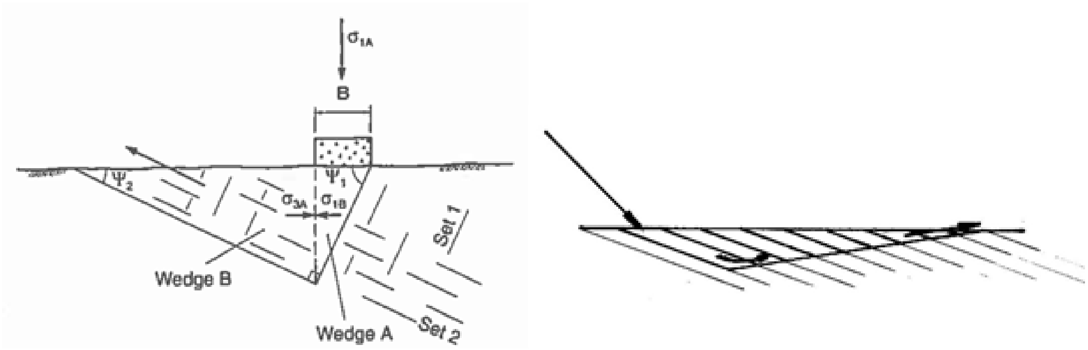


Figure 5.4 – Bearing capacity of a foundation on rock containing inclined discontinuities or weakness zones, either as wedge failure or slumping failure

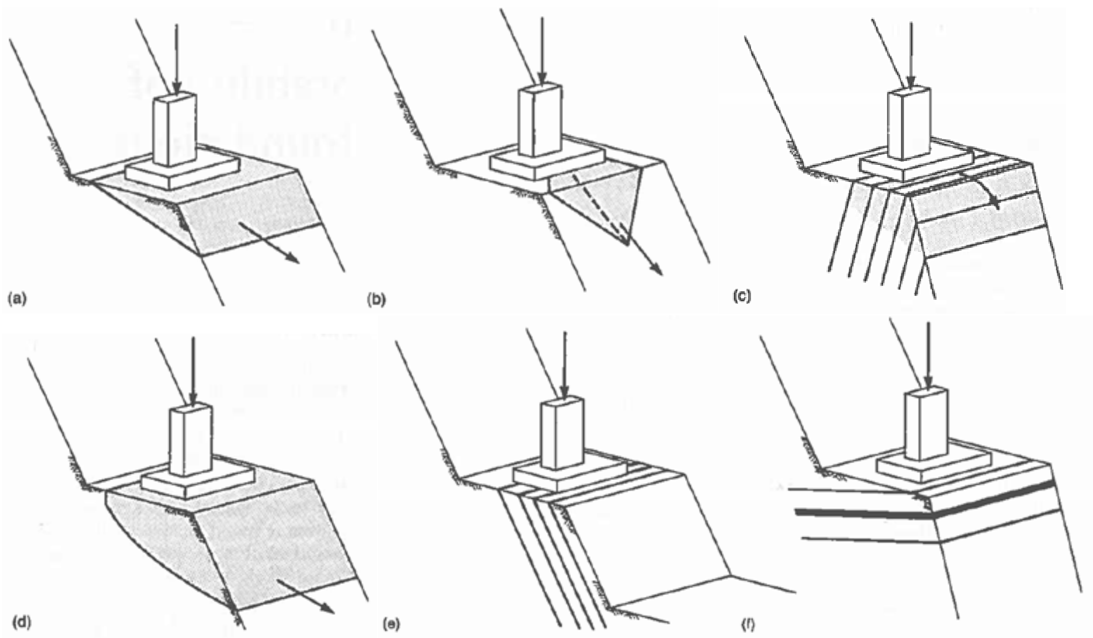


Figure 5.5 – Bearing capacity of a foundation on rock in slopes with possibly containing inclined discontinuities or weakness zones, either as planar sliding, wedge sliding, toppling, or slumping failure or as stable but with excessive deformation

(2) <REQ> Determination of bearing capacity of a spread foundation on rock behaving as a discontinuous medium shall comply with EN 1997-2 and 8.1.3 for failure along discontinuities.

(3) <REQ> Determination of bearing capacity of a spread foundation on rock mass behaving as a continuous medium shall comply with EN 1997-2, 8.1.2.

5.5.2.4 Bearing pressures for structural analysis

(1) <PER> 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) <PER> A more detailed analysis of soil-structure interaction may be undertaken.

(3) <REQ> The distribution of bearing pressure beneath a flexible foundation shall take into account the stiffness of the foundation and the supported structure.

(4) <PER> 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, in order to determine the bending moments and shear forces.

NOTE 40. Formulae for the relative stiffness of a spread foundation on elastic ground and for subgrade modulus are provided in Annex B.14.

NOTE 41. A method for determining whether a foundation is rigid or flexible on the basis of the relative stiffness value is given in Annex B.14.

NOTE 42. 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) <PER> The resistance of a spread foundation to sliding may be determined as the sum of the resistance to sliding on its base (either R_{Tu} for undrained conditions or R_T for drained conditions) plus any resistance to sliding R_p caused by earth pressure on the face of the the foundation, accounting for the deformation compatibility of the two resistances.

(2) <REQ> 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.

(3) <PER> The undrained sliding resistance (R_{Tu}) of a spread foundation on its base on soil or fill may be determined from Formula (5.8):

$$R_{\mathrm{Tu}} = A_{\mathrm{red}} k_{\mathrm{cu}} c_{\mathrm{u}} \tag{5.8}$$

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.

(4) <RCM> 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.

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

(6) <PER> The drained sliding resistance (R_T) of a spread foundation on its base on soil or fill may be determined from Formula (5.9):

$$R_{\mathrm{T}} = N' \tan\delta \tag{5.9}$$

where:

N' is the effective component of the applied force acting normal to the foundation base;

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

(7) <REQ> The value of the ground-structure interface coefficient ($\tan\delta$) shall satisfy Formula (5.10):

$$\tan\delta \leq k_{\tan\delta} \tan\varphi \tag{5.10}$$

where:

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

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

(8) <RCM> 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.

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

(10) <RCM> When verifying the sliding resistance of a spread foundation, the representative angle of internal friction of soil or fill should be taken as its critical state value φ'_{cs} to account for inevitable disturbance of the soil or fill beneath the foundation.

(11) <RCM> 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.

(12) <RCM> The value of R_p should be determined taking account of the nature of the ground including any backfill within the horizontal zone of influence.

5.5.4 Settlement

(1) <REQ> The following components of settlement shall be taken into account when calculating the settlement of spread foundations:

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

NOTE 43. Calculation models for settlements of spread foundations are given in Annexes B7 to B13 for situations where comparable experience exists.

NOTE 44. Consolidation and creep can occur simultaneously, particularly in thick layers of soil of low permeability.

NOTE 45. Settlement by consolidation typically occurs in fine soils with a high degree of saturation.

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

(3) <PER> 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 46. Information regarding the use of calculation models for settlement is provided in Annexes B.7 to B11.

(4) <RCM> Models that are not validated by comparable experience should not be used for calculating settlements.

(5) <RCM> The depth of the compressible soil layer to be taken into account 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.

(6) <RCM> The following factors potentially causing additional settlement to the ones due to loading should be taken into account:

– 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.

(7) <RCM> The settlement of spread foundations should be determined assuming a distribution of bearing pressure resulting from the ground-foundation interaction.

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

(9) <PER> The tilting of an eccentrically loaded foundation, which is of limited size and hence assumed to be rigid, may be estimated 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.

(10) <PER> An analysis of ground-structure interaction may be used to obtain a more detailed calculation of differential settlements.

NOTE 47. Differential settlement calculations that ignore the stiffness of the structure tend to be over-predictions.

5.5.5 Heave

(1) <REQ> When assessing heave for the verification of a serviceability limit state, calculation shall be based on heave due to 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 48. An example of a chemical reaction in the ground causing heave is the transformation of anhydrite (anhydrous calcium sulphate) to gypsum.

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

5.6 Ultimate limit states

5.6.1 General

(1) <REQ> The ultimate limit states of a spread foundation involving overall stability, bearing, and sliding failure shall be verified using Formula (8.1) of EN 1990.

(2) <REQ> 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) <REQ> Spread foundations shall be verified against the occurrence of an ultimate limit state of overall stability in accordance with Clause 4.

NOTE 49. This is particularly relevant when the spread foundation is within the zone of influence of sloping ground; excavations or cuts; 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

(1) <REQ> 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 resulting force acting normal to the foundation base;

(2) <RCM>The 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) <RCM> The design eccentricity of the load acting on a spread foundation should be determined using design actions.

NOTE 50. The design eccentricity is calculated using the partial factors given in Table 5.2 (NDP) and

NOTE 51. Table 5.3 (NDP), unless the National Annex gives different values.

NOTE 52. When calculated using partial factors on actions from Design Case DC1, the design eccentricity of loading e_{d} should be 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
$e_{\mathrm{d}} \leq \left(\frac{7}{15}\right) B$	$e_{\mathrm{d}} \leq \left(\frac{37}{80}\right) D$	$\left(1 - 2\frac{e_{\mathrm{B,d}}}{B}\right)\left(1 - 2\frac{e_{\mathrm{L,d}}}{L}\right) \geq \frac{1}{15}$

(4) <REQ> 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 taking into account the magnitude of construction tolerances.

(5) <RCM> 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 53. The value of ΔB is 0.1 m, unless the National Annex gives a different value.

5.6.2.3 Sliding failure

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

(2) <REQ> 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.

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

(4) <REQ> The design 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 54. The displacements required to mobilize shear resistance at the base of the foundation are much lower than the displacements required to mobilize earth pressures in foundation front face.

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

(6) <REQ> 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 taken into account.

(7) <REQ> The possibility that face resistance may not be available as a result of the soil in front of the foundation being removed by erosion or human activity shall be taken into account.

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

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

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 material factor;

(9) <REQ> When using the resistance factor approach, the design undrained shearing resistance R_{Tud} of a spread foundation against sliding shall be determined using Formula (5.15):

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

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

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

(10) <REQ> The design undrained sliding resistance R_{Tud} shall comply additionally with (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.

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

where:

$N_{\mathrm{rep,fav}}$ is the minimum (favourable) representative force acting normal to the foundation base.

NOTE 55. The value in (5.18) is 0.4 unless the National Annex gives a different value.

(11) <REQ> When using the material factor approach, the design drained sliding resistance R_{Td} of a spread foundation on ground shall be determined from Formula (**Fel! Hittar inte referenskälla.**):

$$R_{\mathrm{Td}} = N'_d \tan \delta_{\mathrm{d}} = N'_d \left(\frac{\tan \delta_{\mathrm{rep}}}{\gamma_{\tan \delta}} \right) \tag{5.17}$$

where:

N'_d is the design value of the effective vertical component of the total action acting normal to the foundation base;

δ_d is the design value of interface friction between the foundation and the ground;

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

$\gamma_{\tan\delta}$ is a partial factor applied to the coefficient of interface friction.

NOTE 56. N'_d is determined applying the corresponding partial factors γ_F to the different actions acting on the foundation base. Values of partial factors γ_F are given in Table 5.2, for persistent and transient design situations, and Table 5.3, for accidental design situations.

(12) <REQ> When using the resistance factor approach, the design drained sliding resistance R_{Td} of a spread foundation on ground shall be determined using Formula (**Fel! Hittar inte referenskälla.**):

$$R_{\mathrm{Td}} = \frac{N'_d \tan \delta_{\mathrm{rep}}}{\gamma_{\mathrm{RT}}} \tag{5.18}$$

where:

N'_d is the design value of the effective vertical component of the total action acting 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 57. N'_d is determined applying the corresponding partial factors γ_F to the different actions acting on the foundation base. Values of partial factors γ_F are given in Table 5.2, for persistent and transient design situations, and Table 5.3, for accidental design situations.

(13) <REQ> In determining $N_{Gd,fav}$ and $N_{G,rep,fav}$, account shall be taken of whether T and N are dependent or independent actions.

5.6.2.4 Toppling and overturning failure

(1) <REQ> The stability against toppling of a spread foundation shall be verified in accordance with EN 1990.

NOTE 58. Toppling is rotational failure that does not involve failure of the ground.

(2) <REQ> Overturning of a spread foundation on soil or fill, including that of a gravity retaining wall, a reinforced fill structure, and a soil nailed structure, shall be verified for bearing failure according to 5.6.2.2.

5.6.3 Verification by prescriptive rules

(1) <REQ> EN 1997-1, 4.5 shall apply.

(2) <Guidance on the use of the presumed bearing pressures is given in the National Annexes or, where not specified, agreed by the relevant parties for a specific project.

5.6.4 Verification by testing

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

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

(3) <REQ> When evaluating the results of large-scale foundation tests to verify limit states, any excess groundwater pressures beneath the foundation shall be measured and taken into account.

(4) <REQ> When using a test to verify limit states for a spread foundation, differences in scale and response between the test foundation and the real foundation shall be taken into account, including the adverse influence of weak layers within the zone of influence of the test.

5.6.5 Verification by the Observational Method

(1) <RCM> EN 1997-1, 4.7 shall apply.

5.6.6 Partial factors

(1) <REQ> EN 1997-1, 4.4.1(4) shall apply.

NOTE 59. Values of the partial factors are given in in Table 5.2 (NDP) for persistent and transient design situations and in

NOTE 60. Table 5.**3** (NDP), for accidental design situations, unless the National Annex gives different values.

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 (e)[3]	
			(a)	(b)	(c)	(d)	(e)
Overall stability	See Clause 4						
Bearing and sliding resistance	Actions and effects-of-actions	γ_F and γ_E	DC1[1]	DC3[1]	DC1[1]	DC1[1]	DC4
	Ground properties	γ_M	M1[2]	M2[2]	M2[2]	Not factored	
	Bearing resistance	γ_{RN}	Not factored			1,4	
	Sliding resistance	γ_{RT}	Not factored			1,1	

[1]Values of the partial factors for Design Cases (DCs) 1, 3, and 4 are given in EN 1990 Annex A, Table A.1.8.
[2]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1 Annex A.1.
[3]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)	(c)	
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[1]	M2[1]	M2[1]	Not factored
	Bearing resistance	γ_{RN}	Not factored			1,20
	Sliding resistance	γ_{RT}	Not factored			1,05

[1]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1 Annex A, Table A.2.

(2) <PER> If the resistance factor approach is used to determine the bearing resistance of spread foundations under inclined loading, Design Case 4 may be used instead of Design Case 1, provided the condition in Formula (5.19) is satisfied:

$$T_{\text{rep}} \leq 0{,}2N_{\text{rep}} \tag{5.19}$$

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.

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

(4) <PER> Provided the conditions specified in EN 1997-1 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 61. 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) <REQ> Verification of serviceability limit states for spread foundations shall comply with EN 1990, 8.4.

(2) <REQ> Verification of serviceability of strip and raft foundations shall comply with EN 1997-1, 9.

(3) <REQ> The adverse effects of foundation displacements shall be taken into account both in terms of displacement of the entire foundation and differential displacements of parts of the foundation.

(4) <REQ> Account shall be taken of displacements caused by actions on the foundation, including the actions listed in EN 1997-1, 4.3.1.2(1).

(5) <REQ> In determining the magnitude of foundation displacements, account shall be taken of comparable experience, as defined in EN 1997-1, 3.1.2.3.

(6) <REQ> The effect of neighbouring foundations, fills, and excavations shall be taken into account, including the stress increase in the ground and its influence on ground compressibility and displacement.

5.7.2 Settlement

(1) <REQ> To ensure the avoidance of a serviceability limit state, determination of differential settlements and relative rotations shall take account of both the distribution of loads and the possible variability of the ground.

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

5.7.3 Tilting

(1) <REQ> 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) <REQ> 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) <RCM> 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 during execution and service life

5.8.1 General

(1) <REQ> EN 1997-1, 10 shall apply.

(2) <REQ>The execution of concrete spread foundations shall comply with EN 13670.

5.8.2 Inspection

(1) <REQ> EN 1997-1, 10.3 shall apply.

5.8.3 Monitoring

(1) <REQ> EN 1997-1, 10.4 shall apply.

5.8.4 Maintenance

(1) <REQ> EN 1997-1, 10.5 shall apply.

(2) <REQ> Drainage systems around spread foundations should be designed for ease of maintenance and renewal during the design life of the structure.

5.9 Testing

(1) <REQ> EN 1997-1, 11 shall apply.

(2) <REQ> If Plate Loading Tests are executed on soil or fill, they shall comply with EN ISO 22476-13 (publication yet to be confirmed).

(3) <RCM> The results of Plate Loading Tests should only be used to verify a design if:

- the size of the plate has been chosen considering the width of the planned spread foundation (in which case the observations are transformed directly); and
- a homogeneous layer up to two times the width of the planned spread foundation exists (in which case the results of smaller sized plates – not considering the planned foundation width – are used to transform the results on an empirical basis to the actual foundation size).

NOTE 62. The depth of the zone tested by the PLT 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.

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

NOTE 63. An adjusted elasticity method is given in Annex B.7.

(5) <RCM> 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 taken into account.

(6) <PER> Dummy footing tests, skip tests, zone tests, and small-scale prototype spread foundation 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) <REQ> EN 1997-1, 12 shall apply.

6 Piled foundations

6.1 Scope and field of application

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

(2) <RCM> Piles should be classified according to their method of execution.

NOTE 64. The classification is given in Table 6.1 (NDP) unless the National Annex gives a different classification.

NOTE 65. The pile type is used to determine resistance factors, see 6.6.3.

NOTE 66. 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) <REQ>EN 1997-1, 4.2.2 shall apply.

6.2.2 Geometrical properties

6.2.2.1 General

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

NOTE 67. For replacement piles without casing, the pile diameter is considered to be equal to the nominal diameter of the drilling tool.

NOTE 68. For replacement piles with casing, the pile diameter is considered to be equal to the external diameter of the casing.

6.2.2.2 Single Pile

(1) <REQ> 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 69. Nominal dimensions are given in the execution standards given in 6.8.

(2) <REQ> The adverse effects of pile geometrical imperfections shall be taken into account in the verification of limit states.

NOTE 70. The execution standards given in 6.8 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 71. Annex C.13 provides calculation models in order to take into account second order effects induced by some geometrical imperfections.

6.2.2.3 Pile groups

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

(2) <RCM> Pile spacing should be sufficient to avoid damage to previously constructed piles, allowing for pile positional and verticality tolerances.

(3) <PER> Closer pile spacing may be used when the piles form part of an embedded earth retaining structure or where pile interaction has been explicitly considered.

6.2.2.4 Zone of influence

(1) <REQ> EN 1997-1, 4.2.1.1 shall apply.

6.2.3 Actions and environmental influences

6.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

6.2.3.2 Permanent and variables actions

(1) <REQ> Actions for piled foundations shall include but not be 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;
- accidental and seismic actions;
- 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.

6.2.3.3 Cyclic and dynamic actions

(1) <REQ> EN 1997-1, 4.3.1.3 shall apply.

(2) <REQ> The adverse effects of cyclic and dynamic stresses in piled foundations on long-term bearing and transverse resistance shall be considered.

NOTE 72. Cyclic and dynamic stresses can result in reduced ground strength and stiffness leading to additional pile displacements and loss of resistance.

NOTE 73. In coarse fills and soils, cyclic and dynamic stresses can result in densification of the ground leading to increased stiffness, particularly in the horizontal direction.

NOTE 74. The compression, tension or lateral pile resistances can be significantly affected by cyclic actions depending on the amplitude and the number of cycles:

- For a number of cycles around 100; cyclic effects are significant when the load range is greater than about 20%, or the cyclic load amplitude is greater than about 10% of the ultimate static pile resistances;
- For a number of cycles around 10 cycles, cyclic effects are significant when the load range is greater than about 35% or the cyclic load amplitude is greater than about 20 % of the ultimate static pile resistance.

6.2.3.4 Actions due to ground displacement

(1) <REQ> The adverse effects of vertical and horizontal movements of the ground on the piled foundation shall be considered.

NOTE 75. See 6.5.2 for a method of calculating downdrag action on piles.

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

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

6.2.3.5 Environmental influences

(1) EN 1997-1, 4.3.1.4 shall apply.

6.2.4 Limit states

6.2.4.1 Ultimate Limit States

(1) <REQ> In addition to EN 1997-1, 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, EN 1992 or EN 1996 according to the pile type);
- combined failure of the ground and the structural pile element;
- failure of the supported structure caused by excessive pile movement.

(2) <RCM> Ultimate limit states other than those given in (1) should be verified as necessary.

6.2.4.2 Serviceability Limit States

(1) <REQ> In addition to EN 1997-1, 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) <RCM> Serviceability limit states other than those given in (3) should be verified as necessary.

6.2.5 Robustness

(1) <REQ> EN 1997-1, 4.1.4 shall apply.

6.2.6 Ground investigation

6.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply.

(2) <REQ> The ground investigation shall include any combination 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;
- geological description of the ground conditions;
- laboratory tests to determine ground shear strength and stiffness from samples boreholes obtained.

(3) <REQ> For piled foundations on or in rock mass, the extent of the ground investigation shall take into account the assumed pile load – rock strength ratio.

(4) <RCM> For piled foundations on or in rock mass, where the assumed pile load – rock strength ratio is low, the ground investigation should also include:

- 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.

(5) <RCM> For piled foundations on or in rock mass, where the assumed pile load – rock strength ratio is high, the ground investigation should also include:

- measure while drilling;
- borehole video logging;
- comprehensive comparable experience.

(6) <REQ> The aggressiveness of the ground and groundwater shall be determined during the ground investigation by considering existing information or results of laboratory tests.

(7) <PER> The ground investigation may be extended to 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.

6.2.6.2 Minimum extent of in situ testing

(1) <REQ> The depth and horizontal extent of the in situ testing shall be sufficient to determine the ground conditions within the zone of influence of the structure according to 6.2.2.4.

(2) <REQ> The situ testing shall determine ground conditions over the full depth of the piled foundation including any overlying fills or low strength soils and not be limited to the anticipated founding stratum at or below the pile base.

(3) <RCM> The minimum depth of situ testing below the anticipated base of a piled foundation d_{min} in soils or very weak and weak rock masses should be determined from Formula (6.1):

$$d_{min} = max\left(5\ m; 3B_{\mathrm{b,eq}}; p_{\mathrm{group}}\right) \tag{6.1}$$

where:

$B_{\mathrm{b,eq}}$ 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) <RCM> The value of d_{min} in strong rock masses should be determined from Formula (6.2):

$$d_{min} = max\left(3\ m; 3B_{\mathrm{b,eq}}\right) \tag{6.2}$$

(5) <RCM> 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) <PER> The value of d_{min} 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.

6.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

(2) <REQ> Piled foundations shall be classified into GC 2 or 3.

6.3 Materials

6.3.1 Ground properties

(1) <REQ> EN 1997-2, 7 to 12 shall apply.

(2) <PER> 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) <RCM> The effect of subsequent excavation, placement of overburden, or changes in groundwater pressure on the values of ground properties should be taken into account.

(4) <RCM> 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) <REQ> EN 1997-1, 5.5, shall apply.

(2) <REQ> Exposure classes for concrete shall comply with EN 206 and concrete cover requirements with EN 1992-1-1.

NOTE 76. For many reinforced concrete piles or piled foundations constructed in natural ground, the exposure class will be XA1, XA2 or XA3. Currently EN 1992-1-1 does not provide guidance for the cover allowance for durability for these exposure classes.

(3) <RCM> In the absence of alternative guidance, the minimum cover for environmental conditions $c_{\mathrm{min,dur}}$ should be 25 mm for reinforced concrete used for both precast and cast-in-place piles.

(4) <RCM> 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.

(5) <PER> The value for Δc_{dev} for precast piles may be reduced in accordance with EN 1992-1-1, 6.4.3(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) <REQ> Grout and mortar used for piles shall comply with the rules given for concrete in EN 1992-1-1, EN 206, EN 445, EN 447, and EN 14199, as appropriate.

NOTE 77. EN 1992-1-1 and EN 206 do not explicitly cover grout or mortar, but they can be used to specify these materials.

(2) <PER> In the absence of guidance, the compressive and tensile strength of grout or mortar may be determined from comparable experience or testing.

(3) <PER> Exposure classes for grout and mortar and rules for durability may comply with 2.

6.3.4 Steel

(1) <REQ> EN 1997-1, 5.6, shall apply.

6.3.5 Steel reinforcement

(1) <REQ> EN 1997-1, 5.6, shall apply.

6.3.6 Ductile cast iron

(1) <REQ> EN 1997-1, 5.7, shall apply.

6.3.7 Timber

6.4 Groundwater

(1) <REQ> EN 1997-1, 6 shall apply.

6.5 Geotechnical analysis

6.5.1 General

(1) <REQ> EN 1997-1, 7 shall apply.

(2) <PER> Combined axial and lateral loading may be analysed by separating each load component and applying the principle of superposition, provided pile internal stresses and displacement behaviour remain substantially elastic.

(3) <RCM> 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) <REQ> Actions due to ground displacement shall be modelled either by treating the displacement as an action or as an equivalent design force (design action).

(2) <RCM> Evaluation of an equivalent 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) <REQ> The adverse effects of the drag force caused by moving ground shall be included in the verification of serviceability and ultimate limit states.

(2) <RCM> The effects of the downdrag or negative shaft friction should be modelled by carrying out an ground-pile interaction analysis to determine the depth of the neutral plane L_{dd} corresponding to the point where the pile settlement s_{pile} equals the ground settlement s_{ground}.

NOTE 78. This also marks the boundary between negative shaft friction above, and positive shaft friction below the neutral plane.

NOTE 79. The depth of the neutral plane L_{dd} is usually different for serviceability and ultimate conditions.

(3) <RCM> 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.

(4) <PER> In addition to EN 1990, 6.1.1(4) and 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) <RCM> The equivalent drag force D_{rep} should be determined from Formula 6.15:

$$D_{\mathrm{rep}} = p \int_0^{L_{\mathrm{dd}}} \tau_{\mathrm{s}} \cdot \mathrm{dz} \tag{6.3}$$

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.

NOTE 80. Calculation models for downdrag are included in Annex C.10.

(6) <RCM> The value selected for the unit shaft friction causing downdrag should be based on upper (superior) ground parameters, in order to provide a cautious estimate of the downdrag force.

6.5.2.3 Heave

(1) <REQ> 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) <REQ> The adverse effects of heave caused by moving ground shall be included in the verification of serviceability and ultimate limit states, especially in order to avoid tensile failure of the pile.

(3) <RCM> Verification of serviceability limit states should take account of 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) <PER> Long-term heave may be disregarded where the imposed permanent actions exceed the heave load.

(5) <REQ> The adverse effect of ground heave shall be included in the structural design of the pile to ensure tensile failure of the pile does not occur.

6.5.2.4 Transverse loading

(1) <REQ> 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) <REQ> The axial resistance of a single pile shall be determined from the results of field and laboratory testing or load tests based on comparable experience.

(2) <REQ> 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).

(3) <REQ> 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 81. Methods of calculating base and shaft resistance are included in Annexes C.5 and C.6 for ground parameters, C.7 for cone penetration test methods, and C.8 for pressuremeter methods.

(4) <RCM> 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 82. The use of equation 6.4 assumes the compatibility of the displacements to mobilise both base resistance and shear resistance taking into account the pile geometry and the difference of stiffness between the ground and the pile.

NOTE 83. 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 B/L. The shaft resistance of soil layers reduces to 0, when a pile is socketed in competent rock with a high E_{rm} compared to E_c.

(5) <REQ> Temporary support ('hold-up') from settling ground that will reduce or reverse during the working life of the pile foundation shall not be included in the computation of shaft resistance.

(6) <RCM> The weight of the pile element 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.

(7) <PER> The weight of the pile element 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.

(8) <RCM> The pile base resistance in compression R_b should be determined from Formula (6.5):

$$R_b = A_b \cdot q_b \tag{6.5}$$

where:

q_b is the end bearing or base resistance;

A_b is the area of the pile base.

(9) <RCM> The pile shaft resistance R_s in compression should be determined from Formula (6.6):

$$R_s = \sum_{i=1}^{n} A_{s,i} q_{s,i} \tag{6.6}$$

where:

$q_{s,i}$ is the unit shaft friction in the i-th geotechnical unit;

$A_{s,i}$ 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) <RCM> The pile shaft resistance in tension R_{st} should be determined from Formula (6.7):

$$R_{st} = \sum_{i=1}^{n} A_{s,i} q_{st,i} \tag{6.7}$$

where:

$q_{st,i}$ is the shaft friction in tension in the i-th geotechnical unit;

6.5.3.2 Testing

(1) <PER> The axial compressive resistance of a single pile at the ultimate limit state may be determined from the results of static, dynamic impact, or rapid load ultimate control tests.

(2) <PER> The axial tensile resistance of a single pile at the ultimate limit state may be determined from the results of static tests.

(3) <RCM> Determination of the axial resistance of a single pile from static pile load tests should account for potential temporary hold-up.

(4) <PER> 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 hold-up.

(5) <RCM> 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) <RCM> 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 testing.

(7) <RCM> 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.

(8) <REQ> In the absence of site-specific correlations, the validity of dynamic impact or rapid load testing shall have been established using static load testing previously carried out in documented

comparable situations on the same pile type, with similar geometry, in comparable ground conditions, and tested to similar load levels.

(9) <PER> 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) <PER> The compressive resistance of a pile may be determined from the results of wave equation analysis based on the registered energy transformation 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, of similar installation method and in comparable ground conditions.

(11) <PER> 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) <PER> Analysis of the results of dynamic impact tests may be carried out using wave equation analysis for confirmation of design verification or for interpolation between test locations when it is necessary to modify the design to cater for different design situations.

(13) <PER> Wave equation analysis may also be used to determine the effect of small changes (less than 30%) in dimensions, length, impact energy, and final set of piles that are not load tested.

(14) <PER> Wave equation analysis or driving formulae may be used to determine driving criteria for control purposes.

6.5.3.3 Prescriptive rules

(1) <PER> The axial compressive resistance of a single pile may be determined using prescriptive measures where specified by a relevant authority.

NOTE 84. Prescriptive rules for pile design can be specified in the National Annex.

6.5.4 Transversely loaded single piles

(1) <PER> The transverse resistance of a single pile may be determined by calculation or by testing.

(2) <PER> The transverse resistance of a single pile may be determined assuming rotation or translation as a rigid body for short piles (ratio between pile length and diameter less than 6) or bending failure and local yielding for longer piles.

NOTE 85. Note that in many cases, design of piles for transverse loading is likely to be controlled by serviceability rather than ultimate limit states and simple mechanisms might not be sufficient.

(3) <REQ> Temporary support from moving ground that will reduce or reverse during the design service life of the pile foundation shall not be included in the computation of transverse resistance.

(4) <REQ> 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 of the fixity of pile base.

NOTE 86. It is common to assume free-head or fixed-head conditions, although in reality the pile head fixity will be in between these two extremes.

(5) <RCM> The transverse resistance of a single pile should take account of potential variations of ground stiffness with depth and, 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 87. For example, upper bound stiffness for stiff soil layers and lower bound for less stiff layers.

6.5.5 Pile groups

(1) <PER> 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) <REQ> Pile group design shall take into account 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) <RCM> Calculation of pile group effects should take into account 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) <PER> 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 taken into account.

(5) <RCM> The ultimate vertical resistance of a pile group R_{group} should be determined from Formula (6.8):

$$R_{\text{group}} = \min\{\textstyle\sum_i^n R_{\text{i}}\,;\, R_{\text{block}}\} \quad (6.8)$$

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) <RCM> In the case of tension loading, the reduction in effective vertical stresses in the ground should be taken into account when deriving the shaft resistance of individual piles in the group.

NOTE 88. For the evaluation of the block failure of pile groups subject to axial tension see Annex C.11.

(7) <RCM> For pile groups, the effects of interaction, the shadow effect of closely spaced piles, and head fixity should be accounted for when deriving transverse resistance from the results of calculations or load tests on individual test piles.

(8) <RCM> Where interaction effects between piles are 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 loading.

(9) <REQ> 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 weakest pile in the group.

6.5.6 Piled rafts

(1) <RCM> The ultimate compressive resistance of a piled raft $R_{\text{piled-raft}}$ should be determined from Formula (6.9):

$$R_{\text{piled-raft}} = \left(\sum_i^n R_{\text{c,i}} + R_{\text{raft}}\right) \tag{6.9}$$

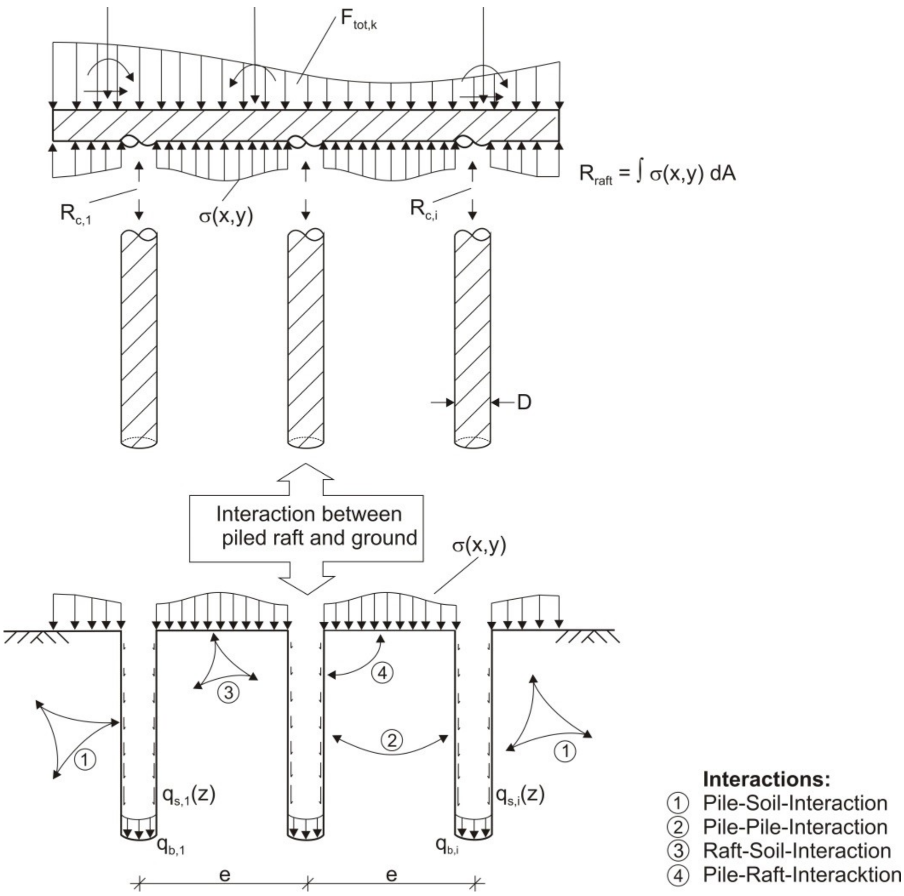
where:

R_{raft} is the ultimate compressive resistance of the raft.

$R_{\text{c,i}}$ is the compressive resistance of the i-th pile

(2) <REQ> 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.



(22)

Figure 6.1 – Interaction effects of a piled raft

(3) <PER> 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) <PER> 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) <PER> 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.

NOTE 89. This is a simplistic assessment as it does not include for the interaction effect of the piles on the behaviour of the raft foundation.

(6) <PER> 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 90. This is particularly beneficial when piles are used for the purpose of settlement or raft bending moment reduction.

NOTE 91. The limiting value here is not necessarily the same as that of a single pile, since it includes pile-raft interaction effects.

6.5.7 Displacement of piled foundations

6.5.7.1 General

(1) <REQ> Piled foundation settlement or transverse displacement shall be determined from the results of load tests or analytical, numerical or empirical calculations, or prescriptive rules based on the observed performance of comparable single piles or pile groups.

NOTE 92. Except for special cases, it is unlikely that load testing of pile groups is feasible, and performance of pile groups will need to be verified by other methods.

(2) <REQ> The validity of analytical, numerical or empirical calculation methods shall be demonstrated using documented load testing of comparable pile foundation and case histories to confirm that the design methods can provide reliable parameter values and reliably predict pile settlement and transverse behaviour.

(3) <REQ> 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) <PER> The settlement or transverse displacement of a single pile may be determined from load testing or calculated using empirical or analytical methods or numerical modelling.

NOTE 93. Owing to rapid degradation of mobilized ground stiffness with pile head movement, design methods based on nonlinear stiffness models are more appropriate for calculating the transverse response of a pile foundation.

NOTE 94. Load transfer methods are usually sufficient to assess settlement or transverse displacement of single pile.

(2) <RCM> Elastic shortening of the pile shaft under axial loading should be included in the calculation of pile head settlement taking into account the effects of cracks and creep.

6.5.7.3 Pile groups and piled rafts

(1) <PER> The settlement and transverse displacement of pile groups and piled rafts may be determined using empirical or analytical methods or numerical modelling.

(2) <RCM> Calculation methods for pile group design should take account of:

– the impact on 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 95. Finite element or finite difference approaches, boundary element approaches, interaction curve or factor approaches are some examples of appropriate methods.

NOTE 96. The use of load transfer functions without the consideration of any interaction between piles is not appropriate to capture group effects.

(3) Interactions between piles should consider the non-linear behaviour of the ground.

NOTE 97. Linear behaviours tend to overestimate the ground displacement adjacent to pile.

6.5.8 Confirmation of pile design by site-specific load testing or comparable experience

(1) <RCM> 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 98. Unlike static load tests, rapid load tests and static load tests do not provide direct information about the pile behaviour under serviceability limit state conditions.

(2) <PER> Pile design for compression loading may be confirmed using dynamic impact tests or rapid load tests provided that these tests have been validated by static pile load tests.

(3) <PER> Site-specific load testing may be omitted where there is comparable experience or evidence of previous successful use for the same type of pile with similar geometry installed in similar ground conditions.

(4) <RCM> The number and type of site-specific pile loads tests n_{test} to confirm pile design by calculation should be selected according to the purpose of the load test.

NOTE 99. 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)
N = total number of piles in similar ground conditions		

(5) <PER> the value of ntest may vary based on local experiences, ground conditions and pile types.

(6) <PER> When selecting the value of ntest, piles with different geometries may be considered as a single set, provided they are anticipated to exhibit a proportional response to loading.

(7) <PER> The value of ntest may be adjusted proportionately when carrying out both Ultimate and Serviceability Control Tests or when carrying out a mix of static, rapid load, or dynamic impact load tests.

(8) <RCM> All pile load testing should be carried out in accordance with 6.9.

(9) <REQ> The design of the piles shall take into account 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) <REQ> 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.

(2) <REQ> 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:

$R_{c,\mathrm{rep}}$ is the pile's representative total resistance in axial compression;

$R_{b,\mathrm{rep}}$ is the pile's representative base resistance in axial compression;

$R_{s,\mathrm{rep}}$ is the pile's representative shaft resistance in axial compression;

γ_{Rd} is a model factor;

$\gamma_{\mathrm{Rc}}, \gamma_{\mathrm{Rb}}, \gamma_{\mathrm{Rs}}$ are resistance factors.

NOTE 100. Values of γ_{Rc}, γ_{Rb}, and γ_{Rs} are given in Table 6.6 (NDP) unless the National Annex gives different values.

NOTE 101. 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 102. Table 6.*4* (NDP) for verification by testing, unless the National Annex gives different values.

Table 6.3 (NDP) – Model factor γ_{Rd} for verification of axial pile resistance by calculation

Verification by		**Model factor γ_{Rd}**	
Ground Model Method	Ultimate Control Tests as specified in Table 6.2 (NDP)	1.2	
	Extensive comparable[1,2] experience without site-specific Control Tests	1.3	
	Serviceability Control Tests as specified in Table 6.2 (NDP)	1.4	
	No pile load tests and limited comparable experience[1,3]	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[4]	1.15	1.4
	Cone penetration test[4]	1.1	1.1
	Profiles of ground properties based on field or laboratory tests[4,5]	1.2	1.2

[1]'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*,
[2]'Extensive comparable experience' assumes $n \geq 10$
[3]'Limited comparable experience' assumes $n < 10$
[4]Value can be multiplied by 0.9 when accompanied by Ultimate Control Tests as specified in Table 6.4 (NDP)
[5]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)[a]	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)[a]	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)[a]	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 modified as follows:

- +0.1 when calibration is based on extensive comparable experience
- +0.25 when calibration is based on limited comparable experience

[a]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) <REQ> 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) <REQ> 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:

$R_{t,\mathrm{rep}}$ is the pile’s representative axial tensile resistance;

γ_{Rd} is a model factor;

γ_{Rst} is a resistance factor.

NOTE 103. The value of γ_{Rst} is given in Table 6.6 (NDP) unless the National Annex gives different values.

NOTE 104. 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 105. Table 6.4 (NDP) for verification by testing, unless the National Annex gives different values.

6.6.1.3 Verification of transverse resistance

(23)<REQ> The transverse resistance of a single pile shall be verified using Formula (6.14):

$$F_{\mathrm{tr,d}} \leq R_{\mathrm{tr,d}} \quad (6.14)$$

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.

(24) <REQ> If using the material factor approach, the design transverse resistance $R_{\mathrm{tr,d}}$ shall be determined according to EN 1990, Formula (8.12), by applying material factors γ_{M} to the representative values of the material properties X_{rep}.

NOTE 106. The value of γ_{M} is given in Table 6.6 (NDP) unless the National Annex gives different values.

(25) <REQ> If using the resistance factor approach, the design transverse resistance $R_{\mathrm{tr,d}}$ shall be determined according to EN 1990, 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 107. The value of $\gamma_{\mathrm{R,tr}}$ is given in Table 6.6 (NDP) unless the National Annex gives different values.

6.6.1.4 Downdrag

(26) <RCM> Downdrag should be classified as a permanent action arising from the relative axial movement when ground settlement exceeds pile settlement.

NOTE 108. See Annex C.10 for detailed models and combinations of actions for downdrag.

(27) <REQ> The design drag force due to settling ground shall be determined from Formula (6.15):

$$D_{\mathrm{d}} = \gamma_{\mathrm{F,drag}} D_{\mathrm{rep}} \quad (6.15)$$

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.

NOTE 109. The value of $\gamma_{\mathrm{F,drag}}$ is given in Table 6.6 (NDP), unless the National Annex gives a different value.

6.6.1.5 Transverse ground loading

(28) <RCM> 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

(29) <REQ> 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):

$$R_{\mathrm{rep}} = R_{\mathrm{calc}} \quad (6.16)$$

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.

(30) <REQ> 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):

$$R_{rep} = min\left\{\frac{(R_{calc})_{mean}}{\xi_{mean}};\frac{(R_{calc})_{min}}{\xi_{min}}\right\} \tag{6.17}$$

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 110. Values of ξ_{mean} and ξ_{min} are given in Table 6.5 (NDP) unless the National Annex gives different values.

NOTE 111. The correlation factors given in in Table 6.5 (NDP) assume field test profiles arranged on a grid with reference spacing d_{ref} of 30 m.

(31) <REQ> 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 constructed piles.

(32) <REQ> For each single data set defined in (3), the coefficient of variation (CoV) of the computed pile resistance for each profile shall be determined.

(33) <REQ> The values of the correlation factors ξ_{mean} and ξ_{min} 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).

Table 6.5 (NDP) – Correlation factors

Correlation Factor[1]	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								

[1]If all piles are in a group are tested, use ξ_{mean} = 1,0 provided load can be transferred through the pile cap. For individually tested piles, use ξ_{mean} = ξ_{min} = 1,0.

(34) <REQ> 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:

$(R_{\text{test}})_{\text{mean}}$ is the mean pile resistance measured in a set of load tests;

$(R_{\text{test}})_{\text{min}}$ 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.

NOTE 112. Values of ξ_{mean} and ξ_{min} are given in Table 6.5 (NDP) unless the National Annex gives different values.

(35) <REQ> 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.

(36) <PER> 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.

(37) <REQ> If ξ_{mean} and ξ_{min} are reduced according to (8), then the verification of limit states in the pile cap shall take into account the load redistribution.

(38) <PER> The values of ξ_{mean} and ξ_{min} may be calculated by considering the area S corresponding to the number of test profiles N:

$$\xi_{\text{mean}}(S) = 1 + \frac{\text{d}}{d_{ref}}(\xi_{\text{mean}} - 1) \text{ or } \xi_{\text{min}}(S) = 1 + \frac{d_{ave}}{d_{ref}}(\xi_{\text{min}} - 1) \tag{6.19}$$

where:

$\xi_{\text{mean}}(S)$ is the value of ξ_{mean} by considering the area S corresponding to the number of test profiles N;

$\xi_{\text{min}}(S)$ 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 113. Equation 6.18 (NDP) is given unless the National Annex provides different equation.

6.6.2 Pile groups and piled rafts

(39) <REQ> The design resistance of a pile group or piled raft $R_{d,group}$ shall be verified using Formula (6.20):

$$F_{d,group} \leq R_{d,group} \tag{6.20}$$

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.

(40) <REQ> If using the material factor approach, the design resistance $R_{d,group}$ shall be determined according to EN 1990, Formula (8.12), by applying material factors γ_M to the representative values of the material properties X_{rep}.

NOTE 114. Values of γM are given in Table 6.6 (NDP) unless the National Annex gives different values.

(41) <PER> If using the resistance factor approach the design resistance $R_{d,group}$ may be determined from Formula (6.21):

$$R_{d,group} = \frac{R_{rep,group}}{\gamma_{R,group}\gamma_{Rd,group}} or \left(\frac{\sum_i^n R_{c,rep,i}}{\gamma_{Rd}\gamma_{Rc}} + \frac{R_{rep,raft}}{\gamma_{R,raft}}\right) \tag{6.21}$$

where:

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

γ_{Rc} is a resistance factor for individual pile axial compressive resistance;

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

$R_{rep,raft}$ is the representative ultimate vertical compressive resistance of the raft.

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

NOTE 115. The values of γ_R, γ_{Rt}, and $\gamma_{R,raft}$ are given inTable 6.6. (NDP) unless the National Annex gives different values.

NOTE 116. The values of γ_{Rd} and $\gamma_{Rd,group}$ are given 1.0, unless the National Annex gives different values.

6.6.3 Partial factors

6.6.3.1 Single piles

(1) <REQ> EN 1997-1, 4.4.1(4) shall apply.

NOTE 117. 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 118. Unless the National Annex gives a specific choice, the approach between RFA and MFA to be used is as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

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	base	shaft
Axial compressive resistance	Actions and effects-of-actions[1]	γ_F and γ_E	Not Used		All	DC1	
	Drag force due to settling ground	$\gamma_{F,drag}$				1.35	
	Ground properties[2]	γ_M				Not factored	
	Base and shaft resistance in compression	$\gamma_{Rb} \mid \gamma_{Rs}$			Full displacement	1.2 (1.0)[4]	1.0 (1.0)[4]
					Partial displ'ment	1.2 (1.0)[4]	1.0 (1.0)[4]
					Replacement	1.2 (1.0)[4]	1.0 (1.0)[4]
					Unclassified	1.35 (1.0)[4]	1.25 (1.0)[4]
	Total resistance in compression	γ_{Rc}			Full displacement	1.1 (1.0)[4]	
					Partial displ'ment	1.1 (1.0)[4]	
					Replacement	1.1 (1.0)[4]	
					Unclassified	1.3 (1.0)[4]	
Axial tensile resistance	Actions and effects-of-actions[1]	γ_F and γ_E	Not Used		All	DC1	
	Ground properties[2]	γ_M				Not factored	
	Shaft resistance in tension	γ_{Rst}			Full displacement	1.15 (1.0)[4]	
					Partial displ'ment	1.15 (1.0)[4]	
					Replacement	1.15 (1.0)[4]	
					Unclassified	1.4 (1.0)[4]	
Transverse resistance	Actions and effects-of-actions[1,3]	γ_F, and γ_E	DC4	DC3	DC1		
	Ground properties[2]	γ_M	M1	M2	M1		
	Transverse resistance	γ_{Rtr}	Not factored		1.4 (1.1)		
	Ground properties[2]	γ_M	M1				

[1]Values of the partial factors for Design Cases (DCs) 1, 3, and 4 are given in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1, Table 4.7.
[3]Including drag force due to moving ground.
[4]Values in brackets are given for accidental design situations.

6.6.3.2 Pile groups and piled rafts

(1) <REQ> EN 1997-1, 4.4.1(4) shall apply.

NOTE 119. 1. Values of the partial factors for pile groups and piled rafts are given in Table6.7 (NDP) for persistent and transient design situations and for accidental situations unless the National Annex gives different values.

NOTE 120. Unless the National Annex gives a specific choice, the approach between RFA and MFA to be used is as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

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[1]	γ_F and γ_E	DC4	DC3	DC1
	Ground properties[2]	γ_M	M1	M2	Not factored
	Vertical resistance	$\gamma_{R,group}$	Not factored		1.4 (1.1)[3]
		γ_{Rc}			See Table 6.6 (NDP)
		$\gamma_{R,raft}$			1.4 (1.1)[3]
Transverse resistance	Actions and effects-of-actions[1]	γ_F and γ_E	DC4	DC3	DC1
	Ground properties[2]	γ_M	M1	M2	
	Transverse resistance	γ_{Re}	Not factored		1.4 (1.1)[3]
Combined axial and transverse resistance	Actions and effects-of-actions[1]	γ_F and γ_E	DC4	DC3	DC1
	Ground properties[2]	γ_M	M1	M2	Not factored
	Compressive and transverse resistance	$\gamma_{R,group}$	Not factored		1.4 (1.1)[3]

[1]Values of the partial factors for Design Cases (DCs) 3 and 4 are given in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1, Table 4.7.
[3]Values in brackets are given for accidental situations.

6.6.4 Structural design and verification

(1) <RCM> The structural resistance of single piles should be verified in accordance with:

– EN 1992-1-1 for reinforced and plain concrete, grout or mortar piles;
– EN 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) <RCM> Ground stiffness should be determined taking into account the magnitude of any axial or transverse deflection of the pile.

(3) <RCM> The representative value of stiffness should be selected as either an upper or lower value, depending on which is more critical.

NOTE 121. Upper values are sometimes critical when transversal loads are present (e.g. from settling soil).

(4) <RCM> Bending stresses due to initial curvature, eccentricities and induced deflection should be taken into account together with stresses due to transverse load.

(5) <RCM> Buckling and torsional stability should be verified taking account of second order effects, particularly for long slender piles.

NOTE 122. Annex C.13 provides calculation models in order to take into account buckling and second order effects.

6.7 Serviceability limit states

(1) <REQ> EN 1997-1, 9 shall apply.

(2) <REQ> Serviceability behaviour of piled foundations shall be determined in accordance with 6.5.7.

(3) <PER> 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.

NOTE 123. For example, explicit verification of serviceability is often omitted for axially loaded piles founded in medium to dense coarse soils, medium to high strength fine soils, and rock.

(4) <PER> Settlement of a compressively-loaded single pile founded in medium to dense coarse soils, medium to high strength fine soils, and rock may be verified using Formula (6.22):

$$F_{\mathrm{cd,SLS}} \leq \gamma_{\mathrm{b,SLS}} R_{\mathrm{b,rep}} + \gamma_{\mathrm{s,SLS}} R_{\mathrm{s,rep}} \tag{6.22}$$

where:

$F_{\mathrm{cd,SLS}}$ is the design axial compression applied to the pile at the quasi-permanent serviceability limit state, 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;

$\gamma_{b,\mathrm{SLS}}$ is a mobilization partial factor for base resistance in the serviceability limit state

$\gamma_{s,\mathrm{SLS}}$ is a mobilization partial partial factor for shaft resistance in the serviceability limit state

NOTE 124. The values of $\gamma_{b,\mathrm{SLS}}$ and $\gamma_{s,\mathrm{SLS}}$ are respectively 0.3 and 0.5 unless the National Annex gives different values.

(5) <RCM> 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 during execution and service life

6.8.1 General

(1) <REQ> EN 1997-1, 10 shall apply.

(2) <REQ> The execution of piled foundations shall 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.

6.8.2 Inspection

6.8.2.1 General

(1) <REQ> EN 1997-1, 10.3 shall apply.

(2) <RCM> In addition to (1), 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) <RCM> For continuous flight auger and displacement auger 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) <RCM> 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) <RCM> 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) <RCM> Cast-in-place or precast concrete piles should 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) <PER> 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) <PER> Results of dynamic impact load testing may also be used to verify pile shaft integrity.

6.8.3 Monitoring

(1) <REQ> EN 1997-1, 10.4 shall apply.

(2) <RCM> In addition to (1), the Monitoring Plan of piled foundations should comply with the execution standards given in 6.8.1.

(3) <RCM> In addition to (1), the Monitoring Plan should include the following, as appropriate:

- settlement, lateral and distortion measurements of the superstructure;
- vibration measurements;
- settlement, lateral and distortion measurements of nearby sensitive structures.

(4) <RCM> Monitoring of pile execution should be carried out for all piles over the full depth of each pile and should include:

- 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.

(6) <RCM> Installation and monitoring records should be inspected after pile execution to verify conformance of the pile to its design criteria.

6.8.4 Maintenance

(1) <REQ> EN 1997-1, 10.5 shall apply.

(2) <RCM> In addition to (1), the Plan of maintenance of piled foundations should comply with the execution standards given in 6.8.1.

6.9 Testing

6.9.1 General

(1) <REQ> EN 1997-1, 11 shall apply.

(2) <RECM> 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.

NOTE 125. As long as EN ISO 22477-2 and EN ISO 22477-3 are not available, national standards are used.

(3) <REQ> Ultimate Control Tests shall be carried out when verification of limit states is to be based on the results of pile load testing.

(4) <RCM> 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) <RCM> Serviceability Control Tests should be carried out on working piles during the main piling works to specified loads in excess of the design serviceability load for the purpose of verifying acceptable pile movement.

(6) <RCM> Control Tests should also be carried out when observations during pile execution indicates conditions that deviate from the anticipated Ground Model or the pile behaviour differs significantly and unfavourably from the design.

(7) <RCM> 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) <RCM> Trial piles should be installed before commencement of the piling works to confirm the chosen pile type, its design, dimensions, resistance, and performance.

(2) <RCM> If only one trial pile is installed, it should be located in the most adverse ground conditions identified on the project site.

(3) <REQ> 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 given in 6.8.1.

(4) <PER> In cases where it is impractical to install or construct full-size large diameter trial piles, a smaller diameter trial pile can 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

(42) <REQ> The test proof load shall be determined allowing for potential drag force, transverse ground force, and temporary support resistance ("hold-up").

(43) <REQ> The proof load P_P for Ultimate Control Tests shall be determined from Formula (6.23):

$$P_{\mathrm{P}} \geq R_{\mathrm{rep}} + D_{\mathrm{sup}} \tag{6.23}$$

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.

(44) <RCM> The value of D_{sup} should be estimated using superior (upper) ground strength and stiffness properties.

(45) <RCM> In presence of a significant vertical temporary support force provided by the ground, the pile should be instrumented.

(46) <PER> When the pile ultimate resistance is unknown at the time of test, the proof load P_p may be determined from Formula (6.24):

$$P_{\mathrm{P}} \geq \gamma_{\mathrm{Rd}} \cdot \xi \cdot \gamma_{\mathrm{R}} \cdot F_{\mathrm{d,ULS}} + D_{\mathrm{add}} + D_{\mathrm{sup}} \tag{6.24}$$

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.

(47) <REQ> The test proof load P_P for Serviceability Control Tests shall be determined from Formula (6.25):

$$P_P = \gamma_{test} \cdot F_{d,SLS} + D_{add} + D_{sup} \qquad (6.25)$$

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 126. The value of γ_{test} is 1.35, unless the National Annex gives a different value.

(48) <RCM> 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) <RCM> Static load tests in compression should comply with EN ISO 22477-1.

(2) <REQ> 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) <PER> 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) <REQ> 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) <RCM> The ultimate compressive resistance should be mathematically defined as the resistance corresponding to cautious infinite settlement.

(6) <RCM> 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 should be adopted.

(7) <PER> As an alternative to (5) and (6), the ultimate compressive resistance may be determined as:

- the maximum applied test load; or
- the critical creep load complying with EN ISO 22477-1; or
- the test load at a pile head settlement equal to 10 % of the pile's base diameter.

(8) <REQ> 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 127. Unless the national Annex gives a specific choice, 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) <REQ> Interpretation of pile 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 128. 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 129. 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.

6.9.5 Rapid load tests

(1) <RCM> Rapid load tests should comply with EN ISO 22477-10.

(2) <REQ> Rapid load testing shall be carried out to verify the maximum static test proof load P_p.

(3) <RCM> The pile resistance in compression R_c should be determined from the results of the load testing and set equal to the maximum mobilised resistance, with allowance for temporary support resistance.

(4) <RCM> 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) <RCM> Dynamic impact load tests should comply with EN ISO 22477-4.

(2) <REQ> Dynamic impact testing shall be carried out to verify the maximum static test proof load P_p.

(3) <RCM> The pile resistance in compression R_c should be determined from the results of the load testing and set equal to the maximum mobilised resistance, with allowance for any drag force or temporary support resistance.

(4) <PER> Where Ultimate Control Tests using dynamic load testing 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.

6.9.7 Pile test reports

(1) <RCM> Pile test reports should comply with EN 1997-1, 12, and the test standards given in 6.9.1.

(2) <REQ> In addition to (1), 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.

(3) <REQ> Pile load test reports shall comply with 6.9.4-6.9.6 and the test standards given in 6.9.1.

(4) <REQ> In addition to (3), 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.

6.10 Reporting

(1) <REQ> EN 1997-1, 12 shall apply.

7 Retaining structures

7.1 Scope and field of application

(1) <REQ> 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) <REQ>EN 1997-1, 4.2.2 shall apply.

(2) <RCM> The design of retaining structures should take account of the following:

- effects of wall construction, including:
  - provision of temporary support to the sides of excavations;
  - changes of in situ stresses and resulting ground movements caused both by the wall excavation and its installation;
  - disturbance of the ground due to driving or boring operations;
- the required degree of water tightness of the finished wall;
- the practicability of constructing the wall to reach a stratum of low permeability, so forming a water cut-off;
- the practicability of forming anchors in adjacent ground;
- the practicability of excavating between any propping of retaining walls;
- access for maintenance of the wall and any associated drainage measures;
- the appearance and durability of the wall and any anchors;
- for sheet piling, the ability of the section to be installed to the design penetration without loss of interlock;
- the stability of borings or slurry trench panels while they are open.

7.2.2 Geometrical properties

7.2.2.1 General

(2) <REQ> EN 1997-1, 4.3.3 shall apply.

7.2.2.2 Ground surfaces

(1) <REQ> 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 130. Anticipated excavation includes post-construction excavation in front of the structure, e.g. due to buried services maintenance.

(2) <RCM> 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, $\Delta a = \min(0.1\ h_s; 0.5\ \text{m})$, where h_s is the distance between the lowest support and excavation level.

(3) <PER> 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) <RCM> Values of Δa larger than those given in (2) should be used when the surface level is particularly uncertain.

NOTE 131. This may be the case for marine structures during dredging operations.

7.2.2.3 Zone of influence

(1) <REQ> EN 1997-1, 4.1.2.1 shall apply.

7.2.3 Actions and environmental influences

7.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

7.2.3.2 Permanent and variables actions

(1) <REQ> Actions for retaining structures shall include but not be 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 of waterfront structures, ice, and wave force;
- potential adverse effects of repeated surcharge loading.

(2) <PER> 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.

7.2.3.3 Cyclic and dynamic actions

(1) EN 1997-1, 4.3.1.3 shall apply.

7.2.3.4 Environmental influences

(1) EN 1997-1, 4.3.1.4 shall apply.

(2) <REQ> The adverse effects of temperature changes shall be considered, especially when determining the loads in struts and props.

NOTE 132. Direct sunlight effects can often be reduced by specific measures, such as coating or painting.

(3) <RCM> Measures should be taken to prevent frost heave and potential ice lenses forming in the ground behind a retaining structure.

NOTE 133. Frost heave can occur in frost susceptible soil, especially in silt.

NOTE 134. Formation of ice lenses can occur in silt with access to free water leading to a significant volume expansion of the soil.

NOTE 135. Possible measures include selection of suitable backfill material, drainage, or insulation.

7.2.4 Limit states

7.2.4.1 Ultimate Limit States

(1) <REQ> In addition to the limit states specified in EN 1997-1, 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.

(2) <RCM> Ultimate limit states other than those given in (1) should be verified as necessary.

(3) <REQ> In addition to EN 1997-1, 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 toppling.

(4) <REQ> Ultimate limit states for gravity walls shall be verified according to Clause 5, in addition to Clause 7.

(5) <REQ> In addition to EN 1997-1, 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) <REQ> Ultimate limit states for embedded retaining walls shall be verified according to this Clause 7.

7.2.4.2 Serviceability Limit States

(1) <REQ> In addition to EN 1997-1, 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) <RCM> Serviceability limit states other than those given in (1) should be verified as necessary.

7.2.4.3 Verification by the Observational Method

(1) <REQ> For all retaining structures, when verification of limit states by the Observational Method is performed, EN 1997-1, 4.7 shall apply.

7.2.5 Robustness

(1) <REQ> EN 1997-1, 4.1.4 shall apply.

7.2.6 Ground investigation

7.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply .

(2) <REQ> Investigations shall include the installation of sufficient devices piezometers to measure groundwater variations within each geotechnical unit taking into account seasonal changes.

7.2.6.2 Minimum extent of in situ testing

(1) <REQ> The depth and horizontal extent of the in situ testing shall be sufficient to determine the ground conditions within the zone of influence.

(2) <REQ> The depth of in situ testing shall comply with EN 1997-3, 5.2.6.2 for gravity retaining structures and with EN 1997-3, 6.2.6.2 for embedded retaining structures.

(3) <REQ> The situ testing shall determine ground conditions over the full height of the retaining wall including any overlying fills or low strength soils.

7.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

7.3 Materials

7.3.1 Ground properties

(1) REQ> EN 1997-2, 7 to 12 shall apply.

7.3.2 Plain and reinforced concrete

(1) EN 1997-1, 5.5, shall apply.

7.3.3 Steel

(1) EN 1997-1, 5.6, shall apply.

7.3.4 Sprayed concrete

(1) <REQ> Sprayed concrete shall comply with 10.

7.3.5 Timber

(1) <REQ> EN 1997-1, 5.8, shall apply.

7.3.6 Masonry

(1) <REQ> EN 1997-1, 5.9, shall apply.

7.3.7 Other structural materials

(1) <PER> As an alternative to 7.3.2, 7.3.2, 7.3.4, 7.3.5, and 7.3.6, other materials may be used provided they comply with a material standard specified by the relevant authority or, where not specified, agreed for a specific project by appropriate parties.

7.3.8 Improved ground properties

(1) <REQ> 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 11.

7.4 Groundwater

7.4.1 General

(1) <REQ> EN 1997-1, 6 shall apply.

(2) <REQ> Potential obstruction of natural groundwater flow caused by linear embedded retaining walls shall be considered.

NOTE 136. The obstruction might be caused by the retaining structure itself.

(3) <RCM> Retaining walls in low permeability ground 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 drainage system is installed (see 12); or
- infiltration is prevented; or
- efficient piezometric control is ensured.

(4) <REQ> 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 Drainage systems

(1) <REQ> Clause 12 shall apply.

(2) <REQ> 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) <REQ> EN 1997-1, 7 shall apply.

(2) <RCM> 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 137. The different calculation models are given in Annex D.

(3) <RCM> Prestressing forces exerted on the retaining structure by anchors or struts should be included in the calculation model.

NOTE 138. Limit equilibrium methods, provide reaction forces necessary to ensure stability but do not consider increased prestressing forces needed to reduce displacements.

7.5.2 Determination of earth pressures

(1) <REQ> Determination of earth pressures shall take account of the expected failure mechanisms and deformations at the limit state under consideration.

NOTE 139. 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 140. The term "earth pressure" is precisely meant as ground pressure and includes rock as well.

(2) <PER> When earth pressures are determined by a total stress analysis in a particular geotechnical unit, total stress analysis may only be adopted if comparable experience exists.

(3) <REQ> Calculations of the magnitudes of earth pressures and directions of forces resulting from them shall take account of:

- surcharge on the ground surface;
- inclination of the ground surface;
- inclination of the wall to the vertical;
- water levels and the seepage forces in the ground;
- the swelling potential of the ground;
- the effect of compaction;
- potential for strain ratcheting;
- the amount and direction of the movement of the wall relative to the ground;
- horizontal and vertical equilibrium for the entire retaining structure;
- the shear strength and weight density of the ground;
- inclination of the ground strata and potential discontinuities;
- the effect of initial stresses and stiffness of the ground;
- the anisotropy of the ground;
- the rigidity of the structure and its supporting system relative to the stiffness of the ground;
- wall roughness.

(4) <REQ> 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) <REQ> The value of the ground-structure interface coefficient (tan δ) shall comply with Formula (7.1):

$$\delta \leq k_{\delta}\varphi \qquad (7.1)$$

where:

φ is the value of the ground's coefficient of internal friction;

k_δ is a constant depending on the roughness of the ground structure interface and local disturbance during execution.

NOTE 141. 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 142. 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 143. 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.

NOTE 144. In fine grained soils, it is commonly assumed that $k_\delta = a/c$, where a is the adhesion to the wall and c the soil's cohesion.

(6) <REQ> The value of k_δ shall not exceed 1.0.

(7) <PER> 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.

(8) <RCM> The value of k_δ should not exceed 2/3 for retaining structures formed with smooth surfaces.

NOTE 145. Retaining structures with smooth surfaces include pre-cast concrete and sheet pile walls.

NOTE 146. This limit can also be applied conservatively to retaining structures with rough surfaces.

(9) <RCM> A value of k_δ = 0 should be used for steel sheet piles walls immediately after installation into clay or peat.

(10) <REQ> In the case of structures retaining rock masses, calculations of the ground 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 147. 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) <REQ> Limiting values of earth pressures shall be determined taking account of the relative movement of the ground and the wall at failure and the corresponding shape of the failure surface.

(2) <RCM> When using tabulated values of earth pressure coefficients or computer software based on limit state 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) <RCM> 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) <PER> 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;

$p_{a,min}$ is the minimum value of p_a.

(2) <RCM> 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) <PER> 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):

$$p'_{\mathrm{a}} = K_{\mathrm{a\gamma}}(\overline{\gamma}_{\mathrm{a}} z_{\mathrm{a}} - u_{\mathrm{a}}) - K_{\mathrm{ac}} c' + K_{\mathrm{aq}} q_{\mathrm{a}} \tag{7.3}$$

where, in addition to the symbols defined for Formula (7.2):

$\overline{\gamma_a}$ 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 148. Values of $K_{a\gamma}$, K_{ac}, and K_{aq} are given in Annex D.

(4) <PER> 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):

$$p_{\mathrm{a}} = (\overline{\gamma}_{\mathrm{a}} z_{\mathrm{a}}) - K_{\mathrm{ac,u}} c_{\mathrm{u}} + q_{\mathrm{a}} \geq p_{\mathrm{a,min}} \tag{7.4}$$

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 149. Values of $K_{ac,u}$ are given in Annex D.

(5) <REQ> The value of $p_{a,min}$ shall be ≥ 0.

NOTE 150. The value of $p_{a,min}$ is 10% of the total vertical stress unless the National Annex gives different values.

(6) <RCM> 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.

NOTE 151. Experience suggests that such low pressures do not occur in practice.

7.5.5 Values of passive earth pressure

(1) <PER> 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):

$$p_p = p'_p + u_p \tag{7.5}$$

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) <PER> 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):

$$p'_p = K_{p\gamma}(\overline{\gamma_p} z_p - u_p) + K_{pc} c' + K_{pq} q_p \tag{7.6}$$

where, in addition to the symbols defined above:

$\overline{\gamma_p}$ is the average weight density of the ground over depth z_p;

q_p is any permanent vertical load applied at formation level; and

$K_{p\gamma}$, K_{pc}, and K_{pq} are passive earth pressure coefficients.

NOTE 152. Values of $K_{p\gamma}$, K_{pc}, and K_{pq} are given in Annex D.

(3) <RCM> Coefficients of passive earth pressure should be cautiously assessed for high values of the friction angle (> 40°).

(4) <PER> When using a total stress analysis for calculation of undrained behaviour, Formula (7.7) may be used instead of (7.5):

$$p_p = (\overline{\gamma_p} z_p) + K_{pc,u} c_u + q_p \tag{7.7}$$

where, in addition to the symbols defined above:

$K_{pc,u}$ is a passive earth pressure coefficient for undrained conditions.

NOTE 153. Values of $K_{pc,u}$ are given in Annex D.

(5) <RCM> 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 $\delta = 0$.

(6) <REQ> Only permanent loads shall be considered on the passive side of the retaining structure.

7.5.6 At-rest values of earth pressure

(1) <RCM> The earth pressure coefficient at rest K_0 should be determined according EN 1997-2, 7.1.7 taking into account in addition the type of retaining structures and the conditions of installation.

NOTE 154. Overconsolidated ratio in clay, cylindrical retaining wall, installation of the wall are some examples of conditions that affect the earth pressure coefficient at rest.

(2) <PER> 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):

$$p_0 = p'_0 + u = K_0(\bar{\gamma}z - u + q) + u \tag{7.8}$$

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.

$\overline{\gamma_0}$ is the average weight density of the ground over depth z_0;

q is the vertical load applied at the surface of the ground; and

NOTE 155. Calculation models to determine K_0 are given in Annex D.

7.5.7 Intermediate values of earth pressure

(1) <REQ> Intermediate values of earth pressure, between active and passive limits, shall be determined taking into account the amount of wall movement and its direction relative to the ground.

NOTE 156. Intermediate values of earth pressure occur if the wall movements are insufficient to mobilise the limiting values.

(2) <PER> 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 157. 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) <REQ> The determination of earth pressures acting behind the wall shall take account of any additional pressures generated by compacting backfill, in relation with the procedures adopted for its compaction.

NOTE 158. Guidance for determining these additional pressures is given in Annex D.

7.5.9 Groundwater pressures

(1) <REQ> Determination of groundwater pressures shall comply with EN 1997-1, 6.

7.6 Ultimate limit states

7.6.1 General

(1) <REQ> The verification of ultimate limit states shall consider drained and undrained conditions, as appropriate.

(2) <REQ> 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.

NOTE 159. Anchors could have to resist the highest reaction derived from several ULS calculations (e.g. rotational failure, structural resistance, and overall stability if the retaining structure plays a part in stabilisation).

7.6.2 Overall stability

(1) <REQ> The overall stability of a retaining structure shall be verified in accordance with Clause 4.

NOTE 160. Figure 7.1 gives examples of limit modes for overall stability of retaining structures.

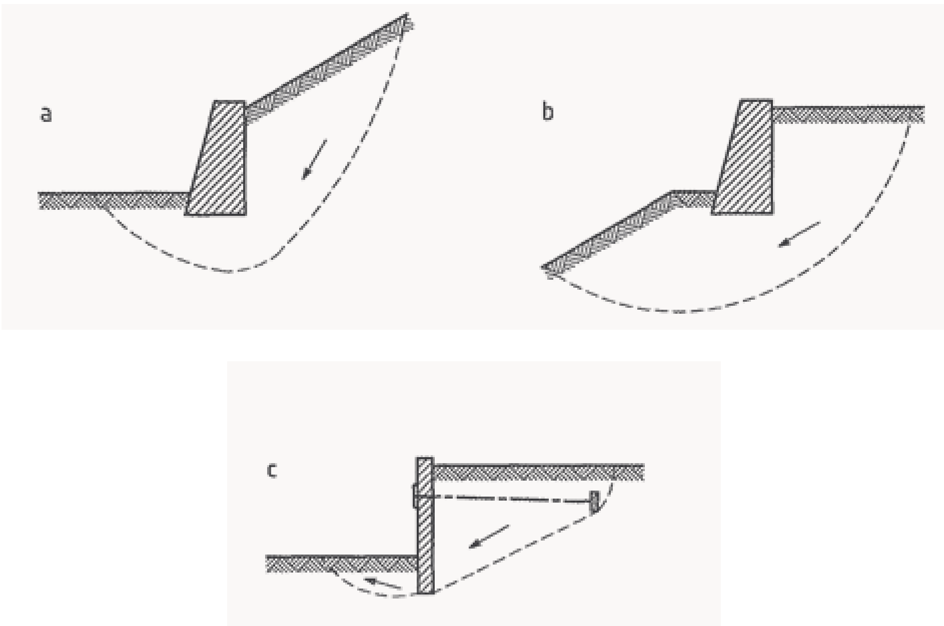


Figure 7.1 — Examples of limit modes for overall stability of retaining structures

(2) <REQ> If stabilising measures are necessary to ensure the overall stability of the site and the retaining structure plays a part in those stabilising measures, then the stability of compound failure surfaces that intersect the retaining structure shall be verified.

NOTE 161. Compound failure surfaces are automatically considered when overall stability is verified in a continuum numerical model.

(3) <RCM> 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 stability), 7.6.5 (stability of excavations), and 7.6.7 (structural failure).

NOTE 162. This ensures consistency and avoids effects of ULS actions on the retaining structure from overall stability calculations being ignored when verifying other local failure mechanisms. This does not exclude that other calculation models are additionally used when checking local failure mechanisms.

NOTE 163. Other information are presented in EN 1997-1, 8.2.

(4) <REQ> 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 EN 1992, EN 1993, EN 1995 or EN 1996 depending on the nature of structural elements (concrete, steel, masonry, timber).

(5) <PER> When a numerical model is used for overall stability calculations with elasto-plastic properties for structural elements, the previous (4) may be omitted provided that the ultimate capacity of structural elements is defined according to EN 1992, EN 1993, EN 1995 or EN 1996 depending on the nature of structural elements (concrete, steel, masonry, timber).

(3) <RCM> If the rotational stability 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 conditions should be used for overall stability calculations:

– the effects of actions into the retaining wall are checked using a continuum numerical model;
– compound 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 164. 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) <REQ> Overall stability of a gravity retaining structure shall be verified according to Clause 4.

(2) <REQ> The bearing, sliding, and toppling resistance of a gravity retaining structure shall be verified according to Clause 5.

7.6.4 Embedded walls

7.6.4.1 Rotational stability

(1) <PER> Resistance to loss of rotational equilibrium may be verified using analytical calculation models or continuum numerical models.

NOTE 165. Verifications consist in checking that the embedded length is sufficient to mobilize the required passive soil resistance.

NOTE 166. Figure 7.2 gives examples of mechanisms involving failure of embedded walls.

NOTE 167. Analytical calculation models include limit equilibrium methods, limit analysis, and beam-on-springs calculations.

NOTE 168. Further information about calculation models is given in Annex D.

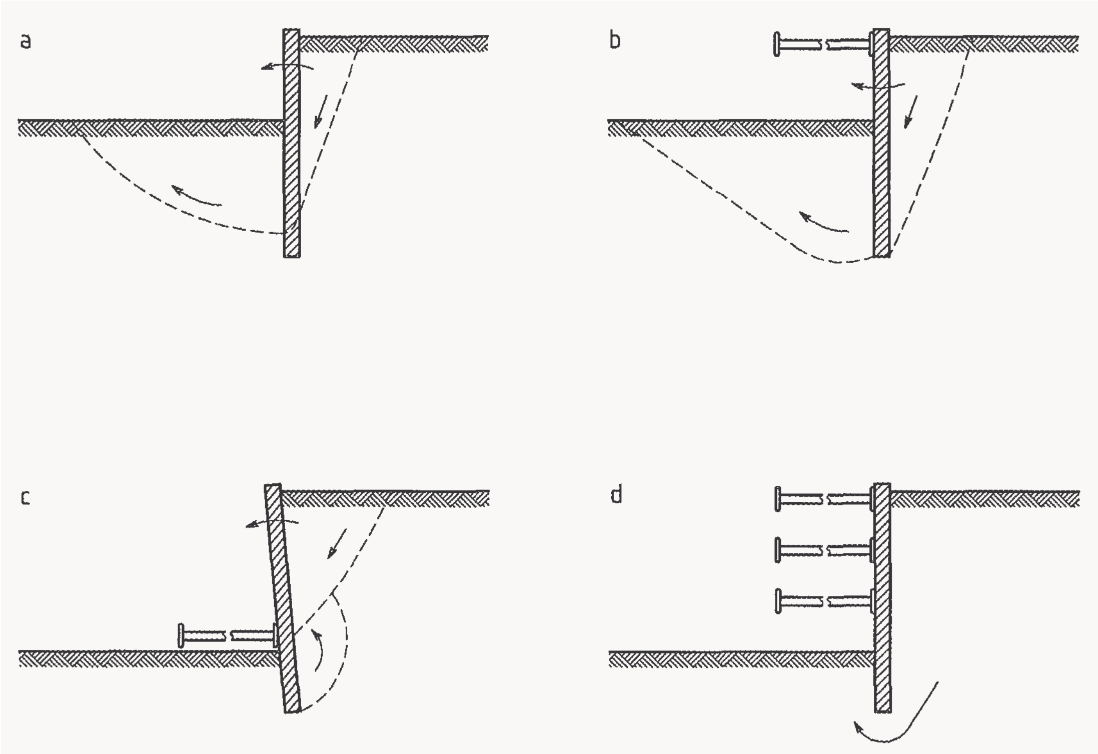


Figure 7.2 — Examples of failure mechanisms for embedded walls

7.6.4.2 Bearing resistance

(1) <REQ> The bearing resistance of an embedded wall that acts as the foundation for a structure, or is subject to significant imposed vertical forces, shall be verified according to this clause and additionally by either Clause 5 or Clause 6, depending on the embedded length.

NOTE 169. Significant vertical forces can be imposed on an embedded wall by inclined anchors.

(2) <REQ> In addition to (1), it shall be verified that the skin friction mobilized to ensure that the vertical equilibrium is compatible with the horizontal equilibrium in terms of stress inclination.

NOTE 170. Skin 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 sense.

NOTE 171. Guidance is provided in 7.5.1(6) and Annex D.

7.6.5 Stability of excavations

(1) <REQ> It shall be verified that failure by heave of the bottom of excavations due to unloading of the ground cannot occur.

NOTE 172. Guidance about suitable models is provided in Annex D.

(2) <RCM> Resistance to basal heave during excavation in fine soils should be verified assuming undrained ground conditions.

(3) <RCM> In addition to (2), resistance to basal heave should be verified assuming drained conditions when undrained conditions are likely to be less critical, particularly in layered soils.

(4) <RCM> Resistance to basal heave in coarse soils should be verified taking in to account hydraulic gradients in the soil.

(5) <REQ> In the presence of hydraulic gradients, it shall be verified that limit states due to internal erosion or piping (see EN 1997-1, 8.2.4.3), hydraulic heave (see EN 1997-1, 8.2.4.2), uplift (see EN 1997-1, 8.2.3.2), or bottom failure mechanisms, i.e. basal heave, are not exceeded.

NOTE 173. See Annex D for basal heave.

(6) <RCM> Measures should be taken to avoid the adverse effects of upward hydraulic gradients.

NOTE 174. Examples of preventive measures include: deep relief wells to protect the passive zone close to embedded walls; increased embedment; embedment down to impervious layers, natural (clayey) or artificial (grouting), that concentrate water head losses.

(7) <REQ> 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) <REQ> It shall be verified that the supporting element can resist a design force effect given by:

$$E_d = \max(\gamma_{Sd}.F_{d,ULS}\ ;\ \gamma_F.F_{d,SLS})$$

where:

$F_{ad,ULS}$ is the design value of the force that the supporting element shall provide to prevent an ultimate limit state of the retaining structure;

$F_{ad,SLS}$ is the design value of the force that the supporting element shall provide to prevent a serviceability limit state of the retaining structure;

γ_{Sd} is a model factor taking into account the stiffness of the retaining wall and the arching effects;

γ_F is used to convert a SLS value to an ULS value (using DC4).

NOTE 175. The value of the model factor is 1.0 unless the National Annex gives another value.

NOTE 176. The value of the partial factor is 1.35 according to DC4 unless the National Annex gives another value.

7.6.7 Structural failure

(1) <REQ> The structural resistance of retaining structures and their component members shall be verified in accordance with:

– EN 1992-1-1 for reinforced or plain concrete retaining walls;
– EN 1993-1-1 and EN 1993-5 for steel retaining walls;
– EN 1994-1-1 for composite steel and concrete retaining walls;
– EN 1995-1-1 for timber members in retaining walls;

– EN 1996-1-1 for masonry retaining walls.

(2) <REQ> Structural resistance shall be verified taking account of all geotechnical failure mechanisms that interfere with the retaining structure.

7.6.8 Partial factors

(1) <REQ> EN 1997-1, 4.4.1, shall apply.

NOTE 177. 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 178. Unless the National Annex gives a specific choice, the approach between RFA and MFA to be used is as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

(2) <RCM> If the resistance factor approach is used, the partial factor γ_{Re} should be applied to the resultant passive earth resistance.

NOTE 179. 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) <PER> When using the resistance factor approach, explicit verification of rotational stability may be omitted for if the uppermost 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[3]				
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	DC4[1]	DC3[1]	DC4[1]
	Ground properties	γ_M	M1[2]	M2[2]	Not factored
	Passive earth resistance	γ_{Re}	Not factored		1,4 (1.1)[3]
Basal heave	See Annex D and Clause 5				

[1]Values of the partial factors for Design Cases (DCs) 3 and 4 are given in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M2 are given in EN 1997-1, Table 4.7.
[3]Values in brackets are given for accidental situations.

7.7 Serviceability limit states

7.7.1 General

(1) <RCM> Where relevant, the assessment of design values of earth pressures should take account of initial stresses in and the stiffness and strength of the ground and the stiffness of the structural elements.

7.7.2 Displacements

(1) <REQ> Limiting values of ground movement around retaining structures shall comply with EN 1997-1, 4.2.5 and 9.3, taking into account the tolerance to displacements of supported structures and utilities within the zone of influence.

(2) <REQ> Ground movement around retaining structures, and their effects on supported structures and services, shall always be checked against comparable experience.

(3) <REQ> Determination of ground movement around retaining structures shall take into account the sequence of work.

(4) <RCM> Vibrations caused by traffic loads or construction machinery close to the retaining wall should be considered when estimating ground movements around retaining structures.

NOTE 180. Guidance on traffic loads is given in EN 1997-1, Annex F.

(5) When linear behaviour is assumed differential ground movements in the zone of influence of the wall are usually under-estimated.

(6) <RCM> When simple linear behaviour is assumed, instead of a complete stress-strain model, the stiffness adopted for the ground and structural materials should be defined according to the extent of deformation and the stress paths expected.

NOTE 181. When linear behaviour is assumed differential ground movements in the zone of influence of the wall are usually under-estimated.

7.8 Implementation of design during execution and service life

7.8.1 General

(1) <REQ> EN 1997-1, 10 shall apply.

(2) <REQ> The execution, and control of concrete gravity walls shall comply with EN 13670.

(3) <REQ> The execution, and control of steel sheet pile walls shall comply with EN 12063.

(4) <REQ> The execution, and control of diaphragm walls shall comply with EN 1538.

(5) <REQ> The execution, and control of pile walls shall comply with EN 1536, EN 14199, or EN 12699 as appropriate.

(6) <REQ> The execution, and control of steel combined walls and high modulus walls shall comply with EN 12063.

<Drafting NOTE: PT6 to check everything in EN 12699 is included in EN 12063 after its completion>

NOTE 182. Feasibility of the construction needs to be ensured without conflicting with execution requirements (e.g. minimum spacing between reinforcement bars in concrete structures).

7.8.2 Inspection

7.8.2.1 General

(1) <REQ> EN 1997-1, 10.3 shall apply.

(2) <RCM> In addition to (1), the Inspection Plan should include:

- 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 wall;
- for gravity walls, 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) <REQ> In addition to 7.8.2.1(1), the Inspection Plan shall include:

- 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) <RCM> In addition to (1), the Inspection Plan should include:

- 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; and
- chemical composition of groundwater; corrosion potential.

7.8.3 Monitoring

(1) <REQ> EN 1997-1, 10.4 shall apply.

(2) <RCM> In addition to (1), the Monitoring Plan should include:

- settlements at established time intervals of adjoining structures or areas, more especially in the case of compressible or poor 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) <REQ> EN 1997-1, 10.5 shall apply.

(2) <RCM> In addition to (1), for permanent retaining structures, the Maintenance Plan should include specifications relative to maintenance of sensitive devices, such as anchors, drains, or pumping wells.

7.9 Testing

(1) <REQ> EN 1997, 11 shall apply.

(2) <RCM> 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) <REQ> EN 1997-1, 12.

8 Anchors

8.1 Scope and field of application

(1) <REQ> 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 (see Figure 8.1).

NOTE 183. This includes anchors within the scope of EN 1537 and mechanical anchors (such as screw, harpoon, and expander anchors) with a free anchor length.

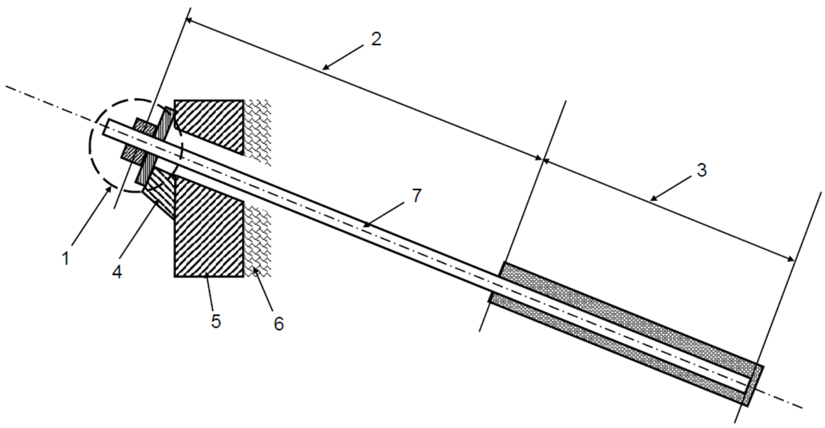


Figure 8.1 – Sketch of an anchor within the scope of Clause 8

1 anchor head, 2 free anchor length, 3 fixed anchored length (e.g. the grout body), 4 load transfer block, 5 anchored structure, 6 soil/rock, 7 tendon

(3) <REQ> Tension elements without a free length shall be designed according to Clause 6 or Clause 10.

NOTE 184. Tension elements without a free length such as piles and micropiles see clause 6

NOTE 185. Tension elements without a free length such as nails and bolts see clause 10.

(4) <REQ> Anchor walls providing fixity for dead-man anchors shall be designed according to Clause 7.

(5) <REQ> All parts and component structures for ground reinforcement shall be designed in accordance with Clause 10

NOTE 186. Parts and component structures to reinforce the ground itself are typically rock bolts, soil nails, sprayed concrete and wire mesh.

8.2 Basis of design

8.2.1 Design situations

(1) <REQ> EN 1997-1, 4.2.2, shall apply.

8.2.2 Geometrical properties

8.2.2.1 General

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

(2) <REQ> The required free anchor length shall be determined in the design of the anchored structure.

NOTE 187. For rock anchors the free anchor length is the extent of the non-resisting formation of the rock mass along the anchor.

(3) <REQ> The anchor head shall be designed to tolerate angular deviations complying with EN 1537.

(4) <REQ>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.

(5) <REQ> The anchor head shall be designed to accommodate deformations and load variation that can occur during the design service life of the structure.

(6) <REQ> Measures shall be taken to avoid adverse interactions between anchors that are located close to each other.

NOTE 188. Details are given in Annex E.

(7) <RCM> The resisting ground should be sufficiently distant from the anchored structure to avoid any interaction between the two.

(8) <RCM> The orientation of the anchor should be chosen to enable self-stressing under deformation.

(9) <REQ> If self-stressing under deformation is not possible, the adverse effects of potential failure mechanisms shall be considered.

(10) <RCM> The orientation of the anchor should be chosen to optimize its pre-stressing capacity into the resisting ground.

8.2.2.2 Zone of influence

(1) <REQ> EN 1997-1, 4.2.1.1 shall apply.

8.2.3 Actions and environmental influences

8.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

8.2.3.2 Permanent and variable actions

(1) <REQ> Design values of the anchor force and lock off load shall be obtained from the verification of limit states for the anchored structure.

(2) <REQ> Anchor forces required to support natural slopes, cuttings, and embankments shall comply with Clause 4.

(3) <REQ> Anchor forces required to support retaining structures shall comply with Clause 7.

(4) <REQ> Anchor forces required to support structures subjected to uplift shall be determined according to EN 1997-1, 8.1.3.

(5) <REQ> The lock-off load shall be sufficient to ensure serviceability of both anchored and supported structures.

(6) <REQ> The lock-off load shall not give rise to a limit state in the ground, in the anchored or in the supported structures.

(7) <REQ> The lock-off load shall be large enough to ensure that the anchor resistance can be mobilised without exceeding the serviceability limit state criteria of the anchored structure.

8.2.3.3 Cyclic and Dynamic actions

(1) <REQ> EN 1997-1 4.3.1.3 shall apply

8.2.3.4 Environmental influences

(1) <REQ> EN 1997-1 4.3.1.4 shall apply

(2) <REQ> The design to achieve an adequate durability shall account for the potential adversely affect of chemical components of ground or groundwater according to EN 1537.

8.2.4 Limit states

8.2.4.1 Ultimate Limit States

(1) <REQ In addition to EN 1997-1, 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) <RCM> Ultimate limit states other than those given in (1) should be verified as necessary.

(3) <REQ> For a group of anchors, verification shall be based on the most critical failure surface.

NOTE 189. 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) <REQ> In addition to EN 1997-1, 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) <RCM> Serviceability limit states other than those given in (4) should be verified as necessary.

8.2.5 Robustness

(49) <REQ> EN 1997-1 Clause 4.1.4 shall apply

8.2.6 Ground investigation

8.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply.

(2) <RCM> The zone of ground into which tensile forces are transferred should be included in ground investigations.

(3) <RCM> The ground investigation should determine the potential influence of difficulties caused as example, but not limited to, by:

– potential obstructions to drilling;
– the process of borehole drilling (drillability);
– abrasivity;
– anchor borehole stability;
– flow of ground water in or out of the borehole;
– geometrical properties of discontinuities and weakness zones in rock;
– resistance capacity or lack of it of the resisting ground;
– adhesion at interface surfaces;
– borehole axis deviations; and
– loss of grout from the borehole.

8.2.6.2 Minimum extent of investigation

(1) <RCM> The extent of the in situ testing 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 weakness zones in the zone of influence of the anchors are well defined.

8.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

8.3 Materials

8.3.1 Ground Properties

(1) EN 1997-2, 7-12 shall apply.

8.3.2 Steel

(1) <REQ> EN 1997-1, 5.6 shall apply.

8.3.3 Grout

(1) <REQ> EN 1997-1, 5.4 shall apply

8.3.4 Other materials

(1) <REQ> 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) <REQ> Clause EN 1997-1, 6, shall apply.

8.5 Geotechnical analysis

(1) <REQ> Clause EN 1997-1, 7, shall apply.

(2) <REQ> In addition to (1) the geotechnical analysis shall address all limit state verifications listed in 8.2.4.

8.6 Ultimate limit states

8.6.1 General

(1) <REQ> The design value of the ultimate limit state resistance of an anchor shall satisfy Formulae (8.1) and (8.2).

$$E_{\mathrm{d,ULS}} \leq \min\left(R_{\mathrm{ad,ULS}}; R_{\mathrm{td}}\right) \tag{8.1}$$

where

$$E_{\mathrm{d,ULS}} = \max\left(F_{\mathrm{ad,ULS}}; \gamma_{\mathrm{F,ULS}} \cdot F_{\mathrm{ad,SLS}}\right) \tag{8.2}$$

and 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.

$F_{\mathrm{ad,ULS}}$ is the design value of the maximum anchor force, sufficient to prevent the ultimate limit state in the anchored structure

$F_{\mathrm{ad,SLS}}$ is the design value of the maximum anchor force, sufficient to prevent the serviceability limit state in the anchored structure

$\gamma_{\mathrm{F,ULS}}$ is a partial factor on the anchor force at the ultimate limit state (see NOTE 4)

(2) <REQ> $E_{d,ULS}$ shall be evaluated according to Clauses 4 and 7 and EN 1997-1, 8.1.3.

(3) <REQ> $E_{d,ULS}$ shall include the effect of anchor lock-off load.

NOTE 190. The value of $F_{ad,ULS}$ may include a model factor according to clauses 4, 5, 7 and 9.

NOTE 191. The value of the partial factor $\gamma_{F,ULS}$ is 1.35 unless the National Annex gives a different value

8.6.2 Geotechnical resistance

(1) <REQ> Anchors shall only be used if their geotechnical design and construction have been verified by:

– investigation or suitability tests; or
– comparable experience.

NOTE 192. Anchors are verified by investigation and suitability tests unless the National Annex states otherwise.

NOTE 193. Comparable experience is defined in EN 1997-1, 3.1.2.3

(2) <REQ> Acceptance tests shall be carried out on all anchors.

(3) <PER> When the geotechnical structure with its anchor is sufficiently robust, it may be accepted to not carry out acceptance tests on all anchors.

NOTE 194. Note: this is typically applicable for rock in the absence of creep.

(4) <REQ> Investigation, suitability and acceptance tests on grouted anchors shall comply with EN ISO 22477-5.

(5) <REQ> 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 as specified in EN ISO 22477-5;
– two investigation tests and three suitability tests, when using Test Method 3 as specified in EN ISO 22477-5.

(6) <REQ> For non-grouted anchor types, the minimum number of tests shall comply with (1) unless otherwise specified by the relevant authority or, where not specified, be agreed for a specific project by the relevant parties.

(7) <REQ> The measured value of the geotechnical resistance of a grouted anchor at the ultimate limit state ($R_{am,ULS}$) shall be obtained from the results of anchor test using Formula (8.3):

$$R_{am,ULS} = \min(R_{am}(\alpha_{ULS}), P_P) \quad (8.3)$$

where:

$R_{am}(\alpha_{ULS})$ 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.

NOTE 195. <REQ> For grouted anchors, the ultimate limit state criterion α_{ULS} in (3) shall be the creep rate:

– α_1 for Test Method 1;
– α_3 for Test Method 3.

NOTE 196. The values of α_1 and α_3 are given in Table 8.3 (NDP), unless the National Annex gives different values.

NOTE 197. The load relating to the physical pull-out resistance can be higher than the value of the load corresponding to aforementioned creep rates.

(8) <REQ> The measured value of the geotechnical resistance of a non-grouted anchor at the ultimate limit state ($R_{\text{am,ULS}}$) shall be obtained from the results of anchor test using Formula (8.4):

$$R_{\text{am,ULS}} = \min\left(R_{am}\left(C_{\text{ad,ULS}}\right); P_{\text{P}}\right) \quad (8.4)$$

where:

$R_{\text{am,}}(C_{\text{ad,ULS}})$ is the measured value of the anchor's geotechnical resistance complying with the ultimate limit state criterion, $C_{\text{ad,ULS}}$;

P_{P} is the proof load.

(2) <RCM> For non-grouted anchors, $C_{\text{ad,ULS}}$ should be specified by the relevant authority or, where not specified, be agreed for a specific project by the relevant parties.

NOTE 198. For non-grouted anchors, $C_{\text{ad,ULS}}$ can be given in the National Annex.

(3) <REQ> If the ultimate limit state criterion is not reached during a test, P_{p} shall be taken as $R_{\text{am,ULS}}$.

(4) <REQ> The characteristic value of an anchor's geotechnical resistance at the ultimate limit state $R_{\text{ak,ULS}}$ shall be determined from Formula (8.5):

$$R_{\text{ak,ULS}} = \frac{\left(R_{\text{am,ULS}}\right)_{\text{min}}}{\xi_{\text{ULS}}} \quad (8.5)$$

where:

$(R_{\text{am,ULS}})_{\text{min}}$ is the minimum value of $R_{\text{am,ULS}}$ measured in a number of tests;

ξ_{ULS} is a correlation factor taking into account the number of tests.

NOTE 199. The value of ξ_{ULS} is 1.0 unless the National Annex gives a different value.

(5) <REQ> The design value of an anchor's geotechnical ultimate limit state resistance $R_{\text{ad,ULS}}$ shall be determined from Formula (8.6):

$$R_{\text{ad,ULS}} = \frac{R_{\text{ak,ULS}}}{\gamma_{\text{a,ULS}}} \quad (8.6)$$

where:

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

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

NOTE 200. The value of $\gamma_{a,ULS}$ is given in Table 8.1 (NDP), unless the National Annex gives a different value.

8.6.3 Structural resistance

(1) <REQ> 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 (8.7):

$$E_{d,ULS} \leq R_{td} \tag{8.7}$$

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) <REQ> The structural design of steel tendons under a proof load shall comply with EN ISO 22477-5.

8.6.4 Partial factors

(1) <RCM> The geotechnical resistance of an anchor at the ultimate limit state should be verified using the resistance factor approach (RFA), with factors γ_R applied to ground resistance, using Formula (8.20) of EN 1990-1.

NOTE 201. The value of γ_R is given in Table 8.1 (NDP) unless the National Annex gives a different value.

Table 8.1 (NDP) – Partial factors for the verification of geotechnical resistance of anchors for fundamental (persistent and transient) 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	$\gamma_{a,ULS}$	1,1[a,b]	1,1 [a]
[a]See Formula (8.6) [b]See Formulae (8.14) and (8.16)				

8.7 Serviceability limit states

8.7.1 General

(1) <RCM> 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 202. In Test Method 1, the serviceability limit state of a grouted anchor is implicitly verified by verification of the ultimate limit state.

(2) <REQ> If Test Method 3 is used, the anchor's design resistance ($R_{\mathrm{ad,SLS}}$) shall comply with Formula (8.9):

$$E_{d,\mathrm{SLS}} \leq R_{\mathrm{ad,SLS}} \tag{8.8}$$

where:

$E_{\mathrm{d,SLS}}$ 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;

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

8.7.2 Geotechnical resistance

(1) <REQ> If Test Method 3 is used, the measured serviceability limit state resistance $R_{\mathrm{am,SLS}}$ of an anchor shall be determined from a minimum of two investigation tests in each geotechnical unit.

(2) <REQ> The measured geotechnical resistance of a grouted anchor at the serviceability limit state ($R_{\mathrm{am,SLS}}$) shall be determined from Formula (8.9):

$$R_{\mathrm{am,SLS}} = \min(R_{am}(\alpha_{\mathrm{SLS}}); P_{\mathrm{C}}; P_{\mathrm{P}}) \tag{8.9}$$

where:

$R_{am}(\alpha_{\mathrm{SLS}})$ is the measured value of the anchor's geotechnical resistance complying with the serviceability limit state criterion α_{SLS};

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;

NOTE 203. The serviceability criterion α_{SLS} is given in Table 8.3 (NDP) unless the National Annex gives a different criterion.

(3) <REQ> The measured geotechnical resistance of a non-grouted anchor at the serviceability limit state ($R_{\mathrm{am,SLS}}$) shall be determined from Formula (8.10):

$$R_{\mathrm{am,SLS}} = \min\left(R_{am}(C_{\mathrm{ad,ULS}}); P_{\mathrm{C}}; P_{\mathrm{P}}\right) \tag{8.10}$$

where:

$R_{am}(C_{\mathrm{ad,SLS}})$ is the measured value of the anchor's geotechnical resistance at complying with the serviceability limit state criterion $C_{\mathrm{ad,SLS}}$;

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) <RCM> For non-grouted anchors, $C_{\mathrm{ad,SLS}}$ should be specified by the relevant authority or, where not specified, be agreed for a specific project by the relevant parties.

NOTE 204. For non-grouted anchors, $C_{\mathrm{ad,SLS}}$ can be given in the National Annex.

(5) <REQ> The characteristic value of the geotechnical resistance of an anchor at the serviceability limit state ($R_{ak,SLS}$) shall be determined from Formula (8.11):

$$R_{\text{ak,SLS}} = \left(R_{\text{am,SLS}}\right)_{\text{min}} \tag{8.11}$$

where:

$(R_{am,SLS})_{min}$ is the minimum value of $R_{am,SLS}$ measured in a number of tests.

(6) <REQ> The design value of the geotechnical resistance of an anchor at the serviceability limit state ($R_{ad,SLS}$) shall be determined from Formula (8.12):

$$R_{\text{ad,SLS}} = \frac{R_{\text{ak,SLS}}}{\gamma_{\text{a,SLS}}} \tag{8.12}$$

where:

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

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

NOTE 205. The value of $\gamma_{a,SLS}$ is given in **Table 8.2** (NDP) unless the National Annex gives a different value.

8.7.3 Partial factors

(1) <RCM> The geotechnical resistance of an anchor at the serviceability limit state should be verified using the resistance factor approach (RFA), with factors γ_R applied to ground resistance, using Formula (8.20) of EN 1990.

NOTE 206. The value of $\gamma_{a,SLS}$ is given in **Table 8.2** (NDP) unless the National Annex gives a different value.

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	$\gamma_{a,SLS}$	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	$\gamma_{a,SLS,test}$		1.25[b]
	Resistance of a temporary anchor at the serviceability limit state			1.15[b]
[a]See Formula (8.122) [b]See Formulae (8.14) and (8.16)				

8.8 Implementation of design during execution and service life

8.8.1 General

(1) <REQ>EN 1997-1, 10 shall apply.

(2) <REQ> Execution of grouted anchors shall comply with EN 1537.

(3) <REQ> Execution of non-grouted anchors shall be as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties. The specifications shall be given in the Geotechnical Design Report and in the execution specification.

(4) <REQ> Prior to their usage, it shall 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) <REQ> EN 1997-1, 10.2

(2) <RCM> In addition to (1) supervision of the installation and testing of anchors should comply with EN 1537.

8.8.3 Inspection

(1) <REQ> EN 1997-1, 10.3 shall apply.

(2) <RCM> In addition to (1) inspection of the installation and testing of anchors should comply with EN 1537.

8.8.4 Monitoring

(1) <REQ> EN 1997-1, 10.4 shall apply.

(2) <RCM> Monitoring of anchors should comply with EN 1537.

8.8.5 Maintenance

(1) <REQ> EN 1997-1, 10.5 shall apply.

8.9 Testing

(1) <REQ> In addition to EN 1997-1, 11, the following shall apply

(9) <REQ> Testing of grouted anchors shall comply with one of the test methods given in EN ISO 22477-5.

NOTE 207. Unless the National Annex gives a specific choice, the Test Method to be used is as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE 208. Limiting values for creep in investigation, suitability and acceptance tests are given in Table 8.3 (NDP).

Table 8.3 (NDP) – Limiting criteria for investigation, suitability and acceptance tests at the ultimate and serviceability states

Test method	Parameter[1]	Anchor type	Investigation test	Suitability test		Acceptance test	
				α_{ULS}	α_{SLS}	α_{ULS}	α_{SLS}
			α_{ULS}	**(8.13)**	**(8.14)**	**(8.15)**	**(8.16)**
1	α_1	All	2 mm	2 mm	Not used	2 mm	Not used
3	α_3	Temporary	5 mm	Not used	1,2 mm	Not used	2,5 mm
		Permanent			1,0 mm		1,5 mm
[1]Creep rate per log cycle of time							

(50)

(10) <RCM> Testing of non-grouted anchors should be carried out in accordance with EN 22477-5, unless specified otherwise by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

8.9.1 Grout

(1) <REQ> The compressive strength of grout used for load transfer shall be verified by testing prior to the use of grout for anchor installation.

(2) <REQ> The testing of compressive strength of grout used for load transfer shall be conducted by two series of tests for every 20 m^3 of mixed grout.

(3) <REQ> Each series of tests shall comprise 3 samples.

8.9.2 Investigation tests

(1) <RCM> The proof load in investigation tests should be estimated from the expected geotechnical resistance of the anchor at the ultimate limit state.

NOTE 209. Limit values for creep at the proof load in investigation tests are given in Table 8.3 (NDP) unless the National Annex gives a different value.

(2) <RCM> 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) <REQ> Anchors for investigation tests shall comply with EN ISO 22477-5.

8.9.3 Suitability tests

(1) <REQ> Suitability tests shall be used to verify that specified criteria are not exceeded at a proof load, P_P, determined from Formula (8.13) for Test Method 1 or (8.14) for Test Method 3:

$$P_P \geq \xi_{a,ULS,test} \cdot \gamma_{a,ULS} \cdot E_{d,ULS} \tag{8.13}$$

$$P_P \geq \xi_{a,SLS,test} \cdot \gamma_{a,SLS,test} \cdot E_{d,SLS} \tag{8.14}$$

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;

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

$\gamma_{a,SLS,test}$ is a partial factor on the anchor's geotechnical resistance in suitability and acceptance tests at the serviceability limit state;

$\xi_{a,ULS,test}$, $\xi_{a,SLS,test}$ are correlation factors, taking account of the number of suitability tests.

NOTE 210. The values of $\xi_{a,ULS,test}$ and $\xi_{a,SLS,test}$ are 1,0 unless the National Annex gives different values.

NOTE 211. The values of $\gamma_{a,ULS}$ and $\gamma_{a,SLS,test}$ are given in Table 8.1 (NDP) and Table 8.2 (NDP), respectively, unless the National Annex gives different values.

NOTE 212. Limit values for creep in suitability tests are given in Table 8.3 (NDP) unless the National Annex gives different values.

(4) <RCM> Grouted anchors with tendon bond lengths spaced at 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.

(5) <REQ> Anchors for suitability tests shall comply with EN ISO 22477-5.

(6) <REQ> The apparent tendon free length of a grouted anchor shall comply with EN 1537.

8.9.4 Acceptance tests

(1) <REQ> Acceptance tests on all anchors shall be carried out prior to their lock off and before they become operational.

(2) <REQ> Acceptance tests shall be used to verify that specified limiting criteria are not exceeded at the proof load, P_P, given by Formulae (8.15) for Test Method 1 or (8.16) for Test Method 3:

$$P_P = \gamma_{a,ULS} \cdot E_{d,ULS} \tag{8.15}$$

$$P_P = \gamma_{a,SLS,test} \cdot E_{d,SLS} \tag{8.16}$$

where:

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

$\gamma_{a,SLS,test}$ is a partial factor on the anchor's geotechnical resistance in suitability and acceptance tests at the serviceability limit state.

NOTE 213. The values of $\gamma_{a,ULS}$ and $\gamma_{a,SLS,test}$ are given in Table 8.1 (NDP) and Table 8.2 (NDP), respectively, unless the National Annex gives different values.

NOTE 214. Limit values for creep in acceptance tests are given in Table 8.3 (NDP) unless the National Annex gives different values.

(51) <REQ> The apparent tendon free length of a grouted anchor shall comply with EN 1537.

(52) <RCM> 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) <REQ> Reporting for anchors shall comply with EN 1997-1, 12.

(2) <REQ> In addition to (1), reporting for grouted anchors shall comply with EN 1537 and EN ISO 22477-5.

(3) <REQ> In addition to (1), reporting for non-grouted anchors shall 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

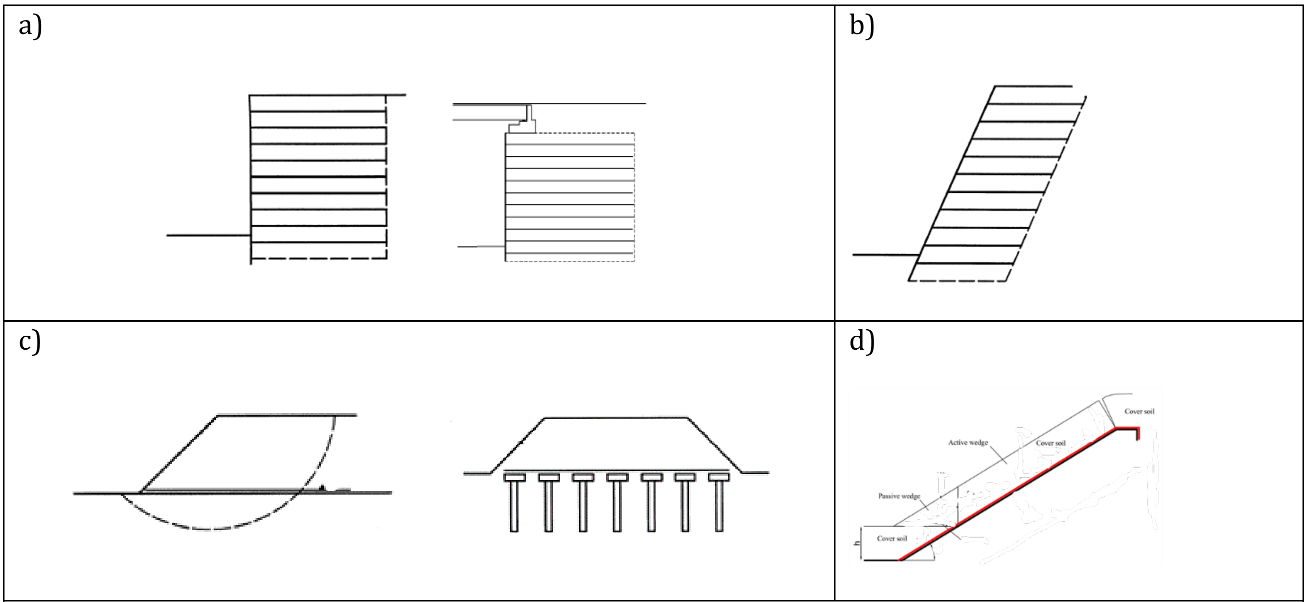
9.1 Scope and field of application

(1) <REQ> This Clause shall apply to reinforced fill structures.

NOTE 215. Reinforced fill structures include those in Figure 9.1.

NOTE 216. Earthwork structures without reinforcement are covered by clause 4 embankments.

NOTE 217. Design of reinforced road pavements, are not coverd by this standard



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
<Drafting note: The figures for this draft has been borrowed from EN 14490>

Figure 9.1 – Reinforced fill structures covered by this standard

9.2 Basis of design

9.2.1 Design situations

(2) <REQ> EN 1997-1, 4.2.2, shall apply.

9.2.2 Geometrical properties

9.2.2.1 General

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

9.2.2.2 Reinforcing elements

(1) <REQ> Verification of limit states shall include determination of allowable construction tolerances of reinforcing elements locations.

(2) <RCM> If facing element are used, the vertical spacing of reinforcing elements should be compatible with the height of the facing elements.

9.2.2.3 Zone of influence

(1) <REQ> EN 1997-1, 4.1.2.1 shall apply.

9.2.3 Actions and environmental influences

9.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

9.2.3.2 Permanent and variable actions

(1) <REQ> Design value of the force in the reinforcement elements shall be obtained from verification of limit states for the reinforced sturcture.

(2) <REQ> Design reinforcement element force required to prevent failure by overall stability shall comply with Clause 4.

(3) <REQ> Design reinforcement elements force required to prevent failure by loss of bearing capacity shall comply with Clause 5.

(4) <REQ> Design reinforcement elements force required to prevent failure by sliding shall comply with Clause 5.

(5) <REQ> Design reinforcement elements force required to prevent failure by loss of equilibrium shall comply with Clause 7.

(6) <REQ> Traffic load shall be included in verfications of reinforced fill structures, as appropriate.

NOTE 218. Guidance on traffic load is given in EN 1997-1, Annex F.

(7) <REQ> 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.

9.2.3.3 Cyclic and dynamic actions

(11) <REQ> EN 1997-1, 4.3.1.3 shall apply.

9.2.3.4 Environmental influcences

(1) <REQ> EN 1997-1, 4.3.1.5 shall apply.

(2) <REQ> 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) <PER> The value of T_{eq} may be specified by the relevant authority or, where not specified, agreed for a specific project by appropriate parties.

(4) <RCM> 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.

(5) <RCM> Measures should be taken to avoid adverse swelling or expansion of frost susceptible soils in the ground near the surface of reinforced structures.

NOTE 219. Possible measures include selection of suitable backfill material, drainage, or insulation.

(6) <REQ> 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

9.2.4 Limit states

9.2.4.1 Ultimate Limit State

(1) <REQ> In addition to EN 1997-1, 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 from the ground 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.

(2) <RCM> Ultimate limit states other than those given in (1) should be verified as necessary.

9.2.4.2 Servicability Limit State

(1) <REQ> In addition to EN 1997-1, 9, the following serviceability limit states shall be verified for all reinforced structures:

- deformations of the reinforced structure itself;
- differential settlement along the facing due to subsoil deformation;
- differential movement between facing and reinforced structure;
- deformation of the reinforced structure, which may 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 (differential settlement or movement).

NOTE 220. Deformations of the reinforced structure can be caused/influenced by the elongation or post-construction elongation of the reinforcement elements themselves

(2) <RCM> Serviceability limit states other than those given in (1) should be verified as necessary.

9.2.5 Robustness

(1) <REQ> EN 1997-1, 4.1.4 shall apply.

9.2.6 Ground investigation

9.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply.

(2) <RCM> The relevant chemical properties of ground and groundwater should be determined for durability assessment of any reinforcing elements and facing elements.

9.2.6.2 Minimum extent of in-situ testing

(1) <REQ> The depth and horizontal extent of the in-situ testing shall be sufficient to determine the ground conditions within the zone of influence.

(2) <REQ> The depth of the in-situ testing for application of reinforced fill as wall and abutments shall comply with 7.2.6.2.

(3) <REQ> The depth of the in-situ testing for application of reinforced fill as reinforced slope, basal reinforcement and reinforced embankments shall comply with 4.2.6.2.

9.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

9.3 Materials

9.3.1 Ground properties

(1) <REQ> EN 1997-2, 7 to12 shall apply

NOTE 221. A classification of fill is given in EN 16907-2.

9.3.2 General related to durability

(1) <REQ> EN 1997, 4.1.6 shall apply.

(2) <REQ> Determination of the loss of strength of reinforcing elements for fills shall take account of the long-term effects of sustained load in reinforcement (creep) and potential damage of the reinforcement during installation.

9.3.3 Geosynthetics

(1) <REQ> In addtion to EN 1997-1, 5.2, geosynthetic reinforcing elements shall comply with EN 13251.

(2) <REQ> The characteristic tensile strength of geosynthetic reinforcement, T_k shall be determined in accordance with EN ISO 10319.

(3) <REQ> 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 $T_{k,cr}$ shall be determined from isochronous creep curves.

NOTE 222. Relative elongation between given times can be related to post construction elongation or specified design service life in voids overbridging application.

(4) <REQ> In addition to 9.3.2.1 (1), a reduction factor η_{gs} shall be applied to the tensile strength of geosynthetic reinforcing elements to account for the loss of strength.

(5) <REQ> The value of ηgs shall be determined from Formula (9.1):

$$\eta_{\mathrm{gs}} = \eta_{\mathrm{cr}} \cdot \eta_{\mathrm{dmg}} \cdot \eta_{\mathrm{w}} \cdot \eta_{\mathrm{ch}} \cdot \eta_{\mathrm{dyn}} \tag{9.1}$$

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 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).

NOTE 223. 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 224. The value of η_{dyn} is the reciprocal of the reduction factor specified in EBGEO as A_5.

NOTE 225. 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 226. η_{cr} include creep strain based on isochrouous curves, to allow for creep and limiting elongation.

NOTE 227. The factor η_{w} can have a value greater than 1.0 if the reinforcement is not covered by soil within one day of installation.

(6) <REQ> The representative tensile resistance Rt,rep,el of a geosynthetic reinforcing element shall be determined from Formula (9.2):

$$R_{\mathrm{t,rep,el}} = \eta_{\mathrm{gs}} T_{\mathrm{k}} \tag{9.2}$$

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.

NOTE 228. Tensile strength determined with (3) is related to SLS.

9.3.4 Steel

(1) <REQ> Reinforcement in the form of strips, bars, or rods shall comply with EN 10025-2, EN 10025-4, or EN10080, as appropriate for the type of steel used.

(2) <REQ> Reinforcement in the form of welded wire ladders or meshes shall comply with EN 10218-2, EN 10223-8, or EN 10080, as appropriate for the type of steel used.

(3) <REQ> Metallic reinforcement shall have a total extension at the maximum load A_{gt} defined in EN ISO 6892-1 of at least 5%.

(4) <REQ> If a steel reinforcing element is galvanised, the hot dip galvanized coating to steel strips, rods, bars, ladders, and welded wire meshes shall comply with EN ISO 1461.

(53) <REQ> Where steel welded wire meshes are 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.

(54) <REQ> Stainless steel and aluminium alloys shall only be used for reinforcement if they comply with a relevant standard specified by the relevant authority or, where not specified, agreed for a specific project by appropriate parties.

(55) <REQ> The representative tensile resistance ($R_{t,rep,el}$) for steel reinforcement to use in reinforced fill structures shall be designed in agreement with one of the following apporaches:

- according to EN 1993-1-1;
- according to Forumla (9.4).

NOTE 229. Unless the National Annex gives a specific choice, the approach to be used is as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

$$R_{t,rep,el} = min\left(A_{ry}f_{yk}; A_{ru}f_{uk}\right) \qquad (9.3)$$

where:

f_{yk}	is the characteristic yield strength of the steel;
$R_{t,rep,el}$	representative tensile resistance;
f_{uk}	is the characteristic ultimate strength of the steel;
A_{ry} and A_{ru}	are the reduced cross-sectional areas of the reinforcing element at yield and ultimate resistance, respectively, allowing for the effects of potential corrosion.

(56) <REQ> The reduced cross-sectional area according shall be determined according to (9), (10), (11), (12) and (13), as appropriate.

(57) <REQ> In addition to to 9.3.2 (1), the cross-sectional area of steel reinforcing elements shall be reduced by an amount based on the potential loss of thickness Δe caused by corrosion in the ground.

(58) <REQ> The value of Δe shall be determined from Formula (9.4):

$$\Delta e = max(AT^n - e_z; 0) \qquad (9.4)$$

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 (factor covering reduction in corrosion rate in time);

e_z is the initial zinc thickness.

NOTE 230. Values for A and n are given in Table 9.1 (NDP) for steel in surrounding soils that comply with the electro-chemical properties of Table B.1 of EN 14475.

NOTE 231. For other types of corrosion protection, studies in the specific ground conditions are required to define the thickness reduction values

Table 9.1 (NDP) – Corrosion parameters for fill steel reinforcement

Steel	***A* (μm)**		***n***	
	Land-based	**Fresh water**	**Land-based[1]**	**Fresh water[2]**
Galvanized	25	40	0.65	0.60
Non-galvanized			0.80	0.75
[1] Land-based = without influence of groundwater or surface water [2] Fresh water = installed fresh water or regularly submerged [EN 14490]				

(59) <REQ> For steel strip reinforcing elements, the reduced cross-sectional areas A_{ry} and A_{ru} in Formula (9.7) shall be determined from Formulae (9.8) and (9.9) respectively:

$$A_{\mathrm{ry}} = A_0 - 2bK_y\Delta\mathrm{e} \tag{9.5}$$

$$A_{\mathrm{ru}} = A_0 - 2bK_u\Delta\mathrm{e} \tag{9.6}$$

where:

A_0 is the initial cross-sectional area of the steel reinforcement;

b is the width of the strip element;

K_y, K_u are corrosion heterogeneity factors, for yield and ultimate respectively;

Δe is the loss of steel thickness at one face at the considered point in time along the design service life of the structure (see formula 9.4).

NOTE 232. The values of Ky and Ku are given in Table 9.2 (NDP), unless the National Annex give different values.

Table 9.2 –(NDP) Corrosion parameters for fill steel reinforcement

Steel	**Strip thickness[a] (mm)**	**Bar diameter (mm)**	K_u	K_y
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				

(60) <RCM> When b < 8 times the thickness of the strip, a reduction in width should be assumed.

(61) <REQ> For rods, bars, ladders, and welded wire meshes, the reduced cross-sectional areas $A_{r,y}$ and $A_{r,u}$ in in Formula (9.3) shall be determined from Formulae (9.7) and (9.8):

$$A_{\mathrm{r,y}} = A_0 - \pi(\phi_0 - K_y \Delta e)\, K_y \Delta e \tag{9.7}$$

$$A_{\mathrm{r,u}} = A_0 - \pi(\phi_0 - K_u \Delta e)\, K_u \Delta e \tag{9.8}$$

where, in addition to the symbols defined for Formulae (9.8) and (9.9):

ϕ_0 is the initial diameter of the reinforcement.

9.3.5 Polymeric coated steel woven wire meshes

(1) <REQ> Reinforcement in the form of polymer coated woven wire mesh shall comply with EN 10218 2, in case of steel wire only and EN 10223-3 for the whole reinforcement product.

(2) <REQ> 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 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) <REQ> The characteristic tensile strength of polymeric coated steel woven wire mesh reinforcement shall be determined in accordance with EN ISO 10319.

(4) <REQ> 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.

(5) <REQ> The reduction factor η_{pwm} shall be determined from Formula (9.9):

$$\eta_{\mathrm{pwm}} = \eta_{\mathrm{dmg}} \cdot \eta_{\mathrm{cor}} \tag{9.9}$$

where:

η_{dmg} is a reduction factor accounting for the adverse effects of mechanical damage during 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 of the polymeric coating by chemical degradation and/or the loss of the Zinc or Zinc/Aluminium layer by corrosion, where applicable.

NOTE 233. The value of η_{cor} is determined by testing standard to be developed.

NOTE 234. The values of η_{dmg} is the reciprocal of the reduction factor specified in ISO TR 20432, as RF_{ID}.

NOTE 235. 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 taken into account in ηcor as it will induce corrosion of the exposed wire.

NOTE 236. If the polymeric coating sustains its watertightness with no chemical nor mechanical damage, no corrosion of the steel wire and its metallic coating can occur.

(6) <REQ> The evaluation of η_{dmg} shall account for the decrease of tensile strength at short term due to damage during execution.

(7) <REQ> 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 237. The polymeric and a zinc-aluminium alloy coatings have no structural function, since theirs only purpose is to protect the metallic wires.

(8) <REQ> The representative tensile resistance $R_{t,rep,el}$ of a polymeric coated woven wire mesh reinforcing element shall be determined from Formula (9.10):

$$R_{\mathrm{t,rep,el}} = \eta_{\mathrm{pwm}} T_{\mathrm{k}} \tag{9.10}$$

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 (Formula 9.9).

9.3.6 Other materials

(1) <REQ> Materials other than those specified in 9.3.3, 9.3.4, and 9.3.5 shall 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) <REQ> EN 1997-1, 6 shall apply.

9.4.2 Drainage systems

(1) <REQ> Clause 12 shall apply.

(2) <REQ> If a drainage 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) <REQ>EN 1997-1, 7 shall apply

(2) <REQ> The external and compound stability of a reinforced fill structure, shall be analysed according to Clauses 4, 5, or 7, as appropriate, with the beneficial effect of reinforcing elements included.

(3) <REQ> The internal stability of a reinforced fill structure shall be analysed according to the type of reinforced fill structure (9.5.2).

(4) <REQ> Horizontal and vertical deformations of a reinforced fill structure shall be analysed according to Clauses 4, 5, or 7, as appropriate.

(5) <REQ> The execution specification shall state requriement on properties of the fill needed to fulfil the verificiation of the limit states.

(6) <PER> 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.

(7) <REQ> Verification of the compound stability of a reinforced fill structure shall include the potential beneficial effect of any reinforcing elements.

9.5.2 Internal stability

9.5.2.1 Reinforced slopes, walls, and bridge abutments

(1) <RCM> 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 238. Details of som of these methods are given in Annex F.3.

9.5.2.2 Basal reinforcement for embankments

(1) <REQ> When analysing potential excessive deformation on embankment edges, resistance to extrusion shall be verified.

(2) <REQ> Resistance to horizontal sliding over the basal reinforcement shall be verified.

NOTE 239. Details of these checks are given in Annex F.4.

(3) <REQ> Temporary roads and working platforms over low strength fine soil with basal reinforcement shall be analysed as low height embankments.

(4) <REQ> 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 rigid inclusions

(1) <PER> Load transfer platforms may be used over discrete inclusions to allow bigger spacing and limit differential deformation on embankment surface.

(2) <REQ> Rigid inclusions shall be designed according to 11.

NOTE 240. The rigid inclusion is according (BII)

(3) <REQ> When analysing embankment edges outside the inclusion zone, analyses according to 9.5.2.2 shall be performed.

(4) <RCM> 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 241. Details of these methods are given in Annex F.5.

(5) <PER> 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) <PER> Overbridging systems that include reinforcing elements may be used over areas prone to subsidence to limit differential deformation on surface.

(2) <REQ> If the design of overbridging systems is for a specified short-term design service life, the system shall comprise a monitoring system, which shall indicate the location of void creation in order to backfill the void within the specified short-term design period.

(3) <REQ> 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 specified limits.

(4) <RCM> 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 242. Details of these methods are given in Annex F.6.

(5) <PER> 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) <REQ> 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.

(2) <REQ> The loads shall be determined using the plane of least frictional resistance in the veneer cover package.

(3) <REQ> The stability of the veneer layer subject to traffic load shall be verified for a transient design situation.

(4) <REQ> The stability of the anchorage at the top of the veneer, and any intermediate anchorages down the slope, shall be verified.

(5) <REQ> The stability of the veneer shall be verified considering the formation of a water table inside the veneer soil.

NOTE 243. Further details are given in Annex F.7.

9.5.3 Resistance of reinforcing elements

9.5.3.1 General

(1) <REQ> The representative tensile resistance ($R_{\mathrm{t,rep}}$) of a reinforcing element shall be determined from Formula (9.11):

$$R_{\mathrm{t,rep}} = \min\left(R_{\mathrm{t,rep,el}}; R_{\mathrm{rep,po}}; R_{\mathrm{rep,con}}\right) \tag{9.11}$$

where:

$R_{\mathrm{t,rep,el}}$ is the representative tensile resistance strength of the reinforcing element;

$R_{\mathrm{rep,po}}$ is the representative value of the pull-out resistance mobilised along the interface between the ground and the reinforcing element;

$R_{\mathrm{rep,con}}$ is the representative value of the resistance at the connection between the facing and the reinforcing element.

(2) <REQ> 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) <REQ> Any shear resistance that is assumed in the calculation shall be limited to punching shear capacity of the surrounding ground.

(4) <REQ> 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.

9.5.4 Pull-out resistance

9.5.4.1 General

(1) <REQ> The resistance of a reinforcing element to pull-out from the fill shall be verified both from the embedded end of the element to the face of the structure and the face of the structure to the embedded end of the element.

(2) <REQ> The representative pull-out resistance ($R_{\mathrm{rep,po}}$) of a reinforcing element shall be determined from Formula (9.12):

$$R_{\mathrm{rep,po}} = P \int_0^{L_{\mathrm{po}}} \tau_{\mathrm{po}}(x) \cdot dx \qquad (9.12)$$

where:

P is the length of the perimeter of the reinforcing element;

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 244. Pull-out resistance can be influenced by dynamic action.

NOTE 245. Figure 9.2 gives an example of pull-out analysis at the embedded end of reinforcing elements.

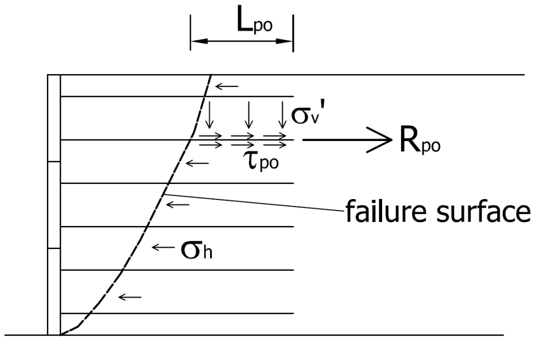


Figure 9.2 – Example of pull-out analysis at the embedded end of reinforcing elements

(3) <REQ> The pull-out resistance shall be based on documented tests in comparable situations or from project-specific tests.

(4) <RCM> The pull-out resistance from the face of the structure should be increased by any connection resistance between facing and reinforcing element as determined according to 9.5.6.

9.5.4.2 Sheet reinforcement for fill

(1) <REQ> For sheet reinforcement (geogrids and geotextiles), the value of τ_{po} in Formula (9.12) shall be determined from Formula (9.13):

$$\tau_{\text{po}}(x) = k_{\text{po}} \tan \varphi_{\text{rep}} \, \sigma'_{\text{n}}(x) \tag{9.13}$$

where:

φ_{rep} is the representative angle of internal friction of the surrounding soil;

σ'_{n} is the normal effective stress acting on the reinforcing element at point x;

x is a distance along 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) <REQ> The perimeter P of sheet reinforcement shall be taken as the sum of the widths of the top and bottom faces.

9.5.4.3 Discrete fill reinforcement

(1) For discrete fill reinforcement (strips and ladders), the value of τ_{po} in Formula (9.12) shall be determined from Formula (9.14):

$$\tau_{\text{po}}(x) = \mu_{\text{po}} \sigma'_{\text{n}}(x) \tag{9.14}$$

where, in addition to the symbols given for Formula (9.13):

μ_{po} is the coefficient of interface friction determined in laboratory pullout tests in representative conditions or from field tests.

(2) <REQ> The perimeter P of strips and ladders shall be taken as twice the width of the individual strips or ladders.

9.5.5 Resistance in direct shear

(1) <REQ> The characteristic resistance to direct shear ($R_{\text{k,ds}}$) shall be determined from Formula (9.15):

$$R_{\text{k,ds}} = B \int_0^{L_{\text{ds}}} \tau_{\text{ds}}(x) \cdot dx = B \int_0^{L_{\text{ds}}} f_{\text{ds}} \sigma'_{\text{n}}(x) \cdot dx \tag{9.15}$$

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 246. The vertical effective stress is a good approximation for the normal effective stress provided the inclination of the reinforcing element is less than 10°

NOTE 247. Figure 9.3 gives an example of horizontal sliding analysis of a reinforced fill structure.

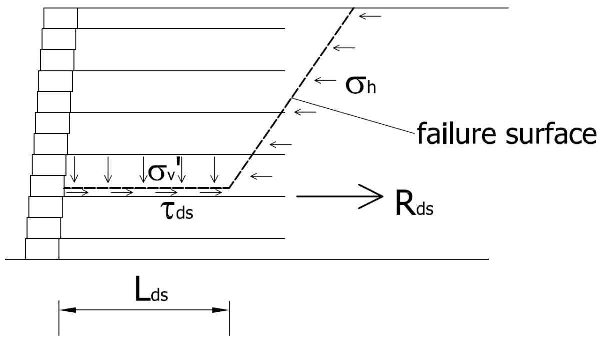


Figure 9.3 – Example of horizontal sliding analysis of a reinforced fill structure

(2) <REQ> 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 -2 for shear along an inclined plane.

(3) <REQ> Mobilised 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) <REQ> The resistance of the connection between the facing and reinforcing element shall be determined by testing the specific connection or by calculation.

(2) <REQ> If it is determined by calculation, the characteristic tensile resistance of a connection for geosynthetics or polymer steel woven wire meshes ($R_{\mathrm{k,con}}$) shall be determined from Formula (9.16):

$$R_{\mathrm{k,con}} = \eta_{\mathrm{el,con}} T_{\mathrm{k}} \tag{9.16}$$

where:

T_{k} is the characteristic tensile strength of the reinforcing element;

$\eta_{\mathrm{el,con}}$ is a reduction factor accounting for anticipated loss of strength with time and from other influences at the connection.

(3) <REQ> The reduction factor $\eta_{\mathrm{el,con}}$ shall be calculated from Formula (9.17) for geosynthetics or Formula (9.18) for polymer steel woven wire meshes:

$$\eta_{\mathrm{el,con}} = \eta_{\mathrm{gs}} \eta_{\mathrm{con}} \tag{9.17}$$

$$\eta_{\mathrm{el,con}} = \eta_{\mathrm{pwm}} \eta_{\mathrm{con}} \tag{9.18}$$

where:

η_{con} 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 9.3.3 and 9.3.5, respectively).

NOTE 248. The value of η_{con} is the reciprocal of the reduction factor specified in EBGEO as A_3, based on tests complying with EN ISO 10321.

(4) <REQ> For steel reinforcing elements, if the determination is by calculation, $R_{d,cs}$ shall comply with EN 1993-1-8.

9.6 Ultimate limit states

9.6.1 General

(1) <REQ> The design value of the ultimate limit state resistance of a reinforcement element shall satisfy formule (xx)

$$E_{d,ULS} \leq \min(R_{d,el}, R_{d,po}, R_{d,ds}, R_{d,con}) \quad (9.19)$$

where:

$E_{d,ULS}$ is the maximum value of the design value of the effects of actions in ultimate limit state (see 9.2.3.2);

$R_{d,el}$ is the design value of the resulting resistance of the reinforcement element;

$R_{d,po}$ is the design value of interface resistance between ground 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 meshe.

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) <REQ> The design tensile resistance ($R_{td,el}$) of a geosynthetic reinforcing element shall be determined from Formula (9.20):

$$R_{td,el} = \frac{R_{t,rep,el}}{\gamma_{Rd,re}\gamma_{M,re}} \quad (9.20)$$

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 249. A method to determine the value of $\gamma_{Rd,re}$ is given in ISO TR 20432, where it has the symbol f_s.

9.6.2.1.2 Steel reinforcement for fill

(1) <RCM> The design tensile resistance ($R_{td,el}$) of steel reinforcement should comply with EN 1993-1-1.

(2) <PER> As an alternativ, for steel reinforcement complying with EN 10025-2 or EN 10080, the design tensile resistance ($R_{td,el}$) may be determined from Formula (9.21):

$$R_{\text{td,el}} = min\left(\frac{A_{\text{ry}}f_{\text{yk}}}{\gamma_{\text{Rd,0}}\gamma_{\text{M0}}}; \frac{A_{\text{ru}}f_{\text{uk}}}{\gamma_{\text{Rd,2}}\gamma_{\text{M2}}}\right) \quad (9.21)$$

where:

f_{yk} is the characteristic yield strength of the steel;

f_{uk} is the characteristic ultimate strength of the steel;

A_{ry} and A_{ru} are the reduced cross-sectional areas defined in 9.3.2.3;

γ_{M0} and γ_{M2} are partial factors whose values are specified in EN 1993-1-1;

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

NOTE 250. The values of $\gamma_{Rd,0}$ and $\gamma_{Rd,2}$ are 1.1 and 1.0, respectively, unless the National annex gives different values.

9.6.2.1.3 Polymeric coated steel woven wire mesh

(1) <REQ> The design tensile resistance ($R_{td,el}$) of polymeric-coated woven wire mesh reinforcing element shall be determined from Formula (9.25):

$$R_{\text{td,el}} = \frac{R_{\text{t,rep,el}}}{\gamma_{\text{Rd}}\gamma_{\text{M,pwm}}} \quad (9.22)$$

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 251. A method to determine the value of γ_{Rd} is given in ISO TR 20432, where it has the symbol f_s.

9.6.2.2 Failure at the interface between the ground and the reinforcing elements (pull-out)

(1) <REQ> The design pull-out resistance ($R_{d,po}$) of a reinforcing element shall be determined from Formula (9.23):

$$R_{\text{d,po}} = \frac{R_{\text{rep,po}}}{\gamma_{\text{R,po}}} \quad (9.23)$$

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) <REQ> The design resistance to direct shear mobilised along the interface between the fill or ground and the reinforcing element ($R_{d,ds}$) shall be determined from Formula (9.24):

$$R_{d,ds} = \frac{R_{k,ds}}{\gamma_{R,ds}} \tag{9.24}$$

where:

$R_{k,ds}$ is the characteristic resistance to direct shear;

$\gamma_{R,ds}$ is a partial factor, given in 9.6.2.6.

(2) <PER> $R_{d,ds}$ may be neglected for discrete reinforcement (strips, steel ladders) provided it can be demonstrated that the reinforcing elements do not reduce the direct shear strength.

9.6.2.4 Rupture of the connections

(1) <REQ> The design tensile resistance of a connection for geosynthetics or polymer woven wire meshes ($R_{d,con}$) shall be determined from Formula (9.25):

$$R_{d,con} = \frac{R_{k,con}}{\gamma_{R,con}} \tag{9.25}$$

where:

$R_{k,con}$ is the characteristic tensile resistance of the connection;

$\gamma_{R,con}$ is a partial factor for the connection, given in 9.6.2.6;

9.6.2.5 Failure of facing elements

(1) <REQ> EN 1997-3, 10 shall apply.

9.6.2.6 Partial factors

(1) <REQ> EN 1997-1, 4.4.1(4) shall apply.

NOTE 252. 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	Values of the partial factors
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 Reinforcing element	Tensile strength of	geosynthetic reinforcement	$\gamma_{M,re}$	1,1
		steel reinforcement	γ_{M0}, γ_{M2}	specified in EN 1993-1-1
		steel reinforcement	$\gamma_{R,re}$	1,2
		polymeric coated steel wire mesh reinforcement	$\gamma_{M,pwm}$	1,25
Rupture Connections between reinforcing elements	Tensile strength of polymeric coated steel wire mesh reinforcement		$\gamma_{M,con}$	1,25
	Tensile strength of polymeric coated steel woven wire mesh connection			1,35

9.7 Serviceability limit states

9.7.1 General

(1) <REQ> EN 1997-1, 9 shall apply.

9.7.2 Serviceability limit states of whole structure and it subsoil

(1) <REQ> Verification of servicability limit state due to loading of the reinforced fill structure including subsoil shall comply with Clauses 4, 5, and 7, as appropriate.

(2) <REQ> It shall be verfied that the deformation of the reinforced fill strucutre are within the limit values for the used facing elements.

NOTE 253. 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) <REQ> Total and differential deformation of the reinforced fill structure both vertically and horizontally shall be in compliance with the specified limit values.

(2) <REQ> Internal deformation of the reinforced fill structure shall be in compliance with the specified limit values.

9.7.4 Serviceability limit states of reinforcing element

(1) <REQ> Elongation of the reinforcing elements both in the short and long term shall be in compliance with specified limit vaues.

9.7.5 Serviceability limit states of facing element

(1) <REQ> EN 1997-3, 10 shall apply

(2) <REQ> Bulging at the toe of a reinforced veneer system shall be limited to specified values.

9.8 Implementation of design during excution and service life

9.8.1 General

(1) <REQ> EN 1997-1, 10 shall apply.

(2) <REQ> The execution and controll of reinforced fill structures shall comply with EN 14475.

(3) <REQ> The execution specifcation shall include level of the excavation with construction tolerances.

(4) <REQ> Groundwater controll measures shall be specified in accordance with clause 12.

9.8.2 Inspection

(1) <REQ> EN 1997-1, 10.3 shall apply

(2) <REQ> In addition to (1), the Inspection Plan shall include:

- 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 colerances;
- 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 adequate performance of any drainage system installed.

9.8.3 Monitoring

9.8.3.1 General

(1) <REQ> EN 1997-1, 10.4 shall apply

(2) In addition to EN 1997-1, 10.4, the Monitoring Plan should include:

- behaviour of temporary support systems;
- monitoring of the behaviour of reinforcement element;
- lateral and vertical displacements and distortions.

9.8.4 Maintenance

(1) <REQ> EN 1997-1, 10.5 shall apply.

9.9 Testing

(1) <REQ> EN 1997-1, 11 shall apply.

9.9.1 Interface strength

(1) <REQ> The determination of interface shear strength between soil and geosynthetic or polymeric coated steel woven wire mesh reinforcement in the laboratory shall comply with EN ISO 12957 (all parts) with respect to the position of the reinforcing element in the reinforced structure.

(2) <REQ> 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) <REQ> The determination of connection tensile strength between individual geosynthetic or polymeric coated steel woven wire mesh reinforcing elements shall comply with EN ISO 10321.

9.10 Reporting

(1) <REQ> EN 1997-1, 12 shall apply.

10 Ground reinforcing elements

<Drafting note> The table of content for this clause has been sligthly adjusted for drafting purposes. At a later stage, a decision will be taken if it should be kept or this Clause should follow the same outline as the other Clauses in EN 1997-3. The decision is linked to TC250 drafting instructions.>

10.1 Scope and field of application

(1) <REQ> This clause shall apply to ground reinforcing elements that provide resistance to prevent a limit state of the geotechnical structure.

NOTE 254. Ground reinforcing element include rock bolts; soil nails; sprayed concrete; wire mesh and facing elements.

NOTE 255. For anchors clause 8 applies.

NOTE 256. Safety nets, snow fences or avalanche protections are not covered by this clause.

NOTE 257. Reinforcing elements in underground openings is not covered by this clause.

(2) <REQ> This Clause shall apply to the verification of the ultimate limit state, serviceability limit state, durability and robustness of the ground reinforcing elements itselves.

(3) <REQ> In addition to EN 1997-1, the appropriate clauses in 4, 5, 6, 7 and 9 for the involved geotechnical structures shall apply.

NOTE 258. This is valid for all clauses in this Clause 10, and therefore not repeated further.

10.2 Basis of design

10.2.1 Design situations

(1) <REQ> EN 1997-1, 4.2.2 shall apply.

(2) <REQ> In addition to (1) design situations for ground reinforcing elements shall include but not be limited to:

– temporary or permanent nature of the reinforcing element;
– 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 britle failure of the reinforced structure;
– effect of corrosion.

10.2.2 Geometrical properties

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

(2) <REQ> Accessibility of drilling and installation equipment shall be taken into account in determining the geometrical properties of the reinforcing element.

10.2.3 Actions and environmental influences

10.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

10.2.3.2 Permanent and variable actions

(1) <REQ> Design value of the force in the reinforcing elements shall be obtained from verification of limit states for the reinforced structure.

(2) <REQ> Design reinforcing elements force required to prevent failure by overall stability shall comply with Clause 4.

(3) <REQ> Design reinforcing elements force required to prevent failure by loss of bearing resistance and by sliding shall comply with Clause 5.

(4) <REQ> Design reinforcing elements force required to prevent failure by loss of equilibrium shall comply with to Clause 7.

10.2.3.3 Cyclic and dynamic actions

(1) <REQ> EN 1997-1, 4.3.1.3 shall apply.

10.2.3.4 Environmental influences

(1) <REQ> EN 1997-1, 4.3.1.5 shall apply.

(2) <REQ> 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.

10.2.4 Limit states

(1) <REQ> EN 1997-1, 8 and 9 shall apply.

(2) <REQ> In addition to EN 1997-1, 8.2, the following ultimate limit states shall be verified, as relevant;

- 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 faicing;
- failure by loss of bearing-resistance in the ground below reinforcing element (punching).

(3) <REQ> In addition to EN 1997-1, 9, the following servicability limit states shall be verified, as relevant;

- elongation of the reinforcing element;
- bulging and deformation of any facing element;
- cracking or spalling of any precast facing panels or sprayed concrete.

(4) <RCM> Limit states other than those given in (1) and (2) should be verified as necessary.

(5) <REQ> If the ground reinforcing elements consist of multiple types of elements, the resistance of each element type and the combined reinforcing resistance shall be verified.

(6) <REQ> In addition to (2) and (3), the verfication of the limit states shall prevent a potential brittle failure of the reinforced structure.

10.2.5 Robustness

(1) <REQ> In addition to EN 1997-1, 4.2.2, the appropriate sub-clauses on robustness in 4, 5, 6 and 7 for the involved geotechnical structures shall apply.

(2) <RCM> Specification of measures to enhance robustness of a reinforced structure with rock should include, as relevant;

- 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.

NOTE 259. One example is a set of discontinuities directly located under existing spread foundations or basements and directly next to the upcoming excavation .

10.2.6 Ground investigation

(1) <REQ> EN 1997-2, 5 shall apply.

(2) <RCM> The zone of ground, into which forces of the reinforcing elements are transferred, should be included in the ground investigation.

10.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1,4.1.2.3 shall apply.

10.3 Materials

10.3.1 Ground

(1) <REQ> EN 1997-1, 5.1 and EN 1997-2 shall apply.

10.3.2 Steel

(1) <REQ> EN 1997-1, 5.6 shall apply.

10.3.3 Grout

(1) <REQ> EN 1997-1, 5.4 shall apply.

10.3.4 Cast and sprayed concrete

(62)<REQ> EN 1997-1, 5.5 shall apply.

10.3.5 Steel fibres

(1) <REQ> Steel fibres in sprayed concrete shall comply with EN 14487-1.

(2) <PER> Fibres of other materials in sprayed concrete may be used.

(3) <REQ> If other material than steel fibres are used, 10.3.8 shall apply.

10.3.6 Coatings

(1) <REQ> For galvanised steel reinforcing elements, the hot dip galvanized coating to steel shall comply with EN ISO 1461.

(2) <REQ> For a zinc-aluminium alloy coated steel welded wire meshes the coating shall comply with EN 10244-2.

(3) <REQ> Epoxy coating shall comply with EN 13438.

(4) <REQ> Polymeric coated steel shall comply with EN 10245 (all parts)

(5) <REQ> The characteristic tensile strength of polymeric coated steel woven wire mesh reinforcing shall be determined in accordance with EN ISO 10319.

10.3.7 Concrete panels and other facing elements

(1) <REQ> The properties of concrete facing panels shall comply with EN 1992-1-1 and, for precast products, also to EN 15258; concrete facing blocks shall comply with EN 771-3.

(2) <REQ> Facing elements made of the same material as the reinforcing elements for fill applications shall comply with the same standard, as defined in 9.3.

(3) <REQ> Facing elements of steel, masonry, or timber shall comply with ENs 1993-1-1, 1995-1, and 1996-1, as appropriate.

10.3.8 Other materials

(1) <REQ> Materials other than specified shall only be used for reinforcing elements if they comply with a standard specified by the relevant authority or, where not specified, agreed for a specific project by appropriate parties.

10.3.9 Durability

(1) <REQ> EN 1997-1, 4.1.6 shall apply.

(2) <RCM> EN 1993-5, 6 should apply.

(3) <REQ> 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, 6.4);
- grout, mortar or concrete protection;
- grouted duct;
- protective surface coating;
- appropriate steel material (see EN 1993-5, 6.1).

(4) <REQ> Galvanic steel corrosion of different connecting elements shall be the prevented.

(5) <RCM> Where the corrosion protection is provided by sacrificial thickness allowance, ground-specific loss of steel thickness (Δe) should be determined.

NOTE 260. Values of $\Delta e/2$ for black steel nails/bolts without any corrosion protection measures for different service lives are given in EN 1993-5, Tables 6.1 and 6.2.

(6) <RCM> Corrosion protection provided by grout cover (with or without duct), surface coating, or use of stainless steel should comply with EN 14490.

(7) <RCM> The selection of an appropriate system of measures for durability should consider:

- the feasability 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) <REQ> EN 1997-1, 6, shall apply.

(2) <REQ> For groundwater control measures, clause 12 shall apply.

10.5 Rock bolts

10.5.1 Geotechnical analyses

10.5.1.1 General

(1) <REQ> EN 1997-1, 7 shall apply

(2) <REQ> In addition to (1) the geotechnical analyses shall address all relevant limit state verifications listed in 10.2.4.

10.5.1.2 Resistance of rock bolts

(1) <RCM> The design of rock bolts should include but not limited to

- Type of bolt;
- connection of bolt to an external structure (or absence of it);
- grouting of bolt (or absence of it);
- use of an additional anchor plate to the bolt (or absence of it);
- effects of corrosion and corrosion protection needs; and
- type of loading of the bolt.

(2) <REQ> Rock bolt 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) <REQ> The rock bolt grid and diameter shall be determined by the depth of discontinuities or weathered zone causing possible failure.

(4) <REQ> The length of the rock bolts shall include a fixed length in the rock beyond the discontinuities or weathered zone.

(5) <RCM> The rock bolt types, geometrical properties and installation grid density should be defined by the distribution of the slope, dip and direction of discontinuities or weathered zone causing possible failure.

(6) <PER> Rock bolts may be prestressed.

NOTE 261. The prestressed tension is usually between 25 and 50 kN.

(7) <REQ> In case of pre-stressing of a rock bolt, its influence both on the rock bolted structure and on the ground shall be addressed.

10.5.1.3 Resistance at the interface of the rock bolt (pullout)

(1) <REQ> The rock bolt total length shall be defined by the extent of discontinuities into the ground mass or extent of weakness zones causing possible failure added with fixed length inside the rock beyond the discontinuities or weathered zone.

(2) <REQ> The fixed length of the rock bolts shall be sufficient to avoid pull-out of the interface between the bolt and the surrounding grout or rock and failure at the interface between the grout and the rock.

(3) <REQ> In order to ensure sufficient fixed length, the pull-out resistance of a rock bolt shall exceed the tension resistance of the rock bolt.

(4) <REQ> The representative pull-out resistance ($R_{\mathrm{rep,po}}$) of a rock bolt shall be determined from Formula (10.1)

$$R_{\mathrm{rep,po}} = P \cdot \tau_{\mathrm{po}} \cdot L_{po} \qquad (10.1)$$

where:

P is the reprensentative perimeter of the interface area, either drilled hole or the rock bolt;

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 fixed length of the rock bolt beyond the discontinuity or weathered zone, where pull-out stresses are mobilised.

10.5.2 Ultimate limit state

10.5.2.1 Verification by partial factor method

(1) <REQ> EN 1997-1, 4.4.1 shall apply.

(2) <REQ> The design tensile resistance ($R_{\mathrm{td,el}}$) of steel of rock bolts shall comply with EN 1993-1-1, 8.

(3) <REQ> The design shear resistance ($R_{\mathrm{sd,el}}$) of steel of rock bolts shall comply with EN 1993-1-1, 8..

(4) <REQ> For rock bolts loaded in tension and shear the angle between loading action direction and the angle of rock bolt installation shall be taken into account.

(5) <REQ> The design pull-out resistance ($R_{\mathrm{d,po}}$) of a rock bolt shall be determined from Formula (10.2).

$$R_{\mathrm{d,po}} = \frac{R_{\mathrm{rep,po}}}{\gamma_{\mathrm{R,po}}} \qquad (10.2)$$

where:

$R_{rep,po}$ is the representative pullout resistance of the rock bolt;

$\gamma_{R,po}$ is a partial factor, given in Table 10.1.

NOTE 262. 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	Values of partial factors
Structural resistance of reinforcing element and any connections.	Steel		See EN 1993-1
Geotechnical resistance, mobilised at the interface between rock bolt, grout and/or rock.	Pullout	$\gamma_{R,po}$	1,5

10.5.2.2 Verification by prescriptive rules

(1) <REQ> EN 1997-1, 4.5 shall apply.

(2) <PER> 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) <REQ> If prescriptive rules are used for verification, the inspection plan shall include quality measures to ensure that the installed bolts fulfill the limitations specifed for the prescriptive rule.

10.5.2.3 Verification by testing

(63) <REQ> EN 1997-1, 4.6 shall apply.

(64) <REQ> For testing 10.5.5 shall apply.

10.5.2.4 Verification by Observational Method

(1) <REQ> EN 1997-1, 4.7 shall apply.

(2) <RCM> The amount, grid and locations of the rock bolts to be installed in relation to the observed conditions at site should be verified by the Observational Method

10.5.3 Servicability limit state

(1) <REQ> EN 1997-1, 9 shall apply, as appropriate.

10.5.4 Implementation of design during execution and service life

(1) <REQ> EN 1997-1, Clause 10, shall apply.

(2) <REQ> Grouted rock bolts shall be grouted over their full length of the rock bolt.

(3) <RCM> Grouted rock bolts should be installed in dry rock conditions.

(4) <RCM> If dry rock conditions cannot be achieved, additional measured should be used.

10.5.5 Testing

(1) <REQ> EN 1997-1, Clause 11, shall apply.

(2) <REQ> Acceptance tests shall be used to confirm an adequate installation of the rock bolt and to control the quality of the grout.

NOTE 263. The required number of acceptance tests depends on the type, size and conditions of the structure to be supported.

NOTE 264. The minimum amount of acceptance tests is 1 every 200 bolts with a minimum of 3, unless the national annex gives a different value.

NOTE 265. Acceptance tests are performed on productions bolts included in the final structure.

(3) <RCM> 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 266. The tests are e.g. oltometer tests and RBT (Rock Bolt Tester) tests.

(4) <PER> In alternative to (2) 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.

(5) <REQ> The acceptance criterion for grouted rock bolts shall be the verification of 10.5.4 (2).

(6) <REQ> 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.

(7) <PER> Pullout tests for rock bolts may be conducted on non-grouted rock bolts or on grouted bolts before hardening of the grout, as an alternative to non-destructive in situ testing.

10.6 Soil Nailing

10.6.1 Geotechnical analyses

10.6.1.1 General

(1) <REQ> EN 1997-1, 7 shall apply

(2) <REQ> In addition to (1) the geotechnical analysis shall address all limit state verifications listed in 10.2.4.

(3) <REQ> Horizontal and vertical displacement of a structure reinforced with soil nail shalls be analysed according to Clauses 4, 5, 7, or 9, as appropriate.

10.6.1.2 Resistance at the interface of the soil nail (pull-out)

(1) <REQ> The resistance of a soil nail due to pull-out from the ground shall be verified for both the part of the soil nail infront and behind the potential critical failure surface.

NOTE 267. Figure 10.1 gives a sketch of a soil nailed wall/cuting.

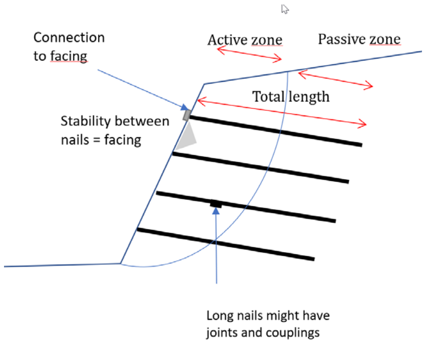


Figure 10.4 – Example of a wall/cutting reinforced with soil nails.

(2) <REQ> The representative pullout resistance ($R_{\text{rep,po}}$) of a soil nail shall be determined from Formula (10.3)

$$R_{\text{rep,po}} = P \cdot \tau_{\text{po}} \cdot L_{po} \tag{10.3}$$

where:

P is the representative length of the perimeter of the failure surface enclosing the soil nail, 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 beyond (or in-front) of the failure surface of the structure, where pull-out resistance are mobilised.

(3) <RCM> 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) <PER> Formula (10.3) may also be used for calculation of representative value of pull-out resistance between the steel-core of the soil nail and the grouted body.

NOTE 268. The failure between steel-core and grouted body can be neglected for soil nails that has been enhanced and verified to avoid this failure mode.

NOTE 269. Pull-out resistance can be influenced by dynamic action.

(5) <RCM> The perimeter of the soil nail, P, should be determined as a nominal value with consideration of nail type and ground properties.

NOTE 270. For soil nails that is not circulair e.g. L-shape or grouted soil nails, the perimater is estimated based on assumed shape of the failure surface enclosing the soil nail.

(6) <PER> The perimeter of a grouted soil nail may be determined as a nominal value of the perimeter of the drilled hole for installation.

(7) <REQ> 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) <REQ> The interface shear resistance shall be confirmed by project-specific investigation tests, before or during execution, see 10.6.5.1.

NOTE 271. Investigation test is used to confirm the ultimate interface friction in the passive zone, active zone or the entire length of the nail.

(9) <PER> As alterantive 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) <REQ> In addition to (8) and (9) 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) <PER> The representative pullout resistance from the active zone (Figure 10.1) may be increased by any resistance mobilised at the connection to the facing determined according to (10.7).

10.6.2 Ultimate limit state

10.6.2.1 General

(1) <REQ> The design value of the ultimate limit state resistance of a soil nail ($R_{\mathrm{d,SN}}$) shall satisfy formule (10.4)

$$E_{\mathrm{d,ULS}} \leq R_{\mathrm{d,SN}} \tag{10.4}$$

$$R_{\mathrm{d,SN}} = \min(R_{d,po}\,, R_{d,el}, R_{d,con}) \tag{10.5}$$

where:

$E_{\mathrm{d,ULS}}$ is the maximum value of the design value of the effects of actions in ultimate limit state (see 10.2.3.2);

$R_{\mathrm{d,po}}$ is the design value of a soil nails interface resistance at the ultimate limit state (pull-out);

$R_{\mathrm{d,el}}$ is the design value of the resulting resistance of the steel core of the soil nail;

$R_{\mathrm{td,con}}$ is the minimum design value of the resulting resistance of the joints/couplings between two different soil nails or the connection to the facing.

10.6.2.2 Verification by partial factor method

10.6.2.2.1 General

(1) <REQ> EN 1997-1, 4.4.1, shall apply.

10.6.2.2.2 Failure at the interface between the ground and the soil nail (pullout)

(2) <REQ> 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:

$R_{rep,po}$ is the representative pull-out resistance of the reinforcing element;

$\gamma_{R,po}$ is a partial factor, given in Table 10.3

(3) <RCM> The representative pull-out resistance may be determined from investigation tests or by comparable experience.

NOTE 272. The criteria to determine the pull-out resistance are given in 10.6.1.2.

(4) <REQ> The design pull-out resistance shall be verified by acceptance tests according to 10.6.5.1.

NOTE 273. The minimum number of investigation and acceptance test is given in Table 10.2 (NDP), unless the national annex give 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 at 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 at minimum of 5 nails.

10.6.2.2.3 Rupture of the soil nail (tensile and shear)

(1) <REQ> The design tensile resistance ($R_{td,el}$) of steel soil nails shall comply with EN 1993-1-1, 8, accounting for any anticipated loss of strength with time.

(2) <REQ> The design shear resistance ($R_{sd,el}$) of steel soil nails shall comply with EN 1993-1-1, 8, accounting for any anticipated loss of strength with time.

(3) <PER> 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 minor, the shear resistance may be omitted from the verification.

(4) <REQ> Where the corrosion protection is provided by sacrificial thickness allowance, the reduced cross-sectional area shall be determined from 10.6.5.2

(5) <REQ> When the design includes shear and bending effects of the soil nail, the structural resulting resistance shall be determined according to the EN 1993-1-1, 8.2.10 for combined axial, shear, and bending actions.

10.6.2.2.4 Tensile resistance of connections, joints and couplings

(1) <REQ> The design tensile resistance of a connection, joint or coupling ($R_{d,con}$) shall be verified for the same design load the soil nail itself.

10.6.2.2.5 Partial factor

(1) <RCM> The ultimate geotechnical resistance of a reinforcing element shall be verified using a factor γ_R on resitance according to Formula (10.3).

NOTE 274. Values of the partial factors are given in 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	Value of partial factor
Structural resistance of reinforcing element and any connections.	Steel		See EN 1993-5
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) <PER> 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) <REQ> If prescriptive rules are used for verification, the Inspection plan shall include quality measures to ensure that the installed soil nails fullfil the limitations specified for the prescriptive rules.

(3) <REQ> If the inspection in (2) gives that the soil nail is not compling with the limitations specified, testing according to 10.6.5 shall be performed to confirm the design.

10.6.3 Servicability limit state

(1) <REQ> EN 1997-1, 9 shall apply.

10.6.4 Implementation of design during execution and service life

(1) <REQ> EN 1997-1,10 shall apply.

(2) <REQ> In additiona to (1), EN 14490 shall apply.

10.6.5 Testing

10.6.5.1 General

(1) <REQ> EN 1997-1,10 shall apply.

10.6.5.2 Pull-out resistance

(1) <REQ> Testing of soil nails shall comply with EN 14490, Annex C.

NOTE 275. Investigation tests are in EN 14490 referred to as sacrificial nail test

NOTE 276. Acceptance tests are in EN 14490 referred to as production nail test

NOTE 277. Limiting values for acceptance criteria in investigation and acceptance tests are given in Table 10.4(NDP), unless the National Annex gives different values.

(2) <REQ> The testload for acceptance tests Pp shall be equal to the design value of the effect of actions E_d (see Formula 10.4).

(4) <REQ> The design pull-out resistance has been verified with the acceptance test when the specified creep rate in Table 10.4 is exceeded at the value of Pp.

(5) <RCM> For investigation tests the targets test load, P_p, should be estimated from the expected representative pull-out resistance (see Formula 10.3).

(6) <REQ> The representative pull-out resistance is determined as maximum test load in the investigation test, where the creep rate does not exceed the acceptance critera in Table 10.4.

Table 10.4 (NDP) – Acceptance critera for investigation and acceptance test of Soil nails.

Acceptance criteria	**Investigation test**	**Acceptance test**
Creep rate[1] 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}[2]	< the elastic extension of L_{db}
1) The creep rate is defined as $(s_2 - s_1)/\log(t_2/t_1)$, where s_1 and s_2 are the measured nail displacments at time 1 and time 2 respectivly. [time 2 > time 1] 2) Ldb is the debonded length of the test nail.		

(7) <PER> The acceptance criteria of the creep rate may be adjusted to a smaller value in the design.

(8) <RCM> The test nails should be evenly distributed throughout the structure.

(9) <RCM> Investigation test should be performed for the part of the soil nail, which has to provide the designed pull out resistance.

(10) <PER> 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) <REQ> If the execution involves excavation, the face stability should be tested in accordance with EN 14490.

(2) <PER> The face stability test may be omitted if the face stability can be proven by comparable experience.

10.7 Wire mesh

10.7.1 Geotechnical analyses

10.7.1.1 General

(1) <PER> Wire mesh solutions may be used, when loosened rock or loosening of rock blocks behind the mesh exceeds a limit state.

10.7.1.2 Rupture of wires

(1) <REQ> Wire mesh shall be designed to be connected to the rock appropriately, that its connection element entends into firm ground beyond any discontinuity or weathered zone.

(2) <REQ> The capacity of the wires and connection of the wires in the wire mesh shall be verified.

(3) <REQ> 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) <REQ> The design resistance of a connection ($R_{\text{d,con}}$) shall be verified for at least the same as the design resistance of the wire mesh itself.

(2) <REQ> If the wire mesh is connected to bolts or nails, the connection resistance of the wire mesh to the anchor plates shall be verified.

(3) <REQ> If the wire mesh is connected to bolts or nails, the size of anchor plates shall well exceed the size of the mesh opening.

(4) <REQ> 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) <REQ> EN 1997-1, 4.2 shall apply.

10.7.2.2 Verification by partial factor method

(1) <REQ> EN 1997-1, 4.4 shall apply.

(2) <REQ> The design tensile resistance ($R_{\text{td,el}}$) of steel of the wires shall comply with EN 1993-1-1, 8.2.3.

(65) <REQ> The design connection resistance ($R_{\text{d,co}}$) of a wire mesh shall be determined from Formula (10.7)

$$R_{\text{d,con}} = \frac{R_{\text{rep,con}}}{\gamma_{\text{R,con}}} \tag{10.7}$$

where:

$R_{\text{rep,con}}$ 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

NOTE 278. Values of the partial factors are given in 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 EN 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) <REQ> EN 1997-1, 4.5 shall apply.

(2) <PER> 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) <REQ> If prescriptive rules are used for verification, the Inspection plan shall include quality measures to ensure that the installed wire meshes fulfill the limitations specifed for the prescriptive rule.

10.7.2.4 Verification by testing

(1) <REQ> EN 1997-1, 4.6 shall apply.

(2) <RCM> Verification by testing of a wire mesh and its connections should be conducted.

10.7.2.5 Verification by Observational Method

(1) <REQ> EN 1997-1, 4.7 shall apply.

(2) <RCM> The extent and locations of the wire meshes to be installed in relation to the observed conditions at site should be verified by the Observational Method.

10.7.3 Servicability limit state

(1) <RCM> The limit state of the wire mesh should also be verified on excessive deformation, if project related serviceability restrictions are put in place.

10.7.4 Implementation of design during execution and service life

(1) <RCM> For structures belonging to GC2 or GC3 loosened rock hanging on to the wire mesh should be removed and the wire mesh should be replaced.

(2) <REQ> If the wire mesh is connected to bolts or nails, the anchor plates shall be visually inspected that they are fully connected to the mesh and ground surface.

(3) <REQ> If the wire mesh is embedded in sprayed concrete, the wire mesh shall be fully covered by sprayed concrete on both sides of the mesh.

10.7.5 Testing

(1) <REQ> Testing shall comply with EN 1997-1, Clause 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) <REQ> The thickness of the sprayed concrete shall be defined by its bearing capacity demands to withhold the loads of rock blocks, grade of jointed rock mass, weathered zones and weakness of the rock mass to prevent outfall of rock.

(2) <REQ> As alternative to (1) the sprayed concrete shall be designed to withhold the earth pressure from the ground according to Clause 7.

(3) <RCM> The minimum thickness should take into account the execution restrictions.

(4) <RCM> The minimum thickness should be defined such, that the bearing capacity is feasible to achieve in practice.

NOTE 279. Thicknesses of 30 mm or less are considered cosmetic.

10.8.2 Ultimate limit state

10.8.2.1 General

(1) <REQ> EN 1997-1, 4.2 shall apply.

(2) <REQ> For the specifications and conformity of sprayed concrete EN 14487-1 shall apply.

10.8.2.2 Verification by partial factor method

(1) <REQ> EN 1997-1, 4.4 shall apply.

(2) <REQ> For sprayed concrete reinforcing verification EN 1992 (all parts) shall apply.

10.8.2.3 Verification by prescriptive rules

(1) <REQ> EN 1997-1, 4.5 shall apply.

(2) <REQ> If prescriptive rules are used for verification, the Inspection plan shall include quality measures to ensure that the installed sprayed concrete fulfill the limitations specifed for the prescriptive rule.

10.8.2.4 Verification by testing

(1) <REQ> EN 1997-1, 4.6 shall apply, as appropriate.

(2) <REQ> For testing during execution 10.8.5 shall apply.

10.8.2.5 Verification by Observational Method

(1) <REQ> EN 1997-1, 4.7 shall apply.

(2) <RCM> The extent and thickness of the sprayed concrete to be installed in relation to the observed conditions at site should be verified by the Observational Method.

10.8.3 Servicability limit state

(1) <REQ> EN 1997-1, 9 shall apply, as appropriate.

10.8.4 Implementation of design during execution and service life

(1) <REQ> Execution shall comply with EN 1997-1, Clause 10.

(2) <REQ> The ground surface shall be prepared / cleaned properly to achieve adhesion bondage between ground and sprayed concrete.

(3) <RCM> Sprayed concrete should be installed in dry or controlled water conditions to avoid reduction of adhesion.

(4) <RCM> Water leakages should be under control before execution of sprayed concrete.

(5) <RCM> For water leakage areas drainage systems or grouting should be considered.

10.8.5 Testing

(1) <REQ> EN 14487 and EN 14488 shall apply.

(2) <REQ> Sprayed concrete shall be tested to verify its energy absorption capacity.

(3) <REQ> The sprayed concrete shall be tested on its adhesion to the ground surface.

(4) <REQ> Nominal sprayed concrete thicknesses shall be verified.

(5) <PER> 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) <REQ> EN 1997-1, 7 shall apply

(2) <REQ> The resistance of a facing element shall be verified in accordance with 10.6.1.

(3) <REQ> In addition to (1) the geotechnical analysis shall address all limit state verifications listed in 10.2.4.

(4) <REQ> Horizontal and vertical deformations of a structure reinforced with facing elements shalls be analysed according to Clauses 4, 5, 7 or 9, as appropriate.

10.9.2 Ultimate limit state

(1) <REQ> The structural resistance of geosynthetic facing elements shall comply with 9.6.

(2) <REQ> The structural resistance of facing elements of concrete, steel, masonry, and timber shall comply with EN 1992 (all parts), EN 1993 (all parts), EN 1996 (all parts) and EN 1995 (all parts), as appropriate.

(3) <PER> The design strength of facing elements may be determined by testing.

NOTE 280. Guidance about design assisted by testing is given in EN 1990, Annex D.

(4) <REQ> The bending and shear resistance to bulging between facing elements when some of them are not connected to ground reinforcings shall be verified.

(5) <REQ> The shear resistance between facing elements and reinforcings when the connection relies purely on friction shall be verified.

(6) <REQ> The stability against toppling of the facing elements not connected to ground reinforcements above the top layer of reinforcement shall be verified.

(7) <REQ> The punching resistance of the facing shall be verified.

(8) <REQ> The flexural resistance and reinforcement detailing of concrete, steel, and other hard facings shall be verified.

(9) <REQ> The durability of the facing material itself and all connections for the design service life shall be verified.

NOTE 281. 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 282. 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 Servicability limit state

(1) <REQ> The bulging of segmental block and flexible facing systems shall be limited to ensure compliance with the specification.

(2) <REQ> The deformations of the structure face shall be limited to avoid spalling and cracking of facing panels.

(3) <REQ> Bulging at the toe of a reinforced veneer system shall be limited to values given in the specification.

10.9.4 Implementation of design during execution and service life

(1) <REQ> EN 1997-1, Clause 10, shall apply.

10.9.5 Testing

(1) <REQ> Execution shall comply with EN 1997-1, Clause 11, and with the appropriate sub-clause in this standard for the involved geotechnical structure.

10.10 Reporting

(1) <REQ> Reporting for reinforcing elements shall comply with EN 1997-1, 12.

11 Ground improvement

11.1 Scope and field of application

(1) <REQ> Ground improvement design shall be classified according to Table 11.1 and divided into two families:

– diffused ground improvement (classes AI and AII); or
– discrete ground improvement (classes BI and BII).

NOTE 283. Examples of ground improvement techniques in these two families are given in Annex G.

NOTE 284. Groundwater control techniques are addressed in Clause 12.

Table 11.1 – Classification of ground improvement design

Class	Family	
	A – Diffused	**B – Discrete**
I	**AI – Diffused with no unconfined compressive strength** Increased shear strength and/or reduced permeability compared to the original ground but can be classified as improved ground	**BI – Discrete with non-rigid inclusions** Containing inclusions with increased shear capacity and stiffness compared to the surrounding ground
II	**AII – Ground improvement zone with unconfined compressive strength** Modified from original natural state and having measurable unconfined compressive strength and significantly stiffer than the surrounding ground and/or of reduced permeability. Usually comprises a composite of a binder and ground. Usually behaves as a structural zone	**BII – Discrete with rigid inclusions** Containing rigid inclusions with unconfined compressive strength significantly stiffer[1] than the surrounding ground; can be an engineered material such as timber, concrete/grout or steel or a composite of a binder and ground
[1]The term "rigid inclusion" is a common usage for many types of discrete inclusions and can apply to a wide range of materials. The stiffness ratio with the surrounding ground can therefore vary between 10 or less and 4000.		

(2) <REQ> This Clause shall apply to ground improvement for the following geotechnical structures and applications:

– slopes, cuttings, and embankments (see also Clause 4);
– spread foundations (see also Clause 5);
– retaining structures (see also Clause 7).

(3) <RCM> For BII techniques, the following conditions should be satisfied:

– structural connection with the foundation is not existing (contact only or presence of a load transfer platform) ;
– structural loads are transferred directly or through a load transfer platform into the ground.

(4) <PER> In the absence of load transfer platform, additional verifications may be considered during the design and the execution according to the design situations.

(5) <REQ> A single element (considered as BI or BII technique) in isolation used to transfer load to the subsoil shall be either designed as a spread foundation (Clause 5) or as a pile (Clause 6).

11.2 Basis of design

11.2.1 Design situations

(1) <REQ> EN 1997-1, 4.2.2 shall apply.

(2) <REQ> For ground improvement subject to alteration over time, design of temporary works shall specify the maximum design life or specify any extensions to the period of temporary use.

NOTE 285. 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

11.2.2.1 General

(1) <REQ> EN 1997-1, 4.3.3 shall apply.

(2) <REQ> Geometric tolerances shall be adopted that are not less than those specified in the execution standards specified in 11.8.

NOTE 286. Some execution standards give very basic tolerances that are not suitable for some designs. For example, drilling tolerances are generally large and might not provide sufficient tolerance for some groundwater control designs.

(3) <REQ> If the design of the ground improvement is sensitive to the deviation of a particular geometrical property then the design value of that property ad shall be determined from Formula (11.1):

$$a_{\mathrm{d}} = a_{\mathrm{nom}} \pm \Delta a \tag{11.1}$$

where:

(66) a_{nom} (67) is the nominal value of the geometrical property;

(68) Δa (69) is the deviation in the geometrical property from its nominal value.

NOTE 287. 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 Control Testing is carried out and no comparable experience is available	Property is determined by measurement or by use of comparable experience
Soil mix/bored/vibrated inclusion diameter	5 % of nominal diameter	0 (not applicable)
Jet Grout inclusion diameter	10 % of a_{nom} or 0.1 m whichever is greater	5 % of a_{nom} or 0.05 m whichever is greater
Compaction Grout inclusion diameter	Measurement of the grout volume per linear metre required	Measurement of the grout volume per linear metre required
Stone or sand inclusion diameter	10 % of a_{nom} or 0.1 m whichever is greater If available average measurement of the installed material quantities (based on tests and controls)	5 % of a_{nom} or 0.05 m whichever is greater If available average measurement of the installed material quantities (based on tests and controls)
Driven or vibrated steel/wood or concrete inclusion diameter	0 (not applicable)	0 (not applicable)
Inclusion/installation location (setting out, depth range, or depth)	As relevant execution standard or as set out by the National Authority	
Deviation with depth	As relevant execution standard or as set out by the National Authority	

11.2.2.2 Zone of influence

(1) <REQ> EN 1997-1, 4.1.2.1 shall apply.

11.2.3 Actions and environmental influences

11.2.3.1 General

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

(2) <REQ> Actions for ground improvement shall include but not be 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;

- discrete inclusion imperfections that result in additional bending moment or shear loads;
- loading due to thermal deformations of the inclusion or zone or surrounding ground;
- the potential impact of ground improvement execution on previously constructed ground improvement;
- bulging of Class BI discrete inclusions;
- the initial expansion or eventual contraction of a ground improvement zone due to the heat of hydration of cementitious materials or freeze/thaw of frozen ground improvement bodies.

NOTE 288. Some types 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.

(3) <RCM> The ground improvement method should be selected taking into account the following influences:

- 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.

11.2.3.2 Cyclic and dynamic actions

(1) <REQ> EN 1997-1, 4.31.3 shall apply.

(2) <RCM> EN 1997-3, 6.2.3.3 should apply if relevant.

11.2.3.3 Actions due to ground displacement

PT6 will harmonize the text later by considering interactions with Clause 6.

(1) <REQ> The adverse effects of vertical and horizontal ground movement on ground improvement inclusions shall be considered.

(2) <RCM> 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.

11.2.3.4 Downdrag

PT6 will harmonize the text later by considering interactions with Clause 6.

(1) <REQ> For Class II ground improvement, downdrag shall be considered at the perimeter of the improved ground zone in situations.

(2) <REQ> The calculation of the maximum drag force shall take account of the shear resistance at the interface between the soil and the ground improvement zone and downward movement of the

ground due to self-weight compression and any surface load around the ground improvement or changes in groundwater levels.

(3) <PER> 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, taking into account any changes in groundwater pressure due to groundwater lowering, consolidation or execution.

(4) <RCM> Interaction calculations should take account of the displacement of the ground improvement relative to the surrounding moving ground.

11.2.3.5 Heave

PT6 will harmonize the text later by considering interactions with Clause 6.

(1) <REQ> Where heave of the ground results in the transfer of a load to the ground improvement then it shall be considered as an action.

(2) <RCM> If ground improvement is subject to heave that results in tensile forces or stresses, the introduction of reinforcement should be considered.

11.2.3.6 Transverse loading

PT6 will harmonize the text later by considering interactions with Clause 6.

(1) <REQ> 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) <RCM> 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) <REQ> 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) <RCM> Extrusion of low strength fine soil around or between discrete ground improvement inclusions should also be considered.

11.2.4 Limit states

11.2.4.1 Ultimate Limit States

(1) <REQ> In addition to EN 1997-1, 8.1, ultimate limit states for AI and AII techniques shall be defined according to:

– slopes, cuttings, and embankments (see also Clause 4);
– spread foundations (see also Clause 5);
– retaining structures (see also Clause 7).

(2) <REQ> In addition to EN 1997-1, 8.1, the following ultimate limit states for BI and BII techniques 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) <RCM> Ultimate limit states other than those given in (1) and (2) should be verified as necessary.

11.2.4.2 Serviceability Limit States

(1) <REQ> In addition to EN 1997-1, 9,serviceability limit states for AI and AII techniques shall be defined according to:

– slopes, cuttings, and embankments (see also Clause 4);
– spread foundations (see also Clause 5);
– retaining structures (see also Clause 7).

(2) <REQ> In addition to EN 1997-1, 9, the following serviceability limit states for BI and BII techniques 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) <RCM> Serviceability limit states other than those given in (1) and (2) should be verified as necessary.

11.2.5 Robustness

(1) <RCM> EN 1997-1, 4.1.4 shall apply.

11.2.6 Ground investigation

11.2.6.1 General

(1) <REQ> EN 1997-2, 5 shall apply.

11.2.6.2 Minimum extent of in situ testing

(1) <REQ> The depth and horizontal extent of the in situ testing shall be sufficient to determine the ground conditions within the zone of influence.

(2) <REQ> For all ground improvement, the minimum depth of in situ testing (d_{min}) below the anticipated depth of any proposed ground improvement shall be determined according to Formula (11.2):

$$d_{\min} = max\left(5\ m; 3D; B_{\mathrm{gi}}\right) \tag{11.2}$$

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) <REQ> For inclusions founded on or in strong homogenous ground, d_{min} shall be determined according to Formula (11.3):

$$d_{\min} = max(2\ m; 3D) \tag{11.3}$$

(4) <PER> The minimum depth of in situ testing for ground improvement by soil replacement may be determined according to formula 11.3 taking D as the depth of replaced soil

(5) <PER> The minimum depth of in situ testing 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.

11.2.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

(2) <REQ> Ground improvement shall be classified into GC 2 or 3.

11.3 Materials

11.3.1 Ground properties

(1) <REQ> EN 1997-2, 7 to 12 shall apply.

(2) <REQ> 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) <RCM>The representative properties of improved ground should be initially selected based on comparable experience, local or general experience.

(2) <REQ> The final representative improved ground properties shall be verified by the testing of ground improvement by either field or laboratory testing of exhumed material incorporated within the ground improvement, comparable experience, calculation, or performance monitoring.

(3) <RCM> In-situ testing of discrete ground improvement should verify the response of the system as a whole either by testing individual inclusions or by testing the system.

(4) <REQ>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) <REQ> The determination of the representative value of the improved ground property shall comply with EN 1997-1, 4.3.2.

11.3.2.3 Class II ground improvement

(1) <RCM> The unconfined compressive strength should be determined on cylindrical samples with a height to diameter ratio of two.

(2) Where the sample dimensions differ, a correction complying with EN 12716, A.1, should be applied.

NOTE 289. The suitability of samples for testing can be evaluated according to EN 12716, Annex B.

(3) <RCM> The stiffness of ground improvement techniques should be determined either from laboratory tests on undisturbed samples, documented correlations, or by monitoring of deformation.

(4) <RCM> During the design, the representative of unconfined compressive strength should be determined according to the following:

- a nominal value of the unconfined compressive strength shall be determined according to engineering judgement and comparable experience;
- a characteristic value of the unconfined compressive strength determined according to EN 1997-1, 4.3.2, using a log-normal distribution assuming a specific value of the mean and the coefficient of variation.

NOTE 290. When assessing characteristic values, the confidence level is 95% unless the National Annex or the relevant authority agreed for a specific project by the relevant parties give another value.

(5) Based on testing, if more than 10 samples are tested, a characteristic value of the unconfined compressive strength determined according to EN 1997-1, 4.3.2, using a log-normal distribution.

NOTE 291. When assessing characteristic values, the confidence level is 90% unless the National Annex or the relevant authority agreed for a specific project by the relevant parties give another value.

(6) <RCM> Based on testing, if fewer than 10 samples are tested, the representative value of unconfined compressive strength $q_{uk,imp}$ should be determined using Equation 11.5.

$$q_{\mathrm{uk,imp}} = k_{\mathrm{field}}\, \mu_{\mathrm{norm}} \tag{11.5}$$

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 292. The value of k_{field} is 0.52 unless the National Annex gives a different value.

(7) <POS> Figure 11.1 can be used to determine the correlation coefficient k_{field} based either on measured normal coefficient of variation or on comparable experience.

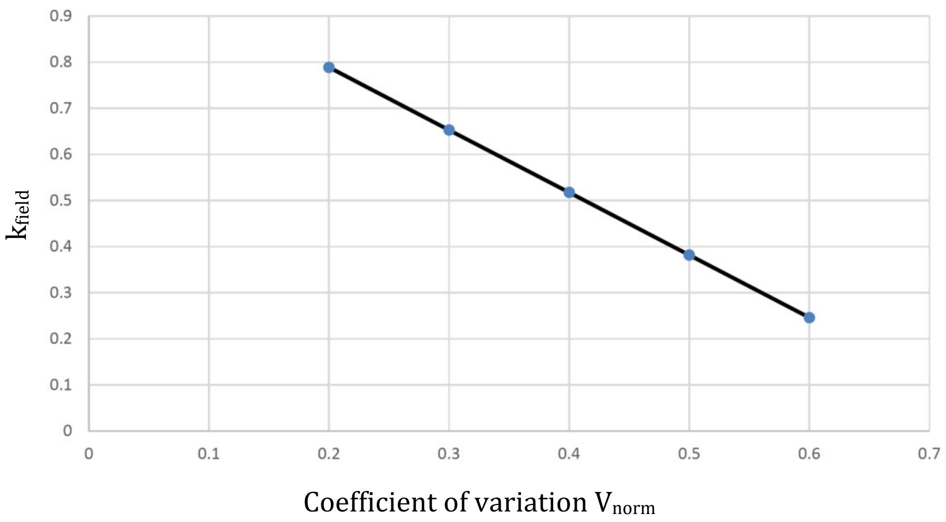


Figure 11.1 – Relationship between correlation coefficient η_{cov} and k_{field}

(8) <REQ> If undisturbed sampling is impractical, the strength shall be determined by documented correlations from other in-situ tests.

(9) <REQ> The selected mean field strength and required coefficient of variation shall be documented in the Geotechnical Design Report.

(10) <REQ> The design value of unconfined compressive strength (q_{ud}) of improved ground shall be determined from Formula (11.4):

$$q_{ud} = \frac{q_{u,rep}}{\gamma_M} = \frac{\eta_t \cdot \eta_c \cdot q_{uk,imp}}{\gamma_M} \tag{11.4}$$

where:

$q_{u,rep}$ 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 reduction 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 293. The value of η_c is 0.85 unless the National Annex gives a different value.

NOTE 294. The value of γ_M is given in EN 1997-1, 8.2.

(11) <RCM> The value of η_t should be determined directly from testing for the specific type of ground improvement.

(12) <RCM> 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 (4) is the time in days since the ground improvement inclusion was installed.

NOTE 295. When t = 28 days, η_t = 1.0.

(13) <REQ> The design strength of concrete, wood, and steel inclusions shall be determined in accordance with ENs 1992-1-1, 1995-1, and 1993-1-1, respectively.

11.3.2.4 Weight density

(1) <RCM> 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) <RCM> For Class II ground improvement that incorporates binder with the ground, the improved or modified weight density should be determined:

- for permeation grouting, by replacing the void content with the injected grout and reassessing density;
- for jet grouting and deep soil mixing, by considering the volume of binder being incorporated within the volume of installed inclusion.

NOTE 296. Density assessment can be impacted by incomplete filling of voids or bleeding within inclusions prior to set.

NOTE 297. Samples can be taken during execution to verify the design assumptions of improved density.

11.3.3 Other materials

(1) <REQ> Material used for Class BI inclusions shall be sufficiently durable and chemically inert so that it does not degrade according to the anticipated ground and groundwater conditions during execution or the design service life.

(2) <RCM> 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.

11.4 Groundwater

(1) <REQ> EN 1997-1, 6 shall apply.

11.5 Geotechnical analysis

11.5.1 General

(1) <RCM> An analysis of the interaction between structure, ground improvement and ground should be carried out to verify that the ultimate and serviceability limit states.

(2) <RCM> The method of analysis selected should take account of the stiffness ratio between discrete inclusions and the surrounding ground.

(3) <RCM> The type and minimum frequency of tests should comply with 11.9.1.

(4) <PER> Static load, other tests, or field trials may be carried out on sacrificial ground improvement inclusions or zones before the design is finalised or on working inclusions that form part of the permanent works to validate the design.

11.5.2 Diffused ground improvement design (AI and AII techniques)

(1) <RCM> As diffused ground improvement techniques, AI and AII techniques should be designed according to:

– slopes, cuttings, and embankments (see also Clause 4);
– spread foundations (see also Clause 5);
– retaining structures (see also Clause 7).

NOTE 298. 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 structures without the use of any ground improvement technique.

NOTE 299. For AI and AII techniques, the main issue is the assessment of the improved ground properties.

NOTE 300. 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) <PER> 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 301. See Annex G.4 for further guidance.

11.5.3 Discrete ground improvement design (BI and BII techniques)

(1) <REQ> 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, especially the head of the rigid inclusions in the load transfer platform taking into account low confining levels;
– a verification of buckling resistance depending on slenderness and soil support parameter (see Annex C).

NOTE 302. 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.

NOTE 303. This design method can be applied when ground improvement relies on inclusions. The overall performance of the improved ground is determined by considering separately the characteristics of the inclusions, the ground and their interaction.

(2) <RCM> The representative total resistance $R_{sys,rep}$ 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 mobilized at a settlement that is compatible with the settlement of the ground improvement system.

(3) <REQ> 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 (see Table 6.1).

(4) <PER> 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 304. 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.

(5) <RCM> Load transfer platforms incorporating tensile elements should be designed in accordance with Clause 9.

(6) <RCM> Load transfer platforms without tensile elements should be designed in accordance with Clause 5.

(7) <RCM> For embankments, when the embankment and the load transfer platform are merged, they should be verified accordingly.

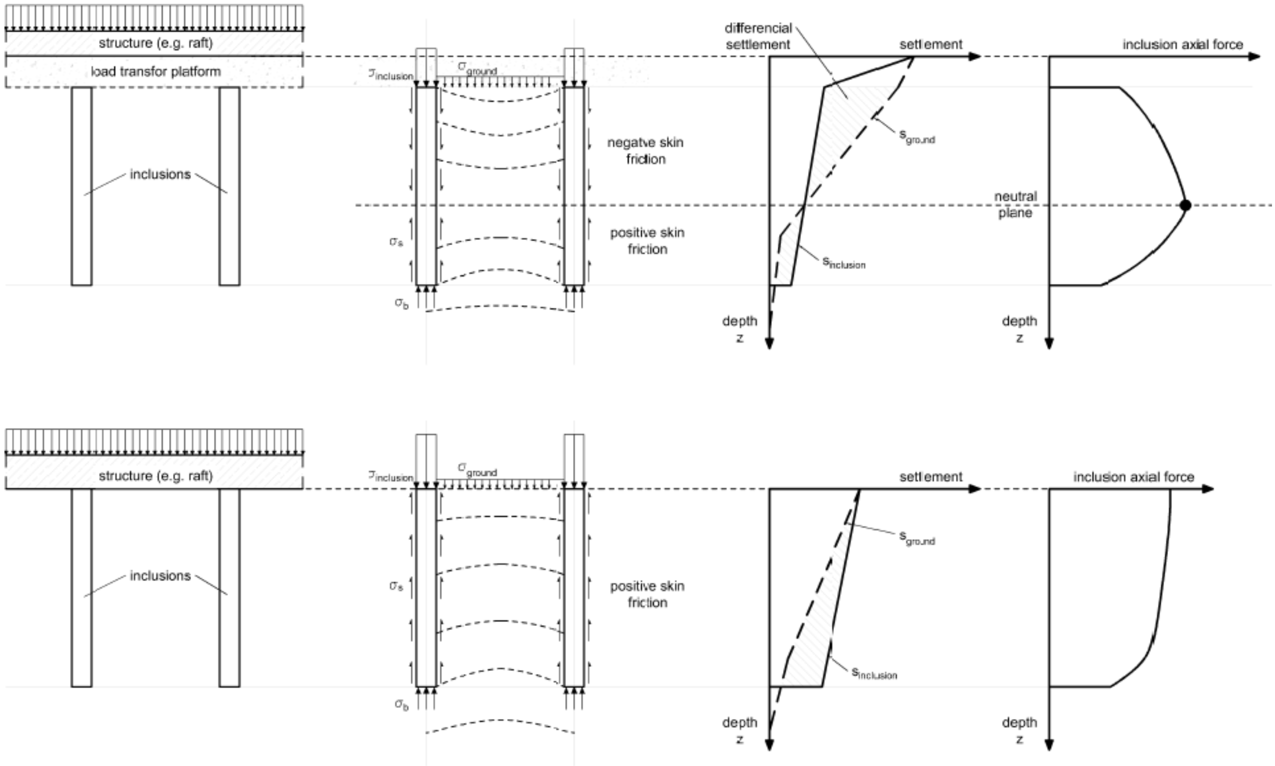


Figure 11.2 – Interaction effects of a ground improvement with rigid inclusions

Figure to be improved

11.6 Ultimate limit states

11.6.1 General

(1) <REQ> For all ground improvement forms, it shall be verified that external stability (including overall stability and sliding, bearing capacity and loss of static equilibrium if relevant), compound stability and internal stability is achieved.

(2) <RCM> The method used to verify ultimate limit states should be selected according to the class and family of ground improvement given in Table 11.1.

NOTE 305. 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 11.3.3.2 and EN 1997-1, 4.3.2. 2. Verify ULS according to the appropriate clause of EN 1997-3	1. Determine properties of non-rigid inclusion according to 11.3.3.2 and EN 1997-1, 4.3.2 2. Verify ULS of the system using separate ground and inclusion properties; 3. Verify ULS according to the appropriate clause of EN 1997-3 4. Verify compression and shear resistance of non-rigid inclusions 5. Determine the strength of the reinforcing element of Geotexile Encased Inclusion according to 9.6.6.
II	1. Determine improved design ground properties according to 11.3.3 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 11.3.3 2. Verify ULS according to 11.2.4 3. Verify structural resistance of the rigid inclusions

11.6.2 Class BII ground improvement

(1) <RCM> The design resistance of Class BII ground improvement ($R_{\mathrm{sys,d}}$) should be determined from Formula (11.7):

$$R_{\mathrm{d,sys}} = \frac{R_{\mathrm{rep,sys}}}{\gamma_{\mathrm{R,sys}}\gamma_{\mathrm{Rd,sys}}} \; or \left(\frac{\sum_i^n R_{\mathrm{ri,i}}}{\gamma_{\mathrm{Rd}}\gamma_{\mathrm{Rc}}} + \frac{R_{\mathrm{g}}}{\gamma_{\mathrm{g}}}\right) \tag{11.7}$$

where:

$R_{\mathrm{rep,sys}}$ is the representative value of the total resistance of the ground improvement system with rigid inclusions;

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

$\gamma_{\mathrm{Rd,sys}}$ is a model factor.

NOTE 306. The value of γR,sys is given in Table 11.4 (NDP) for persistent and transient design situations and accidental design sitations unless the National Annex gives a different value.

NOTE 307. The value of $\gamma_{\mathrm{Rd,sys}}$ is 1.0 unless the National Annex gives a different value.

NOTE 308. The values of γ_{Rd} is given in Clause 6 (see 6.6.2).

NOTE 309. The value of γ_{Rc} is given in Table 6.7.

11.6.3 Partial factors

(1) <REQ> EN 1997-1, 4.4.1(4) shall apply.

NOTE 310. 1. Values of the partial factors for BI and BII techniques are given in Table 11.5 (NDP) for persistent and transient design situations and for 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) 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				
Compressive resistance of diffused ground improvement	Actions and effects-of-actions[1]	γ_F and γ_E	DC1	DC3	Refer to other clauses as appropriate
	Ground properties[2,3]	γ_M	Not factored		
	Total resistance	γ_t	Not factored		
Axial compressive resistance of discrete rigid inclusions	Actions and effects-of-actions[1]	γ_F and γ_E	DC4	DC3	DC1
	Ground properties[2]	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 (1.1)[4]
Transverse resistance of discrete and diffused ground improvement	Actions and effects-of-actions[1]	γ_F and γ_E	DC4	DC3	Not used
	Ground properties[2,3]	γ_M	M1	M3	
	Transverse resistance	γ_{Re}	Not factored		

[1]Values of the partial factors for Design Cases (DCs) 1, 3, and 4 are in EN 1990 Annex A.
[2]Values of the partial factors for Sets M1 and M3 are in EN 1997-1 Annex A.
[3]Including the properties if any improved ground
[4]Values in brackets are given for accidental design situations.

11.7 Serviceability limit states

(2) <REQ> Serviceability limit states of structures founded on ground improvement shall be verified according to Clauses 4, 5, or 6 by calculation or testing.

11.8 Implementation of design during execution and service life

11.8.1 General

(1) <REQ> EN 1997-1, 10 shall apply.

(2) <REQ> The execution of ground improvement techniques not listed above 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) <REQ> Where no execution standard exists, the method of execution control shall be documented in the Geotechnical Design Report.

11.8.2 Inspection

(1) <REQ> EN 1997-1, 10.3 shall apply.

(2) <REQ> Where ground improvement is to be installed within ground that contains natural or artificial chemicals or materials then additional inspection tests shall be carried out to ensure that the required improved ground properties are achieved.

(3) <PER> 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) <REQ> Where materials are to be used for which there is no European testing standard available then inspections 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.

11.8.3 Monitoring

(1) <REQ> EN 1997-1, 10.4 shall apply.

(2) <RCM> Installation parameters for the ground improvement should be monitored and recorded either in real time using bespoke instrumentation or manually by site personnel.

11.8.4 Maintenance

(1) <REQ> EN 1997-1, 10.5 shall apply.

(2) <REQ> 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 and protection of the ground improvement to minimise deterioration and loss of resistance.

NOTE 311. Some ground improvement, for example, jet grouted retaining walls can be exposed to freeze/thaw and so need to be protected.

11.9 Testing

11.9.1 General

(1) <REQ> EN 1997-1, 11 shall apply.

(2) <RCM> The types of testing should be determined according to the ground improvement techniques.

(3) <PER> Typical tests may be the following:

– for AI techniques: in-situ testing to verify density or shear strength by correlation;

- for BI techniques: load test on columns, in-situ testing inside and/or in between columns / testing on incorporated material;
- for AII techniques : testing on collected treated soil samples (unconfined compressive strength for example);
- for BII techniques: load test on columns, load test on rigid inclusion material.

(4) <REQ> The minimum frequency of testing shall be given by the execution standards or agreed by the relevant parties for a specific project.

(5) <REQ> The minimum frequency of inspection tests shall be as given in Table 11.5 (NDP) unless otherwise specified in the execution standard, by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

NOTE 312. The minimum frequency of testing is given in Table 11.5 (NDP), unless than National Annex gives a different minimum frequency.

Table 11.5 (NDP) – Testing frequency for ground improvement (control tests)

Ground Improvement Class	Volume of ground improvement zone (m^3)	No. of tests*
I	< 500	Minimum 4
	≥ 500	1 test per $125 m^2$
II	**No of Inclusions**	
	≤ 600	1 in 100
	601 to 2000	6 + 1 additional per 200 (maximum 13)
	> 2000	13 + 1 additional per 250
*Type of control testing as required by the relevant execution standard or as specified by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties. *For structures in Geotechnical Category 3, the number of tests is 20% greater		

11.9.2 Investigation tests

(1) <RCM> Investigation tests should be zone loading tests, dummy footings (or skip tests) or extraction and testing of samples.

(2) <RCM> For deep soil mixing, the preparation and testing of samples should be based on the Japanese Geotechnical Society Standard JGS 0821:2009 unless otherwise specified by the relevant authority or for a specific project with the relevant parties.

11.10 Reporting

(1) <REQ> EN 1997-1, 12 shall apply.

12 Groundwater controll

12.1 Scope and field of application

<Drafting note> This table of content has been slightly adjusted for drafting purposes. At a later stage, a decision will be taken if it should be kept or this Clause should follow the same outline as the other Clauses in EN 1997-3. The decision is linked to TC250 drafting instructions.>

12.2 Scope and field of application

12.2.1 General

(1) <REQ> This clause shall apply to groundwater control of the ground connected or related to the geotechnical structure involved.

NOTE 313. Geotechnical structures are, but not limited to: dams, levees, embankments, slopes, cuttings, excavations, reinforced fill structures, retaining structures and foundations.

NOTE 314. For embankments, slopes, cuttings and excavations Clause 4 applies.

NOTE 315. For spread foundations Clause 5 applies.

NOTE 316. For retaining walls Clause 7 applies.

NOTE 317. For reinforced fill structures Clause 9 applies.

(2) <REQ> This clause shall not apply to the verification of water retention by dams and levees.

NOTE 318. For these structures additional provisions are needed.

NOTE 319. Methods of assessing critical hydraulic gradients are given in The International Levee Handbook, CIRIA Report C731 (2013).

(3) <REQ> 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.

(4) <REQ> In addition to (3) this clause shall apply to verification of the limit states, durability and robustness of the groundwater control measure itself.

(5) <REQ> In addition to EN 1997-1, the appropriate clauses in 4, 5, 6, 7 and 9 for the involved geotechnical structures shall apply.

NOTE 320. This is valid for all clauses in this Clause 12, and therefore not repeated further.

(6) <PER> Groundwater control measures may be divided in three main groups:

– grouting of the ground to reduce the hydraulic conductivity;

– installation of drainage systems to control and/or decrease the groundwater and/or surface water flow;

– vertical impermeable barriers (cut-off walls) to prevent and/or cut off groundwater flow.

NOTE 321. Ground improvement techniques to increase strength, stiffness, consolidation and consolidation-rate of the ground is covered by clause 11.

(7) <PER> Measures from different groups may be combined to achieve the needed ground water control.

12.2.2 Grouting (reduction of hydraulic conductivity)

(1) <PER>The following may be reasons to utilise groundwater control with grouting to reduce hydraulic conductivity, but is not limited to:

- create a barrier in any groundwater flow under, around or aside a geotechnical structure;
- reduce water ingress into the excavation;
- reduce water egress out of 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.

12.2.3 Drainage system and pumping

(1) <PER>The following may be reasons to utilise groundwater control with drainage system, but is not limited to:

- create controlled surface and groundwater 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 pressure on the geotechnical structure.

12.2.4 Vertical impermeable barriers (cut-off walls)

(1) <PER>The following may be reasons to utilise groundwater cut off, but is 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.

12.3 Basis of design

12.3.1 Design situation

(1) <REQ> EN 1997-1, 4.2.2 shall apply.

(2) <REQ> The selection of measures for groundwater control shall be determined according to its purpose for the geotechnical structure.

(3) <REQ> The design and verification of the geotechnical structure, including the implicated groundwater control measures, shall be conducted in accordance with the clause for the geotechnical structure involved.

(4) <REQ> In addition to (2) and (3), the design situations for groundwater control measures shall include, but 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 measure.

12.3.2 Geometrical and hydrogeological properties

(1) <REQ> For geometrical properties EN 1997-1, 4.3.3 shall apply.

(2) <REQ> For the zone of influence EN 1997-1, 4.1.2.1 shall apply.

(3) <REQ> For groundwater properties EN 1997-1, 6.1 shall apply.

(4) <REQ> For hydrogeological properties EN 1997-2 shall apply.

12.3.3 Actions and environmental influences

(1) <REQ> EN 1997-1, 4.3.1 shall apply.

(2) <REQ> The limiting design value of the relevant 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; or

– limiting values to avoid impact in the zone of influence.

12.3.4 Limit States

(1) <REQ> EN 1997-1, 8 and 9 shall apply.

(2) <REQ> In addition to EN 1997-1, 8, the following ultimate limit states, potentially caused by the groundwater control, shall be verified, as relevant:

– failure of the ground due to excessive grout pressure;
– failure of ground outside the barrier due to increase in groundwater pressure, as a result of cut-off groundwater flow,
– structural capacity of any vertical cut-off wall,

(3) <REQ> In addition to EN 1997-1, 9, the following serviceability limit state, caused by the groundwater control, shall be verified;

– verification of the limiting design value of the relevant geotechnical structure's serviceability criterion set on groundwater control.

(4) <REQ> If grouting is applied, this clause shall apply to verify hydraulic conductivity of the ground inside the zone of influence of the geotechnical structure involved.

(5) <RCM> In addition to EN 1997-1, 9, the following serviceability limit states, potential caused by the groundwater control, should be verified:

- deformations of adjacent geotechnical structures and utilities due to lowering of groundwater;
- flooding of adjacent geotechnical structures and utilities due to installation of barriers, as a result of cut-off groundwater flow

(6) <RCM> Limit states other than those given in (2), (3), (4) and (5) should be verified as necessary.

12.3.5 Robustness

(1) <REQ> EN 1997-1, 4.1.4 shall apply.

12.3.6 Ground investigation

(1) <REQ> EN 1997-2, 5 shall apply.

(2) <REQ> Ground investigations shall provide results to identify groundwater, hydrogeological conditions and hydraulic properties.

(3) <RCM> Hydrogeological investigations by water measurements should be applied to identify hydrogeological conditions and of the rock mass.

(4) <RCM> The zone of ground, into which groundwater control measure extends, should be included in the ground investigation.

12.3.7 Geotechnical reliability

(1) <REQ> EN 1997-1, 4.1.2 shall apply.

12.4 Material

12.4.1 Ground

(1) <REQ> EN 1997-1, 5.1 and EN 1997-2 shall apply.

12.4.2 Grouting materials

(1) <REQ> For grout EN 1997-1, 5.4 shall apply.

(2) <REQ> For silica-fume, chemical grouting and other materials than grout, 10.3.5 shall apply.

12.4.3 Drainage systems and pumps

(1) <REQ> For geosynthetic drainage systems EN 1997-1, 5.3 shall apply.

(2) <REQ> The material for the drainage system to be selected shall be defined in accordance with ground properties and environmental demands.

NOTE 322. Typical materials are drainage pipes, pumps.

(3) <PER> Drainage systems may also be non-material.

NOTE 323. Typical examples of these are ditches, wells and bore holes.

12.4.4 Vertical impermeable barriers (cut-off walls)

(1) <REQ> For geomembrane, geosynthetic or plastic barriers ISO/TS 13434 shall apply,

(2) <RCM> For slurry walls UFGS 02 35 27, Soil-Bentonite (SB) Slurry Trench, should apply.

(3) <REQ> For steel sheet pile barriers EN 1993-1 and EN 1993-5 shall apply.

12.4.5 Other materials

(1) <REQ> Materials other than specified 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.

12.5 Groundwater

(1) <REQ> EN 1997-1, 6 shall apply.

12.6 Grouting (reduction of hydraulic conductivity)

12.6.1 Geotechnical analysis

12.6.1.1 General

(1) <PER> Grouting may be selected to fulfil the limiting design value of groundwater pressure or groundwater flow as specified in 12.2.4.

(2) <REQ> The required reduction of the hydraulic conductivity, limitation in changes of groundwater level, leakage, flow or pressure shall be defined in the first step of the design in accordance with 12.2.4 (2).

(3) <REQ> In addition to this Clause, for design considerations of grouting EN 12715 shall apply.

12.6.1.2 Material specification

(1) <REQ> The grouting material to be selected shall be defined in accordance with ground properties and environmental demands.

(2) <REQ> The grouting material shall comply with 12.3.2.

(3) <REQ> The grout recipe shall be defined taking into account but not limited to:

- Time related properties, related to mixing, hydration and hardening;
- Ratios of material and water components;
- Rheological properties, such as liquid limits and viscosity;
- Penetration related properties, such as grain size vs. apertures;
- Pressures of grouting and groundwater;
- Salinity of groundwater;

– Necessity and types of additives;

12.6.1.3 Design specification

(70) <REQ> The grouting technique and sequence shall be integrated into the design and verification of grouting.

(71) <REQ> The necessity of grouting shall be defined in correlation with the conditions and the requirements.

NOTE 324. Possible outcomes on the necessity are systematic grouting, grouting only when trigger values are exceeded, or leaving un-grouted.

(72) <REQ> The design selection of grouting shall be defined in correlation with the conditions and the requirements of the geotechnical structure.

(73) <RCM> The execution specification should include on-site verification and stop-criteria, based on pressure, flow or mass regulation.

NOTE 325. The use of on-site verification and stop-criteria avoids excessive grouting, insufficient grouting and grout mass propagation outside the intended grouted rock mass.

(74) <REQ> The design of rock grouting shall take into account other possible measures, structures or elements in the ground, that affects grouting results.

NOTE 326. Typically, rock bolts and anchors installed after grouting cause renewed leakages, when punching through the un-grouted zone or discontinuities.

(75) <REQ> The design, verification and execution specification of grouting shall include, but is not limited to:

– 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;

(76) <PER> The selection of appropriate grouting may include multiple different types of grouting materials.

12.6.2 Ultimate limit states

(1) <REQ> The ULS for the geotechnical structure involved shall apply according the appropriate clause.

(2) <REQ> For uplift, hydraulic heave, internal erosion and piping EN 1997-1, 8.1.4 shall apply.

12.6.3 Serviceability limit states

(1) <REQ> For the verification of the limiting design value bygrouting EN 1997-1, 4.4.1 (5) shall apply.

(2) <PER> For grouting the limiting design value, defined in 12.2.4, 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 Lugeon value;
- limiting value of groundwater flow;
- limiting value of hydraulic conductivity.

(3) <REQ> Inspection and monitoring shall be used to verify the compliance with (1) during the design service life of the ground water control system.

(4) <REQ> For hydrogeological aspects in the serviceability limit states EN 1997-1, 9.4 shall apply.

(5) <RCM> The Observational Method should at least be one of the verification methods.

12.7 Drainage system and pumping

12.7.1 Geotechnical analysis

(1) <PER> A drainage system may be selected to fulfil the limiting design value of groundwater pressure or groundwater flow as specified 12.2.4.

(2) <REQ> The selection of an appropriate drainage 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.

(3) <RCM> The necessity for the use of pumps should be determined.

(4) <RCM> As appropriate, the pumping capacity requirements should be established.

(5) <REQ> The verification of the appropriateness of the selected drainage system shall include;

- quantity and pressure of any discharge;
- chemical content of any discharge.

(6) <REQ>Unless it can be demonstrated by comparable experience and assessment of any water discharge that the drainage system will operate adequately without maintenance, a Maintenance Plan shall be specified.

(7) <REQ> In addition to (5) the following shall be included, if drainage system without possibility of inspection and maintenance are used:

– consequence of the failure of the drainage system, with respect to both safety and cost of failure;

– it shall be demonstrated, both by comparable experience and by assessment of any water discharge, that the drainage system will operate adequately without maintenance.

12.7.2 Ultimate limit states

(1) <REQ> The ULS for the geotechnical structure involved shall apply according the appropriate clause.

12.7.3 Serviceability limit states

(1) <REQ> For the verification of the limiting design value by drainage systems and pumping EN 1997-1, 4.4.1 (5) shall apply.

(2) <REQ> For hydrogeological aspects in the serviceability limit states EN 1997-1, 9.4 shall apply.

(3) <PER> For drainage systems the limiting design value, defined in 12.2.4, may be expressed in terms of:

– limiting values of groundwater levels at different locations within the zone-of-influence;

– limiting value of groundwater flow.

(4) <REQ> Inspection and monitoring shall be used to verify the compliance with (1) during the design service life of the ground water control system.

(5) <REQ> The SLS for the geotechnical structure involved shall apply according the appropriate clause.

12.8 Vertical impermeable barriers (cut-off walls)

12.8.1 Geotechnical analysis

(1) <PER> A barrier (cut-off wall) may be selected to fulfil the limit design value of groundwater pressure or groundwater flow as specified 12.2.4.

(2) <REQ> The required barrier (cut-off wall) on groundwater level, leakage, flow or pressure, or the necessity and extent of it shall be defined in the first step of the design.

(3) <REQ> The selection of an appropriate cut-off wall, to obtain the design groundwater pressure and groundwater flow as specified the appropriate clause., shall be assessed from the Geotechnical Design model.

(4) <REQ> The selection of an appropriate impermeable 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.

12.8.2 Ultimate limit states

(1) <REQ> The ULS for the geotechnical structure involved shall apply according the appropriate clause.

(2) <REQ>Verification of any structural resistance of the cut-off wall shall comply with clause 7.

12.8.3 Serviceability limit states

(1) <REQ> For the verification of the limiting design value by vertical impermeable barriers EN 1997-1, 4.4.1 (5) shall apply.

(2) <REQ> For hydrogeological aspects in the serviceability limit states EN 1997-1, 9.4 shall apply.

(3) <PER> For impermeable barriers the limiting design value, defined in 12.2.4, may be expressed in terms of:

– limiting values of groundwater levels at different locations within the zone-of-influence;

– limiting value of groundwater flow;

– limiting value of leakage under or around the barrier.

(4) <REQ> Inspection and monitoring shall be used to verify the compliance with (1) during the design service life of the ground water control system.

(5) <REQ> The SLS for the geotechnical structure involved shall apply according the appropriate clause.

12.9 Implementation of design during execution and service life

12.9.1 General

(1) <REQ> The relative 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) <REQ> For the application of the Observational Method during execution EN 1997-1, 10.6 shall apply.

(3) <REQ> For the execution of grouting EN 12715 shall apply.

(4) <REQ> For the execution of vertical drainage EN 15237 shall comply.

12.9.2 Supervision

(1) <REQ> EN 1997-1, 10.2 shall apply.

(2) <REQ> Supervision shall include the use of grouting equipment in relation with the design, demands and assumptions used in the design.

(3) <REQ> Supervision shall include check of proper installation of groundwater control system and functionality control of it.

12.9.3 Inspection

12.9.3.1 General

(1) <REQ> EN 1997-1, 10.3 shall apply.

(2) <REQ> Inspection shall include the ground or groundwater conditions on site in relation with the assumptions made in the Geotechnical Design Model.

(3) <RCM> Ground water levels should be continuously measured.

(4) <REQ> If the ground or groundwater conditions are found to be significantly different than assumed in the Geotechnical Design Model, the design shall be revised accordingly.

NOTE 327. 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, as required
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 experience from the vicinity of the site - Any other relevant evidence

(5) <RCM>The following items should be considered in the inspection in relation to groundwater:

- 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.

12.9.3.2 Grouting

(1) <REQ> The inspection shall be in compliance with 12.5.3.

(2) <RCM> As appropriate, the ingress, flow and/or egress of water should be measured.

(3) <RCM> As appropriate, the reduction in hydraulic conductivity water should be measured or derived from other measurements.

(4) <REQ> Inspection shall include the compliance of grouting sequencing with the design, demands and assumptions used in the design.

12.9.3.3 Drainage systems and pumping

(1) <REQ> The inspection shall be in compliance with 12.6.3.

(2) <REQ> In addition to EN 1997-1, 10, the Inspection Plan specified should also include:

- 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

- flow measurements from any drains;

- effectiveness of any sub-horizontal borehole drains.

(3) <RCM> In addition to (2), the Inspection Plan should include:

- chemical composition of groundwater;

- durability of the reinforcing element.

12.9.3.4 Vertical impermeable barriers (cut-off walls)

(1) <REQ> The inspection shall be in compliance with 12.7.3.

(2) <RCM> The groundwater levels, absence of flow ingress, and/or egress of water on both sides of the barrier should be measured prior to installation.

12.9.4 Monitoring

12.9.4.1 General

(1) <REQ> EN 1997-1, 10.4 shall apply.

(2) <RCM> The results of monitoring should define the necessity and steer the implementation of further groundwater control.

(3) <RCM> Groundwater level monitoring should be conducted continuously or semi-continuously.

(4) <RCM> Groundwater level monitoring should be conducted prior, during and after groundwater control works and works affecting groundwater levels.

12.9.4.2 Grouting

(1) <REQ> The monitoring shall be in compliance with 12.5.3.

(2) <REQ> Grouting time, pressures, flow and mass intake shall be monitored during grouting.

(3) <REQ> Groundwater levels and changes herein should be monitored.

(4) <RCM> Control measures should be conducted on the hydraulic properties after grouting.

(5) <RCM> For work in freezing conditions the air and rock temperature should be monitored.

(6) <RCM> In case of freezing conditions heating or frost prevention measures should be implemented.

12.9.4.3 Drainage systems and pumping

(1) <REQ> The monitoring shall be in compliance with 12.6.3.

(2) <RCM> Piezometric levels under buildings or in adjoining areas should be monitored, especially if deep drainage or permanent dewatering systems are installed or if deep basements are constructed.

(3) <REQ> As appropriate, when pumps are installed, the pumping amounts shall be monitored.

(4) <REQ> As appropriate, effects of dewatering operations on the groundwater table shall be monitored.

12.9.4.4 Vertical impermeable barriers (cut-off walls)

(1) <REQ> The monitoring shall be in compliance with 12.7.3.

(2) <RCM> The groundwater levels, absence of flow ingress, and/or egress of water on both sides of the barrier should be monitored during installation and use.

12.10 Testing

12.10.1 General

(1) <REQ> EN 1997-1, 11 shall apply.

12.10.2 Grouting

(1) <REQ> Testing of grout material properties shall be conducted prior to start of grouting.

(2) <RCM> 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.

(3) <RCM> Testing of grouting should be conducted prior to start of grouting and after grouting.

12.10.3 Drainage system and pumping

(1) <RCM> Pumps should be tested prior to installation.

(2) <RCM> Drainage systems should be rinsed or flushed after installation.

12.11 Reporting

(1) <REQ> EN 1997-1, 12 shall apply.

Annex A

(informative)

Slopes, cuttings, and embankments

A.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clauses 4 regarding slopes, cuttings, and embankments.

NOTE 328. 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

(2) This Annex covers calculation methods for the stability of soil and fill slopes and rock masses.

A.3 Calculation models for analysing the stability of soil and fill slopes

(1) <RCM> A calculation method for analysing the stability of soils slopes should only be used if it is appropriate for the Ground Model, potential failure surface, and loading conditions.

NOTE 329. Table A.1 provides a non-exhaustive list of calculation models for soil slopes based on limiting equilibrium.

NOTE 330. Procedures for numerical models are given in EN 1997-1, 8.2.

(2) <PER> Three-dimensional effects may be considered when using a two-dimensional calculation method, provided the adjustment is on the safe side and the method is validated.

(3) <RCM> When choosing a calculation model for analysing the stability of soil slopes, the following should be included in the Geotechnical Design Model, as appropriate:

– weight density determined using the single source principle [see EN 1990, 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 of both;
– creep deformations due to shear;
– type of anticipated failure;
– possibility of progressive failure along the slip surface (strain compatibility);
– external actions and their duration and direction;
– use of stabilizing measures; and
– adjacent or intersecting structures.

Table A.1 – Calculation methods for analysing the stability of soil and fill slopes

<table>
<tr><th colspan="2">Method</th><th>Type of method [a,b]</th><th>Special design conditions/limitations</th><th>Comments and assumptions</th></tr>
<tr><td>1</td><td>Bishop (simplified and rigorous)</td><td>Slices, circular arc</td><td>Not recommended with external horizontal loads</td><td>Simplified ignores interslice shear forces when interslice forces are horizontal</td></tr>
<tr><td>2</td><td>Generalized limit equilibrium</td><td>Slices, any shape of surface</td><td rowspan="4">Applicable with all slope geometries and soil profiles</td><td>---</td></tr>
<tr><td>3</td><td>Janbu generalized (modified)</td><td rowspan="3">Slices, circular arc, non-circular, polyline</td><td>Location of interslice normal force is assumed by a line of thrust</td></tr>
<tr><td>4</td><td>Morgenstern-Price</td><td>Direction of interslice forces by variable user function</td></tr>
<tr><td>5</td><td>Spencer</td><td>Constant interslice forces function</td></tr>
<tr><td>6</td><td>Sarma</td><td>Slices, polyline</td><td>Seismic loading, critical acceleration. Static conditions: horizontal load set to zero</td><td>Can include non-vertical slices and multi-wedge failure mechanisms</td></tr>
<tr><td>7</td><td>Kinematical approach of limit analysis</td><td>Multiple body, blocks, circular, planar or logarithmic spiral</td><td>---</td><td>Based on the compatibility of velocity fields, no consideration to stress diffusion</td></tr>
<tr><td>8</td><td>Block/wedge method</td><td>Multiple body, polyline</td><td>Pre-defined planar failure surface. Divided into three segments</td><td>Earth-pressure can be used as driving and resisting force. No moment equilibrium</td></tr>
<tr><td>9</td><td>Multiple wedge method</td><td>Multiple body, blocks, wedges, plane surfaces</td><td>---</td><td rowspan="3">No moment equilibrium.</td></tr>
<tr><td>10</td><td>Infinite slope</td><td rowspan="2">Single body, plane surface</td><td>Long shallow slopes</td></tr>
<tr><td>11</td><td>Culmann, finite slope</td><td>Steep slopes, drained analysis</td></tr>
<tr><td>12</td><td>Logarithmic spiral</td><td>Single body; logarithmic spiral</td><td>Homogeneous soil, drained analysis</td><td>Satisfies moment and force equilibrium</td></tr>
<tr><td colspan="5">[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]References: 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)</td></tr>
</table>

A.4 Calculation models for analysing the stability of rock masses

(1) <RCM> A calculation method for analysing the stability of rock masses should only be used if it is appropriate for the Ground Model, potential failure surface, and loading conditions.

NOTE 331. Table A.2 provides a non-exhaustive list of calculation models for rock masses based on limiting equilibrium.

(2) <RCM> When choosing a calculation method for analysing the stability of rock masses, the following should be included in the Geotechnical Design Model:

– occurrence and orientation of discontinuities;
– infilled discontinuities;
– seepage and groundwater pressure distribution;
– type 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 masses

No.	Type of failure	Method	Special design conditions/limitations	Comments and assumptions
1	Circular failure	Bishop, Janbu, Morgenstern, Spencer	Blocky or weathered rock mass. Tension crack with or without water	Method of slices, circular (see Table 4.1)
2	Plane failure	---	Tension crack with or without water	Plane surface, blocks
3	Wedge failure	---	Tension crack with or without water	Wedge
4	Block toppling	---	---	Blocks
5	Flexure toppling	---	---	Columns
6	Block-flexure toppling	---	---	Blocks and columns
7	Secondary toppling	---	---	---
8	Rock fall	---	---	---

Annex B

(informative)

Spread foundations

B.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 5 regarding spread foundations.

NOTE 332. 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 Annex covers:

- checklists;
- calculation models for bearing resistance; and
- calculation models for foundation settlement.

B.3 Checklists

(1) <POS> The following features can 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) <POS> The following features of rock can 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) <PER> The undrained bearing resistance factors in Formula (5.4) may be determined from Formula (B. 1):

$$\begin{aligned} N_{\mathrm{cu}} &= \pi + 2 \\ N_{\gamma\mathrm{u}} &= -2\sin\beta \end{aligned} \qquad \text{(B. 1)}$$

where:

β is the slope of the ground surface, downwards from the edge of the foundation.

(2) <PER> The following non-dimensional factors may be used in Formula (5.4):

- base factor b_{cu};
- depth factor d_{cu};
- ground inclination factor g_{cu};
- load inclination factor i_{cu}; and
- shape factor s_{cu}.

(3) <PER> The non-dimensional factors in (2) may be determined from Formula (B. 2):

$$\begin{aligned} b_{\mathrm{cu}} &= 1 - \frac{2\alpha}{\pi + 2} & d_{\mathrm{cu}} &= 1 + 0{,}33\tan^{-1}\left(\frac{D}{B}\right) \\ g_{\mathrm{cu}} &= 1 - \frac{2\beta}{\pi + 2} \geq 0 & i_{\mathrm{cu}} &= \frac{1}{2}\left(1 + \sqrt{1 - \frac{T}{A'c_{\mathrm{u}}}}\right), T \leq A'c_{\mathrm{u}} \\ s_{\mathrm{cu}} &= 1 + 0{,}2\left(\frac{B'}{L'}\right) & &\text{for a rectangular foundation or 1,2 for circular foundation} \end{aligned} \qquad \text{(B. 2)}$$

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 333. 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) <PER> The drained bearing resistance factors in Formula (5.7) may be determined from Formula (B. 3):

$$
\begin{gathered}
N_{\mathrm{q}} = e^{\pi tan\varphi'}\tan^2\left(45 + \frac{\varphi'}{2}\right) \\
N_{\mathrm{c}} = \left(N_{\mathrm{q}} - 1\right)cot\varphi' \\
N_{\gamma\mathrm{u}} = 2\left(N_{\mathrm{q}} + 1\right)\tan\varphi' \text{ for a rough base (i. e. } \delta \geq \varphi'/2)
\end{gathered}
\tag{B. 3}
$$

(77) where:

φ′ is the soil angle of internal shearing resistance;

δ Is the angle of interface friction between the foundation and the ground.

(5) <PER> The following non-dimensional factors may be used in Formula (5.8):

- 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) <PER> The non-dimensional factors in Formula (5.8) may be calculated from Formula (B. 4):

$$
\begin{gathered}
b_{\mathrm{c}} = b_{\mathrm{q}} - \left(\frac{1 - b_{\mathrm{q}}}{N_{\mathrm{c}} \tan \varphi'}\right); b_{\mathrm{q}} = b_{\gamma} = (1 - \alpha \tan \varphi')^2 \\
d_{\mathrm{c}} = d_{\mathrm{q}} - \left(\frac{1 - d_{\mathrm{q}}}{N_{\mathrm{c}} \tan \varphi'}\right); d_{\gamma} = 1 \\
d_{\mathrm{q}} = 1 + 2 \tan \varphi' \,(1 - \sin \varphi')^2 (D/B) \text{ for } D/B \leq 1.0 \\
d_{\mathrm{q}} = 1 + 2 \tan \varphi' \,(1 - \sin \varphi')^2 \tan^{-1}(D/B) \text{ for } D/B > 1.0 \\
g_{\mathrm{c}} = g_{\mathrm{q}} - \left(\frac{1 - g_{\mathrm{q}}}{N_{\mathrm{c}} \tan \varphi'}\right) = \left(\frac{g_{\mathrm{q}} N_{\mathrm{q}} - 1}{N_{\mathrm{q}} - 1}\right); g_{\mathrm{q}} = g_{\gamma} = (1 - \tan \beta)^2 \\
i_{\mathrm{c}} = i_{\mathrm{q}} - \left(\frac{1 - i_{\mathrm{q}}}{N_{\mathrm{c}} \tan \varphi'}\right) = \left(\frac{i_{\mathrm{q}} N_{\mathrm{q}} - 1}{N_{\mathrm{q}} - 1}\right); i_{\mathrm{q}} = \left(1 - \frac{T}{N}\right)^{m}; i_{\gamma} = \left(1 - \frac{T}{N}\right)^{m+1} \\
m = m_B = \frac{2 + (B'/L')}{1 + (B'/L')} \text{ when } T \text{ acts in the direction of } B' \\
m = m_L = \frac{2 + (L'/B')}{1 + (L'/B')} \text{ when } T \text{ acts in the direction of } L' \\
m = m_\theta = m_L \cos^2 \vartheta + m_B \sin^2 \vartheta \text{ for other loading directions} \\
s_{\mathrm{c}} = \left(\frac{s_{\mathrm{q}} N_{\mathrm{q}} - 1}{N_{\mathrm{q}} - 1}\right) \\
s_{\mathrm{q}} = 1 + \left(\frac{B'}{L'}\right) \sin \varphi' \text{ for a rectangular or circular } (B' = L') \text{ foundation} \\
s_{\gamma} = 1 - 0.3 \left(\frac{B'}{L'}\right) \text{ for a rectangular or circular } (B' = L') \text{ foundation}
\end{gathered} \tag{B.4}
$$

where, in addition to the symbols defined for Formula (B.2):

φ' is the soil angle of internal shearing resistance;

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 334. 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) <PER> 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:
  $q' = (\gamma - \gamma_{\mathrm{w}})D$ and $\gamma' = (\gamma - \gamma_{\mathrm{w}})$
- for groundwater level at a depth Dw below the ground surface but above the foundation level:
  $q' = \gamma D_{\mathrm{w}} + (\gamma - \gamma_{\mathrm{w}})(D - D_{\mathrm{w}})$ and $\gamma' = (\gamma - \gamma_{\mathrm{w}})$
- for groundwater at the foundation level:
  $q' = \gamma D$ and $\gamma' = (\gamma - \gamma_{\mathrm{w}})$
- 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

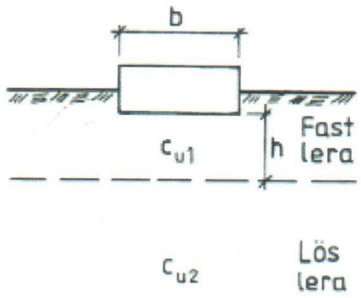


Figure B.1 – Foundation on a stronger layer over a weaker layer

<Figure to be edited: *h* to be changed to D_1>

(1) <PER> The undrained 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 335. This formula assumes that the stress from the foundation spreads at a rate of 1 horizontal to 2 vertical through the stronger layer.

(2) <PER> The drained 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):

$$R_{\mathrm{Nu}} = A\left(1 + \frac{0.2B}{L}\right)(\pi + 2)c_{\mathrm{u2}} + A'\gamma_1' D_1{}^2\left(1 + \frac{2D}{D_1}\right)\left(\frac{K_{\mathrm{ps}} tan\varphi_1{}'}{B}\right) + A'\gamma_1 D$$
$$\lambda = \frac{q_2}{q_1} = \frac{(\pi + 2)c_{\mathrm{u2}}}{0.5\gamma_1{}' B N_\gamma} \qquad \text{(B. 6)}$$

where:

φ'_1 is the drained angle of internal shearing resistance of the 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) <PER> 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.26):

$$R_{\mathrm{N}} = A\,\sigma_{\mathrm{v0}} + A'\,k_{\mathrm{p}}\,p^{*}_{\mathrm{LM,e}} \tag{B.7}$$

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 as

$$p^{*}_{\mathrm{LM,e}} = \sqrt[n]{\prod_{i=1}^{n} p^{*}_{\mathrm{LM}}} = \sqrt[n]{\prod_{i=1}^{n} \left(p_{LM(zi)} - p_{0(zi)}\right)} \tag{B.8}$$

$p_{LM(z)}$ 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 336. The effect of the load inclination is taken into account by considering an additional parameter applied on k_p

NOTE 337. This method is described in NFP 94-261.

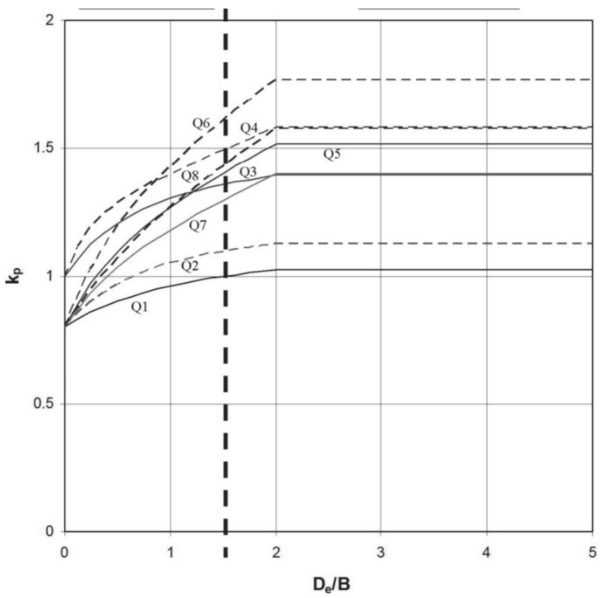


Figure B.2 – Bearing resistance factor k_p versus equivalent embedment depth D_e divided by foundation width B for ground types and foundation shapes given in Table B.2

(2) <RCM> 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 k_p 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) <PER> The total settlement s of a spread foundation on fine or coarse soil may be determined from Formula (B. 9):

$$s = \frac{pB(1-\upsilon^2)I_\mathrm{s}}{E_\mathrm{m}} \tag{B. 9}$$

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 338. NOTE 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 339. 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) <PER> If no reliable settlement results, measured on neighbouring 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) <RCM> 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 may be considered to be linear.

NOTE 340. Great caution is required when using the adjusted elasticity method in the case of non-homogeneous ground.

(4) <PER> In case of a spread foundation on rocks, the design value of E_m may be determined from Formula (B.10).

$$E_{\mathrm{m}} = E_{\mathrm{rm}} \tag{B.10}$$

where:

E_{rm} is the rock mass modulus (see EN1997-2, 9.1.4 (5));

NOTE 341. Note: In literature, there are other expressions for E_{rm} that can be used taking into account their applicability and limitations.

B.8 Calculation model for settlement evaluation based on stress-strain method

(1) <PER> 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) <PER> 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. The values adopted for the stiffness parameters (such as E_{m} and υ) should in this case represent the undrained behaviour with $\upsilon = \upsilon_{\mathrm{u}} = 0.5$.

B.10 Calculation model for settlements caused by consolidation

(1) <PER> 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 may then be used. 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) <PER> 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. However, the rate of consolidation settlement should preferably be obtained using permeability values obtained from in-situ tests-

B.12 Calculation model for settlement evaluation using pressuremeter test results

(1) <PER> The settlement of a spread foundation may be determined from the results of Ménard pressuremeter tests using Formula (B. 11):

$$s = (q - \sigma_{\mathrm{v0}}) \left[\frac{2B_0}{9E_{\mathrm{d}}} \left(\frac{\lambda_{\mathrm{d}} B}{B_0} \right)^{\alpha_r} + \frac{\alpha_{\mathrm{r}} \lambda_{\mathrm{c}} B}{9E_{\mathrm{c}}} \right]$$
$$\frac{1}{E_{\mathrm{d}}} = \frac{0.25}{E_1} + \frac{0.3}{E_2} + \frac{0.25}{E_{3\leftrightarrow 5}} + \frac{0.1}{E_{6\leftrightarrow 8}} + \frac{0.1}{E_{9\leftrightarrow 16}} \tag{B. 11}$$

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;

$E_{\mathrm{i\leftrightarrow j}}$ 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
λ_{d}	1	1,12	1,53	1,78	2,14	2,65
λ_{c}	1	1,1	1,2	1,3	1,4	1,5

Table B.5 – Correlations for deriving the rheological factor α_r for spread foundations

Type of ground	Description	E_M/p_{LM}	α_r
Peat			1,00
Clay	Over-consolidated Normally consolidated Remoulded	> 16 9 - 16 7 - 9	1,00 0,67 0,5'
Silt	Over-consolidated Normally consolidated	> 14 5 - 14	0,67 0,50
Sand	---	> 12 5 - 12	0,50 0,33
Sand and gravel	---	> 10 6 - 10	0,33 0,25
Rock	Highly weathered rock Disintegrated rock mass Highly fractured rock mass Normally fractured, very blocky rock mass	---	0,67 0,33 0,50 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. 11):

$$s = C_1\, C_2 (q - \sigma'_{v0}) \int_0^{z_1} \frac{I_z}{C_3\, E'} dz \qquad \text{(B. 12)}$$

where:

C_1 Is 1 – 0,5 × [$\sigma'_{v0}/(q - \sigma'_{v0})$];

C_2 is 1,2 + 0,2 × 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

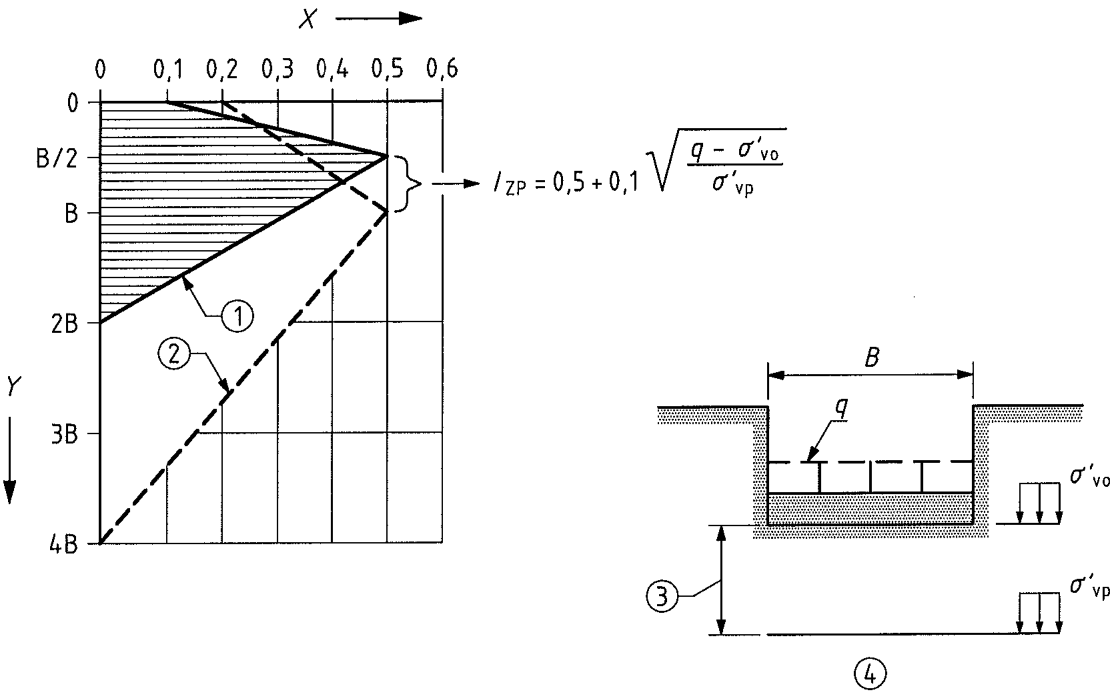
σ'_{v0} is the initial effective vertical stress at the level of the foundation

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 D.1 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 342. This calculation model was 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) <PER> 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):

$$K_s = 5.57\left(\frac{E_f}{E_g}\right)\frac{(1-v_g^2)}{(1-v_f^2)}\left(\frac{B}{L}\right)^{0.5}\left(\frac{D_f}{L}\right)^3 \tag{B. 13}$$

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).

(2) <PER> 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 343. For K_s values between these values the relative deflection and the bending moments in the foundation are a function of K_s.

(3) <PER> 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.

Annex C

(informative)

Piled foundations

C.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 7 regarding piled foundations.

NOTE 344. 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 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 345. Examples of pile types classified according to Table 6.1 are given in Table C.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
	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
Piles not listed above		Steel helical piles Compressed-air driven piles

C.4 Pile shaft resistance based on ground parameters

(1) <PER> Under total stress conditions, the representative value of unit shaft resistance $q_{s,rep}$ in fine soils and fills may be derived from Formula (C. 1):

$$q_{s,rep} = \alpha c_{u,rep} \tag{C. 1}$$

$c_{u,rep}$ is the representative undrained shear strength of the ground;

α is an adhesion factor for piles in soil.

NOTE 346. 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 347. 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) <PER> The value of $q_{s,rep}$ in weak and medium strong rock masses may be derived from Formula (C. 2):

$$\frac{q_{s,rep}}{p_{ref}} = k_1 \left(\frac{q_{u,rep}}{p_{ref}}\right)^{k_2} \tag{C. 2}$$

$q_{u,rep}$ 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 348. 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 349. The value of k_2 typically varies between 0.57 and 0.61 but is commonly taken as 0.5.

(3) <PER> Under effective stress conditions, the value of $q_{s,rep}$ in fine soils, fills, and rock mass may be derived from Formula (C. 3):

$$\overline{q_{s,rep}} = K_s \overline{\sigma'_v} tan\delta_{rep} = \beta \overline{\sigma'_v} \tag{C. 3}$$

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 350. The earth pressure coefficient depends on the strength of the soil, pile type, method of execution, and distance above the pile base.

NOTE 351. 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 352. The value of δ_{rep} is typically taken as φ_{rep} for cast-in-place concrete piles and between $0.67\varphi_{rep}$ and $0.75\varphi_{rep}$ for precast concrete and steel piles, where φ_{rep} is the representative value of the soil's angle of internal friction.

NOTE 353. 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) <PER> Under total stress conditions, the representative value of unit base resistance $q_{b,rep}$ in fine soils and fills may be derived from Formula (C. 4):

$$q_{b,rep} = N_c c_{ub,rep} + \sigma_{vb} \tag{C. 4}$$

$c_{ub,rep}$ 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 354. The value of N_c typically ranges between 6 and 10, although N_c = 9 is commonly used.

(2) <RCM> 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) <PER> The value of $q_{b,rep}$ in very weak and weak fine-grained rock masses may be derived from Formula (C. 5):

$$\frac{q_{b,rep}}{p_{ref}} = k_3 \left(\frac{q_{u,rep}}{p_{ref}}\right)^{k_4} \tag{C. 5}$$

$q_{u,rep}$ 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 355. The value of k_3 typically about 15 for cemented rocks.

NOTE 356. The value of k_4 typically varies between 0.4 and 0.6 but is commonly taken as 0.5.

(4) <PER> Under effective stress conditions, the value of $q_{b,rep}$ in fine soils, fills, and rock mass may be derived from Formula (C. 6):

$$q_{b,rep} = q'_{b,rep} + u_b = N_q \sigma'_{vb} + (\sigma'_{vb} + u_b) \tag{C. 6}$$

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 357. The bearing factor depends on the angle of internal friction of the ground, density index, and vertical effective stress at the pile base.

(5) <RCM> When the self-weight of the pile is not included as a separate action, the term ($\sigma'_{vb} + u_b$) in Formula (C.xx) should be omitted.

C.6 Axial pile resistance based on CPT profiles

(1) <PER> The representative value of unit shaft resistance $q_{s,rep}$ in coarse soils and fills may be derived from Formula (C. 7):

$$q_{s,rep} = c_s q_c \tag{C. 7}$$

q_c is the measured cone resistance;

c_s is an empirical cone factor for shaft resistance.

NOTE 358. 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 359. 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 c_s and c_b 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 c_s 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) <PER> The representative value of unit base resistance $q_{b,rep}$ in coarse soils and fills may be derived from Formula (C.8):

$$q_{b,rep} = 0.5 c_b k_{shape} \left(\frac{q_{c,I,mean} + q_{c,II,mean}}{2} + q_{c,III,mean} \right) < 15 MPa \qquad (C.8)$$

$q_{c,X,mean}$ 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 360. The empirical factor c_b depends on ground and pile types (see Table C.2).

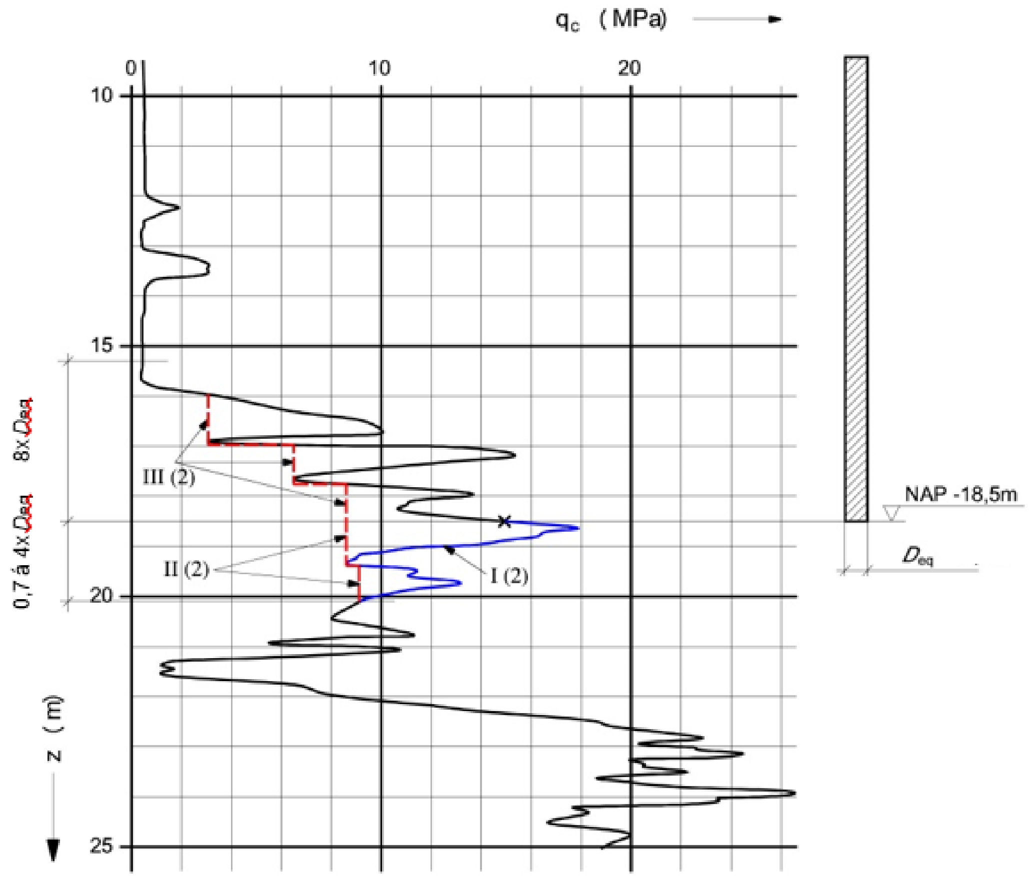


Figure C.1 – Definition of zones I, II, and III

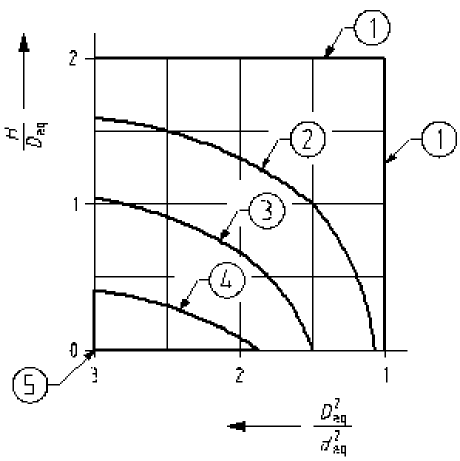


Figure C.2 – Chart to determine k_{shape}

Key: Boundary 1, k_{shape} = 1.0; 2, = 0.9; 3, = 0.8; 4, = 0.7; 5, = 0.6

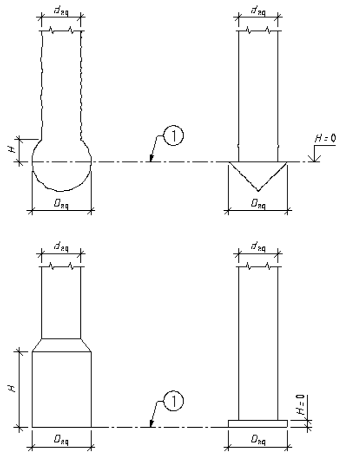


Figure C.3 – Chart to determine *h*

(3) <RCM> 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) <RCM> 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) <PER> The representative value of unit shaft resistance $q_{s,rep}$ may be derived from Formula (C.9):

$$q_{s,rep} = k_{s,PMT}(a_{PMT}p_l + b_{PMT})(1 - e^{-c_{PMT}p_l}) \leq q_{s,max} \quad (C.9)$$

$k_{s,PMT}$ is a dimensionless parameter that depends on pile type and ground type;

p_l is the PMT limit pressure at a depth z; and

a_{PMT}, b_{PMT}, c_{PMT} are parameters that depend on ground type.

NOTE 361. Values of $k_{s,PMT}$ are given in Table C.4 for selected pile types.

NOTE 362. Values of a_{PMT}, b_{PMT}, and c_{PMT} are given in Table C.5 for selected pile types.

NOTE 363. 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 III	200	380	320	320	320

(2) <PER> The representative value of unit base resistance $q_{b,rep}$ may be derived from Formula (C.10):

$$q_{b,rep} = k_{b,PMT} \frac{1}{z_1 + 3z_2} \int_{-z_1}^{3z_2} p_l\,(z) dz \qquad \text{(C.10)}$$

$k_{b,PMT}$ is a dimensionless parameter that depends on pile type and ground type;

$p_l(z)$ is the PMT limit pressure at a depth z; and

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 364. 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 not usually not taken into account						

C.8 Axial pile resistance based on empirical tables

(1) <PER> The representative value of unit shaft resistance $q_{s,rep}$ for bored piles in soils may be determined from Table C.8.

NOTE 365. 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) <RCM> 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_c (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) <RCM> 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	1770-2300	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) <PER> 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.

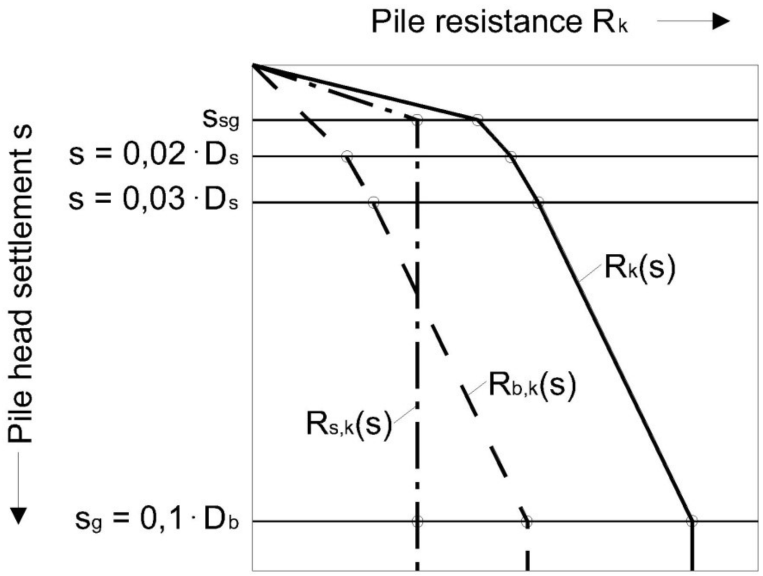


Figure C.4 – Load-displacement curves for bored piles

C.9 Downdrag due to vertical ground movements

C.9.1 General

(1) <RCM> The drag force caused by downdrag should be classified as a permanent action.

NOTE 366. '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 367. 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) <RCM> Potential downdrag should be included in the verification of serviceability limit states.

(3) <RCM> 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

(78) <PER> The calculation model shown in Figure C.5 may be used to calculate the drag force owing to potential downdrag.

NOTE 368. 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.

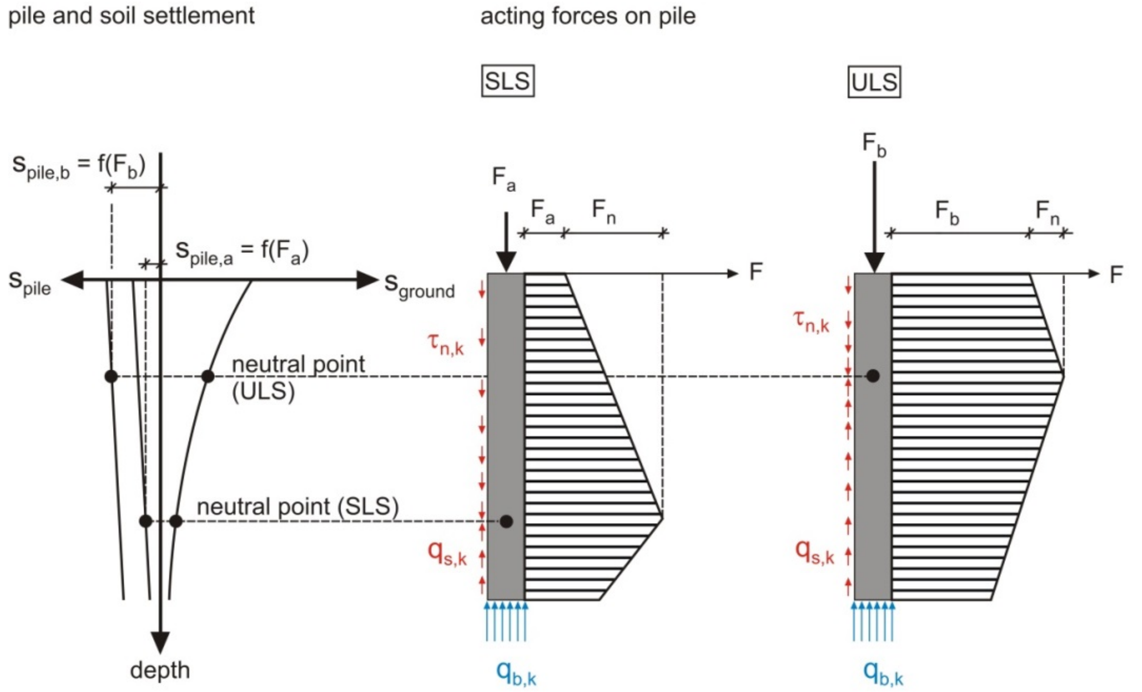


Figure C.5 – Force distribution for assessment of drag force on a pile subject to downdrag

<Drafting Note: Figure to be redrawn and symbols updated to be consistent with Clause 6>

NOTE 369. 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.

(79) <RCM> 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. The ground settlement of should include immediate and primary consolidation, together with potential secondary consolidation (creep).

(80) <PER> 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.

(81) <PER> As an alternative to (2) and (3), the values of s_{ground} and s_{pile} may be determined by an interaction analysis to find the depth of the neutral point L_{dd} where $s_{pile} = s_{ground}$.

(82) <RCM> In addition to EN 1990-1, 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):

$$F_{cd} = \max \begin{cases} \sum_{i\geq 1} G_{k,i} + Q_{k,1} + \sum_{j>1} \psi_{2,j} Q_{k,j} \\ \sum_{i\geq 1} G_{k,i} + D_{rep,SLS} + \sum_{j\geq 1} \psi_{2,j} Q_{k,j} \end{cases} \quad (C.\ 12)$$

where:

$G_{k,i}$ is the i-th characteristic permanent action;

$Q_{k,1}$ is the leading characteristic variable action;

$Q_{k,j}$ is the j-th accompanying characteristic variable action;

$D_{rep,SLS}$ is the representative drag force at the serviceability limit state;

$\psi_{2,j}$ is a combination value for accompanying variable actions.

NOTE 370. Formula (C.12) is a modification of the quasi-permanent combination of actions given in EN 1990-1.

(83) <RCM> In addition to EN 1990-1, 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.6):

$$F_{cd} = \max \begin{cases} \sum_{i\geq 1} \gamma_{G,i} G_{k,i} + \gamma_Q Q_{k,1} + \sum_{j>1} \gamma_{Q,j} \psi_{0,j} Q_{k,j} \\ \sum_{i\geq 1} \gamma_{G,i} G_{k,i} + \gamma_{F,drag} D_{rep,ULS} + \sum_{j\geq 1} \gamma_{Q,j} \psi_{0,j} Q_{k,j} \end{cases} \quad (C.13)$$

where: $D_{rep,ULS}$ is the representative drag force over the depth of ground above the neutral plane under ultimate conditions;

$\gamma_{G,i}$, $\gamma_{Q,j}$ are partial factors applied to permanent and variable actions, respectively;

$\psi_{o,j}$ is a combination factor for accompanying variable actions;

$\gamma_{F,drag}$ is a partial factor dependent on the assumptions regarding ground parameters and the particular method of analysis used to determine $D_{rep,ULS}$.

C.9.3 Simplified approach for calculating downdrag

(1) <PER> For simple cases, approximate approaches may be used.

(2) <PER> 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. In this case the drag force may be disregarded for the verification of the ultimate limit state.

(3) <PER> 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. 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.

(4) <PER> For SLS conditions, the neutral plane may be assumed to be located at the base of the settling fill or soil layer and 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) <PER> 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 371. Possible mechanisms for groups of tension piles in layered soils are illustrated in Figure C.6.

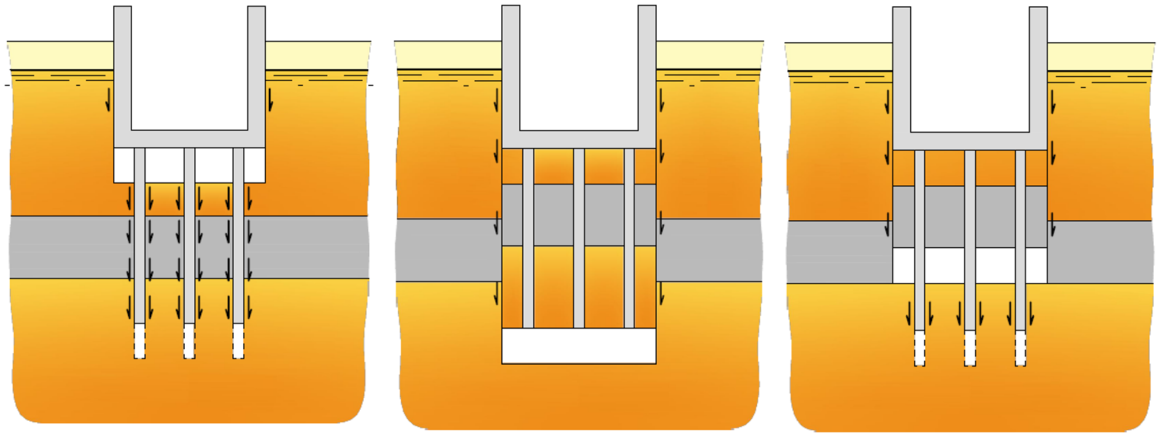


Figure C.6 – Possible mechanisms for groups of tension piles in layered soils

a) Pull out from ground; b) Lift off of a block of soil; c) Combined pull out and lift off

(1) <PER> For the evaluation of the block failure, the representative weight of the soil block surrounding an individual pile $W_{\text{block,rep}}$ (see **Fel! Hittar inte referenskälla.**) may be determined from Formula (C.10):

$$W_{\text{block,rep}} = n_z \left[s_x s_y \left(L - \frac{1}{3} \sqrt{\left(s_x{}^2 \; . \; s_y{}^2\right)} \cot \varphi_{\text{rep}} \right) \right] \eta_z \gamma \tag{C. 14}$$

<Drafting NOTE: PT6 to check formula, since root appears to cancel the s_x^2 and s_y^2 terms>

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.

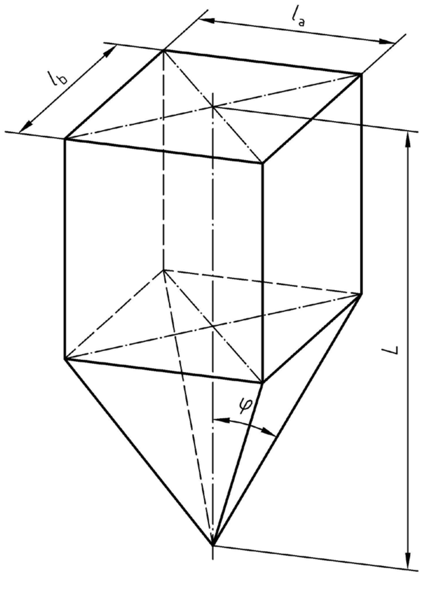


Figure C.7 Block failure of a single pile under tension as part of a pile group

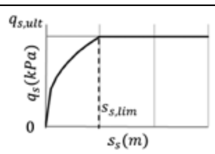
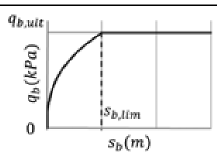
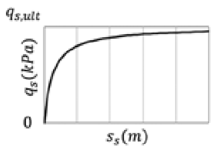
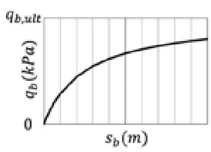
Drafting note: change l_b to s_y and l_a to s_x

C.11 Calculation model for single pile settlement using load transfer functions

(1) <PER> Settlement of single piles may be determined using load transfer functions.

NOTE 372. Examples of load transfer functions are given in Table C.10.

Table C.10 – Example load transfer functions

Curve	Cubic root		Hyperbolic	
	Shaft	Base	Shaft	Base
Shape				
q/q_{ult}	$\sqrt[3]{\dfrac{s_{\mathrm{s}}}{s_{\mathrm{s,max}}}}$	$\sqrt[3]{\dfrac{s_{\mathrm{b}}}{s_{\mathrm{b,max}}}}$	$\dfrac{s_{\mathrm{s}}}{M_{\mathrm{s}}B + s_{\mathrm{s}}}$	$\dfrac{s_{\mathrm{b}}}{M_{\mathrm{b}}B + s_{\mathrm{b}}}$
Deformation parameter	$s_{\mathrm{s,max}}$	$s_{\mathrm{b,max}}$, depending on diameter	M_{s}	M_{b}
Initial slope	∞	∞	$q_{\mathrm{s,ult}}/M_{\mathrm{s}}B$	$q_{\mathrm{b,ult}}/M_{\mathrm{b}}B$

C.12 Calculation model for single pile lateral displacement using load transfer functions

To be prepared by PT6

C.13 Buckling and second order effects

C.13.1 General

(84) <REQ> 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.

(85) <RCM> 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 373. The mobilisation of the ground resistance is dependent on the transversal deflection of the pile (see Figure C.14-1). The ground resistance is limited by different failure mechanisms which are dependent on the subsoil conditions as well as on the foundation geometry.

NOTE 374. The following differential equation is a validated calculation model for buckling of a uniform pile in uniform soil:

$$EI \cdot \frac{d^4y}{dx^4} + C \cdot y + F \cdot \frac{d^2y}{dx^2} = 0 \qquad \text{(C13.1)}$$

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

(86) <REQ> The structural resistance (ULS) and the deformation of piles (SLS) shall be verified in accordance with the structural design codes for concrete structures (EN 1992), steel structures (EN 1993), composite steel and concrete structures (EN 1994) and timber structures (EN 1995).

(87) <REQ> 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

(88) <REQ> The numerical method shall consider the second order moment caused by the transversal deformation during the axial loading of the pile.

NOTE 375. Numerical methods can be used for heterogeneous ground conditions and for piles with non-uniform cross section along the pile length.

NOTE 376. Numerical methods are usually based on Equation C13.1 for which the eigenvalues corresponds to the buckling forces.

(89) <RCM> An initial deformation of the pile according to Clause C13.2.c 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

(90) <PER> The design value of buckling resistance N_{bd} for a fully embedded pile may be determined using Formula 13.2:

$$N_{bd} = \frac{y_f \cdot E_d I \cdot \left(\frac{\pi}{L_{bd}}\right)^2 + p_{fd} \cdot B \cdot \left(\frac{L_{bd}}{\pi}\right)^2}{y_f + e_{0d}} \qquad \text{(C13.2)}$$

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.14-1 and Figure C.14-2

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

(91)<RCM> The design value of the buckling length L_{bd} for a fully embedded pile should be determined using Formula C13.3:

$$L_{bd} = \pi \cdot \sqrt[4]{\frac{E_d I \cdot y_f}{p_{fd} \cdot B}} \qquad \text{(C13.3)}$$

NOTE 377. 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 378. 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

(92)<REQ> An initial curvature of the pile shall be applied, considering production imperfections, installation effects and angular distortion of joints.

(93)<PER> With a given initial curvature R_{0d} the parameter e_{0d} can be determined by Formula C14.4:

$$e_{0d} = \frac{(L_{bd})^2}{8 \cdot R_{0d}} \qquad \text{(C13.4)}$$

(94) <PER> 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 C14.1.

NOTE 379. Smaller values of R_{0d} are likely for piles with B<150 mm and for driven piles encountering boulders or heavily inclined bedrock.

Table C13.1. Design values of pile curvature.

Pile type	R_{0d} **no joints**	R_{0d} **one joint[1]**
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	50

1within the buckling length

<Drafting note> TG should further investigate recommended values, in particular for timber and in situ piles

(95) <RCM> The following addition to e_{0d} 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 380. Classification of cross-sectional types for buckling is found in Table 6.2 in EN 1993-1-1.

<Drafting note> TG should check if these values are compatible with EN 1993 and if an additional reduction of E should be made?

C.13.4 Corresponding second order moment

(96) <REQ> Cross-sectional checks shall be performed according to the structural Eurocodes taking into to account the corresponding second order moment during axial loading.

(97) <PER> For a pile of length equal or greater than L_{bd} according to Eq. (C13.3), the corresponding second order moment during axial loading may be accounted for by using Formula (C.13.5):

$$M_{2Ed} = N_{Ed} \cdot \frac{e_{0d}+y}{2} \quad \text{(C.13.5)}$$

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 C13.1

and $y = \dfrac{N_{Ed} \cdot e_{0d}}{2 \cdot \left(\sqrt{B \left(p_{fd}/y_f\right) E_d\, I}\right) - N_{Ed}}$ (C13.6)

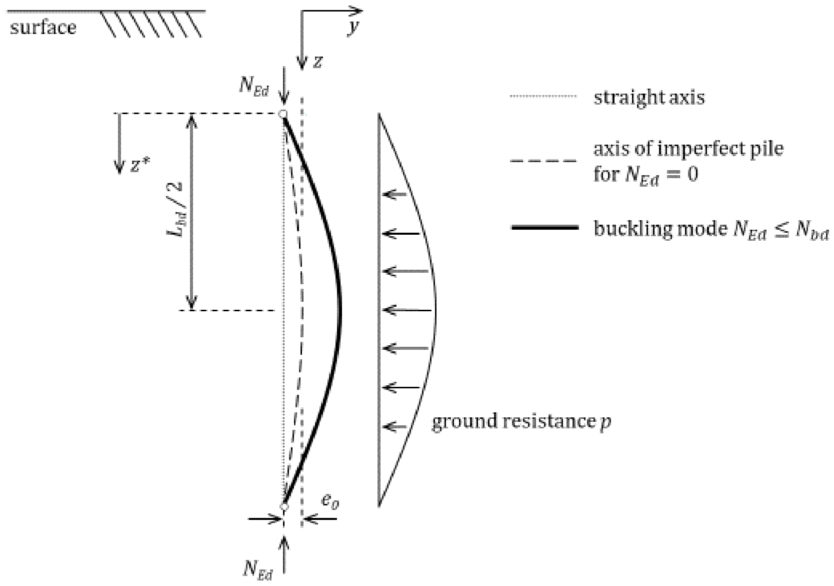


Figure C.13-1 – Transverse deflection of a pile caused by a compressive force.

C.13.5 Mobilisation of ground Resistance

C.13.5.1 General

(98)<PER> For calculating the mobilisation of the ground resistance stress p as function of the transversal deflection in the direction of y a bilinear model as shown in Figure C.13-2 may be used representing the actual non-linear soil resistance. The bilinear equation is defined by Formula C13.7.

$$p = min\left(\frac{p_f}{y_f} \cdot y; p_f\right) \quad \text{(C13.7)}$$

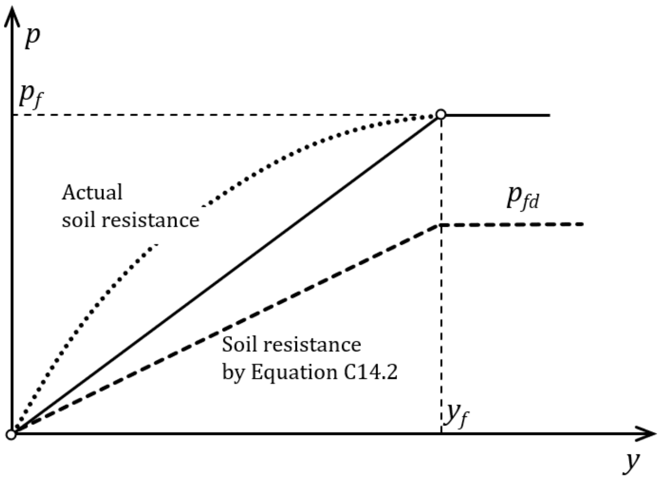


Fig C13-2 – Model of soil resistance as a function of the transversal deflection of a pile.

(2) <PER> A non-linear soil model may be adopted provided it has been validated for the conditions that apply during buckling.

NOTE 381. A non-linear soil model is given in EN ISO 19902:2014-01 and provides information about the soil resistance p at small transversal deflections y.

(3) <REQ> If piles are additionally loaded transversally, e.g. from settling soil, displacement of sloping ground or from structural actions, they shall be verified using second order theory.

(4) <RCM> Seismic loading can result in loss of shear strength in soils susceptible to liquefaction (e.g. saturated sand of loose density and collapsible fine-grained soils). In such design situations, $p_f = 0$ should be assumed.

C.13.5.2 Ground resistance from undrained soil parameters

(1) <PER> 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) <PER> To account for long-term deformations resulting from creep of a highly viscous soil (e.g. low strength clay or organic clay), $p_{fd} = 6 \cdot c_{ud}$ may be applied.

(3) <PER> A weighted average of the undrained soil response may be applied in the case of combined long-term and short-term loads.

(4) <PER> To account for limited soil resistance to close the ground surface $p_{f,d}$ may be determined using formula C14.8:

$$p_{fd} = c_{ud} \cdot \left(2 + \frac{2}{3} \cdot \frac{z}{B}\right) + \sigma'_z \qquad \text{(C13.8)}$$

where:

z is the depth below the ground surface, see Figure C.13-1

σ'_z is the effective vertical stress of the soil at the depth z

C.13.5.3 Ground resistance from drained soil parameters

(99) <PER> For drained soil conditions the ultimate resistance stress may be determined using formula C14.9:

$$p_{fd} = K_{qd} \cdot \sigma'_z + K_{cd} \cdot c'_d \qquad \text{(C13.9)}$$

NOTE 382. NOTE The coefficients K_{qd} and K_{cd} in formula C14.10 may be taken from Figure C13-3.

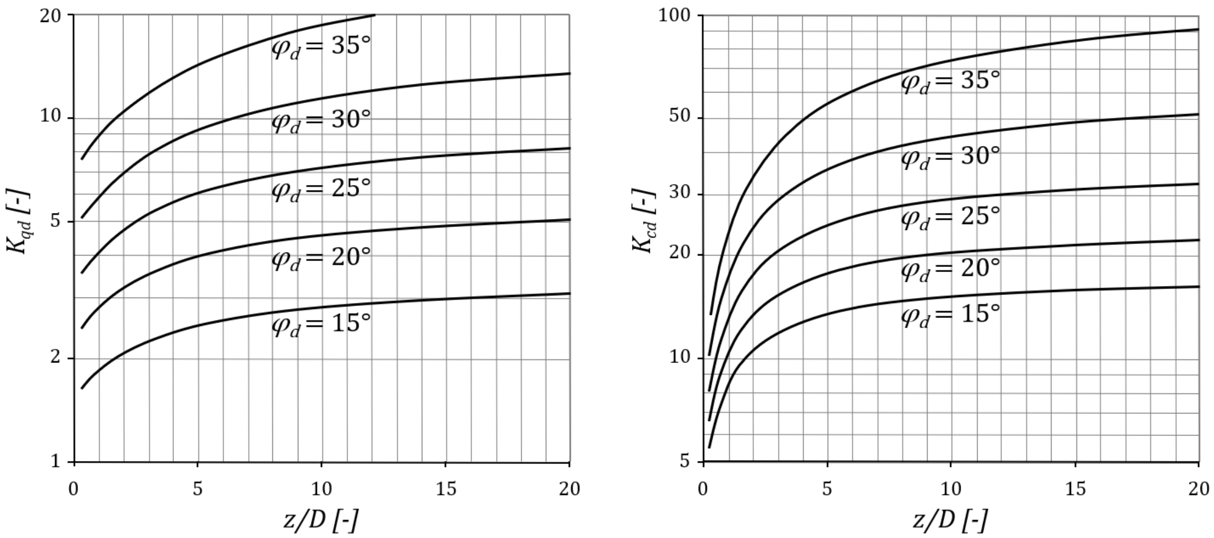


Fig C13-3 – Coefficients K_{qd} and K_{cd} for calculating the ultimate drained soil resistance according to Brinch Hansen (1961).

C.13.5.4 Displacement y_f at ultimate ground resistance

(100) <PER> If a bilinear ground model according to formula (C13.7) 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 C13.1.

Table C13.1. Values of transversal displacement y_f.

Soil conditions	y_f
Coarse soils	$0{,}1 \cdot B$
Fine soils, long-term loading	$0{,}12 \cdot B$
Fine soils, short-term loading	$0{,}05 \cdot B$

(101) <PER> The buckling resistance 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 383. A reduction to the ultimate ground resistance p_f when $y > y_f$ can be calculated assuming equivalent overall ground pressure along the buckling length.

C.13.6 Slenderness of piles

C.13.6.1 General

(102) <RCM> The slenderness ratio λ of a fully embedded pile should be calculated by Formula C.13.10:

$$\lambda = \frac{L_{\mathrm{bd}}}{\sqrt{2} \cdot i} = \frac{L_{bd}}{\sqrt{2 \cdot I/A}} \qquad \text{(C.13.10)}$$

where:

i is the radius of gyration;

L_{bd} is the buckling length calculated according to Equation 14.3;

A is the cross-sectional area of the pile

C.13.6.2 Concrete piles

(103) <RCM> 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 EN 1992-1-1, 5.8.3.1.

(104) <RCM> At least half of the cross-sectional area of an unreinforced pile should be subjected to compression.

C.13.6.3 Steel piles

(105) <RCM> 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 384. A large slenderness ratio is $\lambda \geq 0.2$, and a large axial force is $N_{\mathrm{Ed}}/N_{\mathrm{cr}} \geq 0.04$, according to EN 1993-1-1, 6.3.1.2(4). For piles fully embedded in the ground a large axial force is $N_{\mathrm{Ed}}/N_{\mathrm{cr}} \geq 0.10$ according to EN 1993-5, 5.3.3(3).

NOTE 385. For a fully embedded straight pile the critical buckling load is:

$$N_{cr} = 2 \cdot \sqrt{EI \cdot \frac{p_f \cdot B}{y_f}} \qquad \text{(C.14.11)}$$

C.13.6.4 Composite steel-concrete piles

(106) <RCM> Second order effects should be calculated for composite steel-concrete piles if $N_{\mathrm{Ed}}/N_{\mathrm{cr}} \geq 0.10$.

NOTE 386. N_{cr} is calculated using Equation C.14.11 with the effective flexural stiffness $(EI)_{eff}$ according EN 1994-1-1, 6.7.3.3.

C.13.6.5 Timber piles

(107) <RCM> 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 EN 1995-1-1, 6.3.2.

NOTE 387. The relative stiffness is: $\lambda_{rel} = \frac{\lambda}{\pi} \cdot \sqrt{\frac{f_{c,0,k}}{E_{0,05}}}$ (C13.12)

C.13.7 Partial factors

(108) <RCM> Superior or inferior representative values should be adopted for the ground stiffness and ground strength depending on which is critical.

NOTE 388. High values are sometimes critical when transversal loads, e.g. from settling soil, are present.

(109) <REQ> Partial factors on the ultimate transversal ground resistance p_f derived from ground strength parameters shall be in accordance to set M2 in EN 1997-1, Annex A.

(110) <RCM> A partial factor of γ_{pf} = 1,4·K_M should be applied to a measured value of ultimate transversal ground resistance, p_f.

Bibliography

Brinch Hansen, J., (1961): The Ultimate Resistance of Rigid Piles Against Transversal Forces, Geoteknisk Institut, Copenhagen, Bulletin No. 12

See the annex in preparation by the ad-hoc group (document sent later)

Annex D

(informative)

Retaining structures

D.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in EN 1997-3, 7.

NOTE 389. 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 Annex covers:

- limit values of earth pressures;
- at rest values of earth pressures;
- compaction effects;
- general principles and application of calculation models: limit equilibrium, beam on springs, numerical continuum 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) <PER> 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) <PER> 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) <PER> Selected values of $K_{a\gamma}$ and $K_{p\gamma}$ may be determined from Figure D.2 and Figure D.3.

NOTE 390. Values are also given in tabular form by Kérisel and Absi (1990).

(4) <PER> The value of K_{aq} may be determined from Formula (D. 1):

$$K_{\mathrm{aq}} = k_{\mathrm{aq}} \cos\delta \tag{D. 1}$$

where:

k_{aq} is the inclined active earth pressure coefficient;

K_{aq} is the component of k_{aq} normal to the wall face.

(5) <PER> The value of K_{pq} may be determined from Formula (D. 2):

$$K_{\mathrm{pq}} = k_{\mathrm{pq}} \cos\delta \tag{D. 2}$$

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) <PER> The values of k_{aq} and k_{pq} may be determined from Formulae (D.3)-(D.8):

$$k_{aq} = \left(\frac{\cos\delta - \sin\varphi\cos\omega_\delta}{\cos\alpha + \sin\varphi\cos\omega_\alpha}\right) e^{-2\varepsilon_a \tan\phi} \tag{D.3}$$

$$k_{pq} = \left(\frac{\cos\delta + \sin\varphi\cos\omega_\delta}{\cos\alpha - \sin\varphi\cos\omega_\alpha}\right) e^{2\varepsilon_p \tan\phi} \tag{D.4}$$

$$\sin\omega_\delta = \frac{\sin\delta}{\sin\varphi} \tag{D.5}$$

$$\sin\omega_\alpha = \frac{\sin\alpha}{\sin\varphi} \tag{D.6}$$

$$\varepsilon_a = \frac{(\omega_a + a)}{2} + \frac{(\omega_\delta - \delta)}{2} + \beta - \lambda \tag{D.7}$$

$$\varepsilon_p = \frac{(-\omega_a + a)}{2} - \frac{(\omega_\delta + \delta)}{2} + \beta - \lambda \tag{D.8}$$

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 391. Positive orientations of these angles are indicated in Figure D.1.

NOTE 392. 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 393. When $\alpha = \beta = \lambda = 0$, K_{aq} is approximately equal to $K_{a\gamma}$ and K_{pq} to $K_{p\gamma}$.

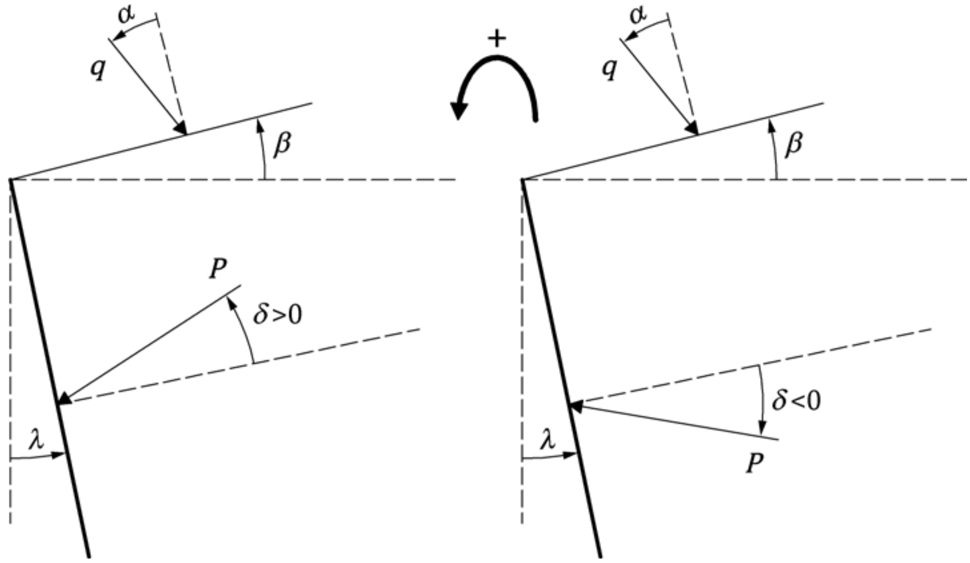


Figure D.1 – Orientation for angles *α*, *β*, *δ*, and *λ* (left: active earth pressure; right: passive)

(7) <PER> When $\varphi > 0$, the values of K_{ac} and K_{pc} may be determined from Formulae (D. 9)-(D. 12):

$$K_{\mathrm{ac}} = \frac{1 - \left(\frac{\cos\delta - \sin\varphi\cos\omega_{\delta}}{1 + \sin\varphi}\right) e^{-2\varepsilon_{\mathrm{a}}\tan\phi}\cos\delta}{\tan\varphi} \tag{D. 9}$$

$$K_{\mathrm{pc}} = \frac{\left(\frac{\cos\delta + \sin\varphi\cos\omega_{\delta}}{1 - \sin\varphi}\right) e^{-2\varepsilon_{\mathrm{p}}\tan\phi}\cos\delta - 1}{\tan\varphi} \tag{D. 10}$$

$$\varepsilon_{\mathrm{a}} = \frac{(\omega_{\delta} - \delta)}{2} + \beta - \lambda \tag{D. 11}$$

$$\varepsilon_{\mathrm{p}} = \frac{(\omega_{\delta} + \delta)}{2} - \beta + \lambda \tag{D. 12}$$

where ω_{δ} and ω_{α} are given in Formulae (D. 5) and (D. 6) and the other symbols are as defined in (6).

NOTE 394. 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) <PER> 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):

$$K_{\mathrm{ac,u}} = K_{\mathrm{pc,u}} = 1 + \sin^{-1}\left(\frac{a}{c}\right) + cos\left(\sin^{-1}\left(\frac{a}{c}\right)\right) \tag{D. 13}$$

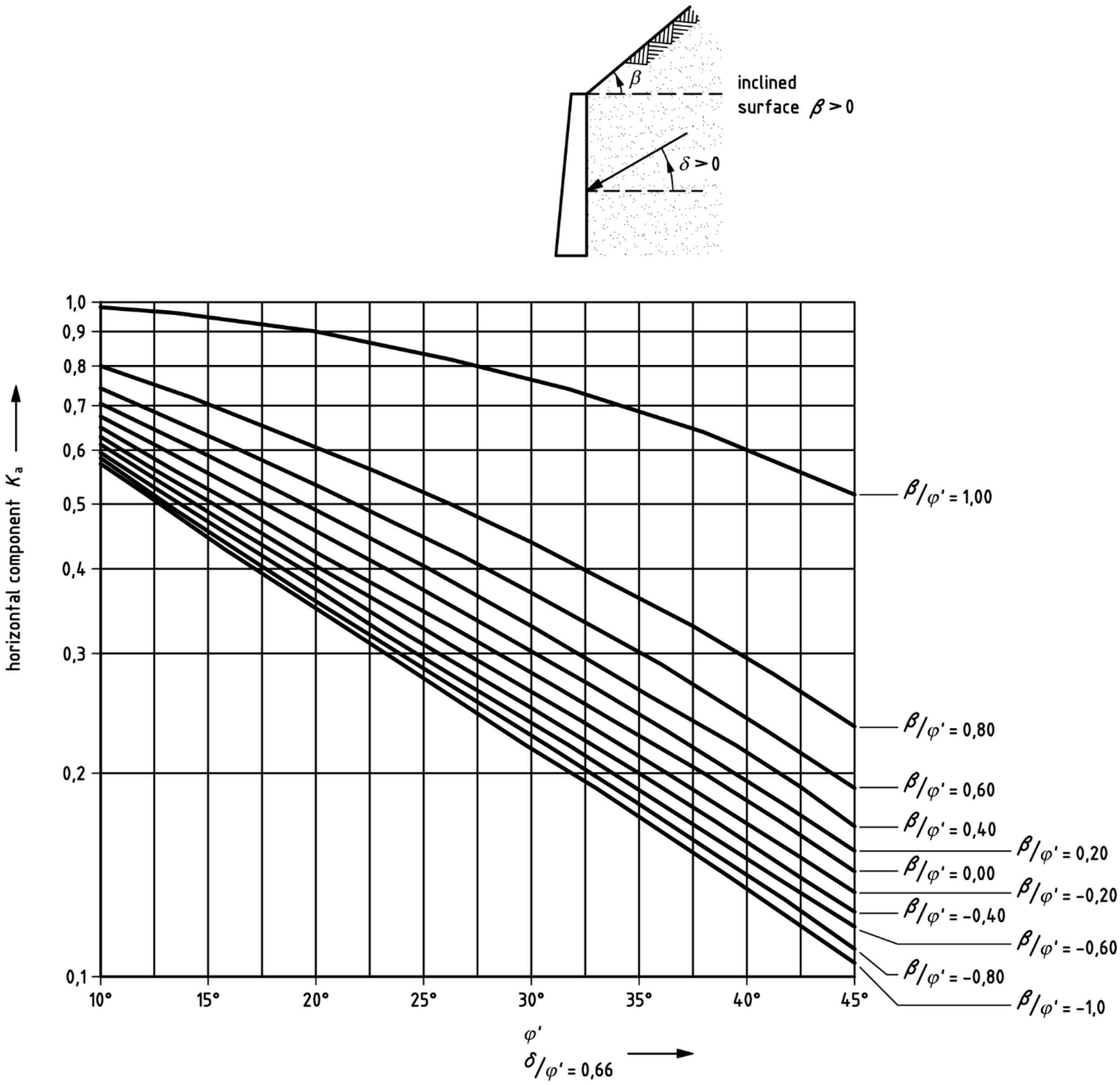


Figure D.2 – Coefficients of effective active earth pressure K_a (horizontal component) with inclined retained surface ($\delta/\varphi' = 0,66$)

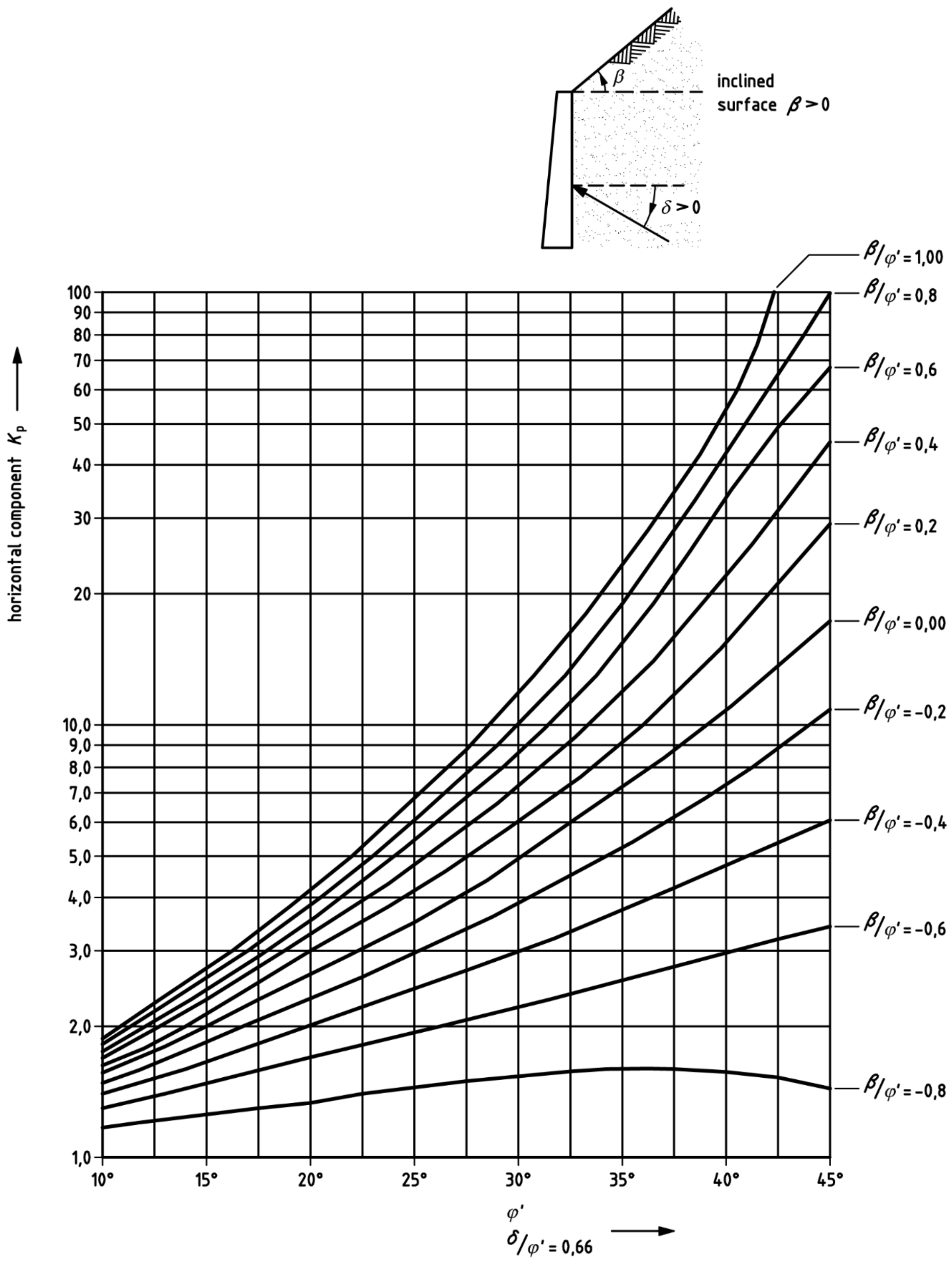


Figure D.3 – Coefficients of effective passive earth pressure K_p (horizontal component) with inclined retained surface (δ/φ' = 0,66)

D.4 Calculation model to determine at-rest values of earth pressure

(1) <PER> 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):

$$K_0 = (1 - \sin\varphi)\sqrt{R_o} \times (1 + \sin\beta) \leq K_{p\gamma} \tag{D. 14}$$

where:

φ is the soil's internal angle of shearing resistance;

R_o is the over-consolidation ratio at depth z_0 (equal to $\sigma'_{v,max}$ / σ'_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) <RCM> Formula (D.14) should not be used for very high values of R_o or in circumstances involving geological reloading.

NOTE 395. Formula (D.14) can lead to unrealistic values of K_0 close to the ground surface, where the vertical stress is low.

(3) <RCM> The direction of the resulting force should be assumed to be parallel to the ground surface.

(4) <PER> 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 396. Assuming linear elastic behaviour and considering reloading stress paths, where υ is Poisson's ratio of the soil:

$$K_d = \upsilon/(1 - \upsilon) \tag{D. 15}$$

NOTE 397. 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 398. 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

NOTE 399. 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) <PER> 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):

$$p'_c = \begin{cases} K_{p\gamma}\overline{\gamma}_c z \text{ for } z \le z_{c,min} \\ p'_{c,max} \text{ for } z_{c,min} \le z \le z_{c,max} \\ K_0\overline{\gamma}_c z \text{ for } z \ge z_{c,max} \end{cases} \quad \text{(D. 16)}$$

$$z_{c,min} = \frac{p'_{c,max}}{\overline{\gamma}_c K_{p\gamma}} \quad \text{(D. 17)}$$

$$z_{c,max} = \frac{p'_{c,max}}{\overline{\gamma}_c K_0} \quad \text{(D. 18)}$$

where:

$p'_{c,max}$ is the maximum horizontal earth pressure due to compaction;

$\overline{\gamma}_c$ is the average weight density of the ground over depth $z_{c,max}$;

$K_{p\gamma,0}$ is the passive earth pressure coefficient (with wall friction equal to zero);

K_0 is the at-rest earth pressure coefficient;

$z_{c,min}$ is the minimum depth at which p'_c applies;

$z_{c,max}$ is the maximum depth at which p'_c applies.

(2) <PER> For non-yielding walls, compaction pressure may be represented by the bi-linear profile shown in Figure D.4(b).

NOTE 400. Compaction pressures from soil placement in layers more realistically produces a distribution similar to that shown in Figure D.4(a).

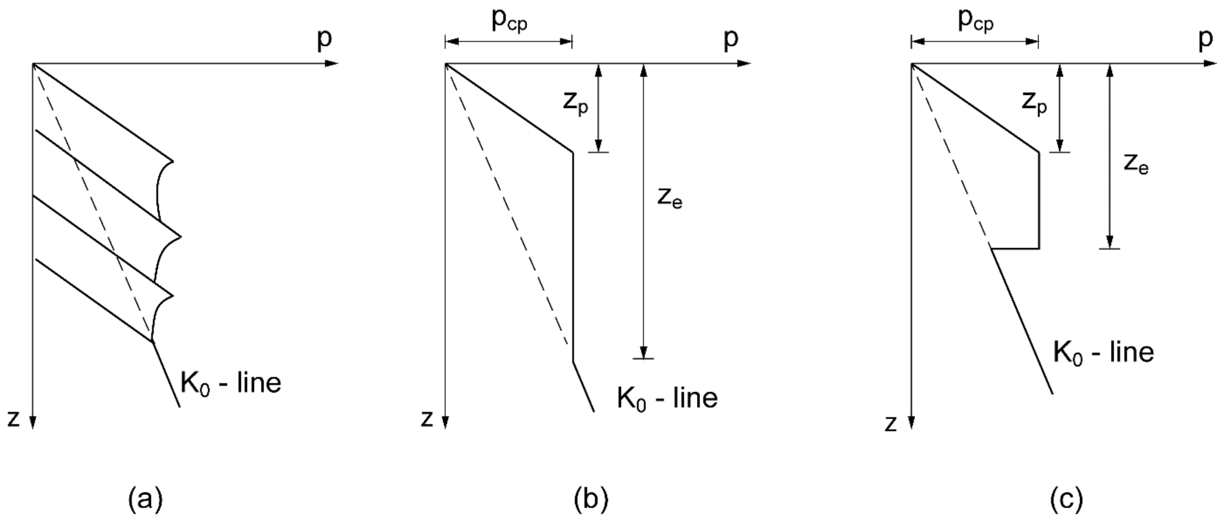


Figure D.4 – Distribution of compaction earth pressure (a); simplified profile for non-yielding wall (b) and yielding wall (c)

(3) <PER> The value of the maximum compaction earth pressure $p'_{c,max}$ may be taken from Table D.1.

(4) <PER> For yielding walls, the simplified depth profile shown in Figure D.4c may be adopted. 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)

Wall	Intensive compaction Width b of backfilled space		Light compaction (vibratory compactor mass ≤ 250 kg)
	b ≤ 1.0 m	b ≥ 2.5 m	
Non-yielding	40	25	15
Yielding	25 (z = 2.0 m)		15 (z = 2.0 m)
Use interpolation for intermediate values of b			

D.6 Basal heave

(1) <RCM> Mechanical heave due to excavation is generally associated with settlements outside and should be considered as part of overall stability mechanisms.

(2) <PER> Specific models may be used to deal with the following situations:

- conventional models for overall stability calculation 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 may locally initiate an instability process for which rigid block mechanisms may not be considered as realistic enough;
- mechanical extrusion of soft clay that occurs simultaneously with excavation at depth cannot be realistically compensated by external shear resistance, as conventional rigid block mechanisms would assume.

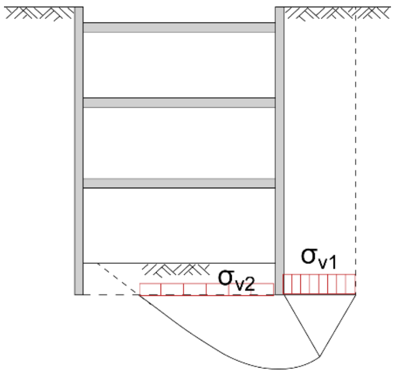


Figure D.5 – Verification against basal heave

(3) <PER> 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.

(4) <PER> In such conditions, the limit value of the vertical stress that can be applied at toe level outside the excavation σ_{v1} may be determined as following:

$$\sigma_{\mathrm{v1}} = \frac{\gamma B}{2} N_{\gamma} + \sigma_{\mathrm{v2}} N_{\mathrm{q}} + c N_{\mathrm{c}} \quad \text{(D.19)}$$

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 vertical stress at toe level inside the excavation.

(5) <PER> Mechanical heave during excavation in fine soils may be analysed assuming undrained conditions and total stress analysis, using $N_{\gamma} = 0$.

(6) <PER> 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.

(7) <PER> Formula (D.19) may in both cases be replaced by the following:

$$\sigma_{\mathrm{v1}} = \sigma_{\mathrm{v2}} N_{\mathrm{q}} + c N_{\mathrm{c}} \quad \text{(D.20)}$$

(8) <RCM> 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) <RCM> The values of σ'_{v1} and σ'_{v2} in Formula (D.8) 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) <REQ> In such situation, these hydraulic gradients and unit weights also need to be evaluated and considered for the calculation of the retaining wall itself.

(11) <RCM> 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:

$$\gamma H_e + q_s \le N_c c_u =$$
$$with\ N_c = 5 * \left(1 + 0.2\frac{H_e}{B}\right) * \left(1 + 0.2\frac{B}{L}\right)\ if\ \frac{H_e}{B} \le 2.5,\ \ N_c = 7.5\left(1 + 0.2 * \frac{B}{L}\right) \quad \text{(D.21)}$$

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 401. For more details, see, Bjerrum and Eide, 1956, Stability of strutted excavations in clay, Géotechnique, Vol. 6, No. 1. pp. 32-47. .

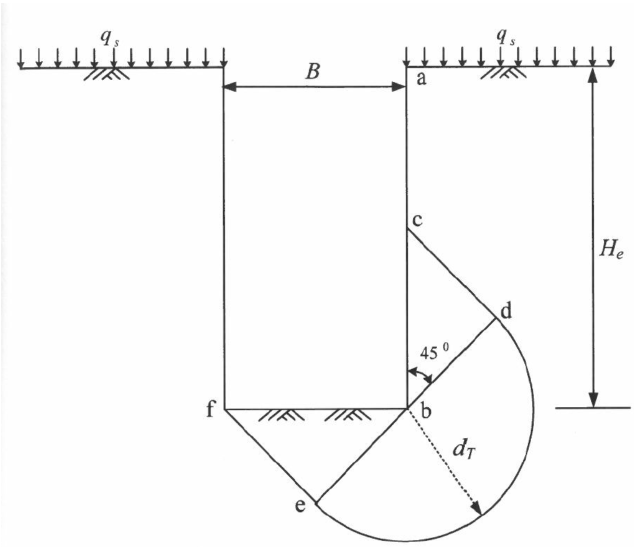


Figure D.6 – Base heave in fine soils (Bjerrum and Eide, 1956)

D.7 Limit equilibrium models

(1) <PER> 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 failure (see §7.6.4.1).

NOTE 402. 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 403. 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 404. 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) <PER> 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 405. 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.8 Beam-on-spring models

(1) <PER> 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 failure (see 7.6.4.1).

(2) <RCM> 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 406. Wall displacements are usually calculated relative to the ground surface, ignoring any displacement of the ground surface.

(3) <PER> Intermediate values of earth 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 407. This is a simplification that assimilates the ground to independent springs.

NOTE 408. 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 409. Spring stiffness values heavily are very software specific.

(4) <RCM> 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 410. Such methods include empirical (see D.6) and continuum numerical models.

NOTE 411. 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 412. Beam-on-springs models are able to take account of increased earth pressures behind rigid supports when they are prestressed.

(5) <PER> Empirical relationships based on past experience may be used to derive soil settlements behind the wall from its horizontal displacement.

NOTE 413. Ratios between maximum vertical and maximum horizontal displacements usually lie between 0.5 and 1.

D.9 Calculation model to determine intermediate values of earth pressure

(111) <PER> The value of the subgrade reaction coefficient k may be estimated from the approximate Formula (D. 22):

$$k = \frac{E_s}{d} \tag{D. 22}$$

where:

E_s is the soil's modulus of elasticity; and

d is the interaction length.

(112) <RCM> 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 may generally be 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 (typically pumping phases without excavation, tidal effects on quay walls, high water head and increased embedded length in order to reach an impervious layer), the interaction length may no longer depend on the bending stiffness, as high differential water pressures affect the total height.

NOTE 414. 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 415. As expressed in A.38(1) NOTE 5, the soil modulus E_s to consider is intermediate between initial loading modulus E_i and an unload-reload modulus E_{ur} (see D.9).

(113) <PER> As an alternative to (1) and (2), other methods may be used for structures that mobilize passive pressure in backfill.

NOTE 416. For example, bridge abutments.

(114) <RCM> Backfill soil reaction forces on bridge abutments should consider the increase in passive earth pressure with wall movement.

(115) <PER> For temperature induced seasonal wall movements, the predominant pattern is a combination of horizontal translation and rotation about the wall base. The horizontal component

of the mobilised passive earth pressure coefficient $K_{ph,mob}$ along the wall height may be determined from Formula (D. 23):

$$K_{ph,mob}(z) = K_0 + \left(K_{ph} - K_0\right)\frac{v(z)/z}{a + v(z)/z} \quad \text{(D. 23)}$$

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, $v(z) = s_h(1 - z/h)$

s_h horizontal displacement at the wall top;

h Is the height of the retaining wall;

a is a backfill-dependent coefficient.

NOTE 417. Formula is given in RE-ING (2016).

(116) <PER> In the absence of detailed specifications, the value a = 0.02 may be used.

D.10 Numerical continuum models

(117) <PER> The most critical geotechnical failure mechanism or combination of failure mechanisms (overall or bottom instability, rotational failure, foundation failure, etc.) may be determined by numerical continuum models using shear strength reduction approach.

(118) <RCM> 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 418. 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 419. 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.

(119) <RCM> In undrained conditions, when calculation are 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.11 **Vertical wall stability**

(1) <RCM> 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) <RCM> 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 420. 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 421. 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 422. Despite using a negative value of the inclination δ to derive earth pressure on the retained side, the vertical component may 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.

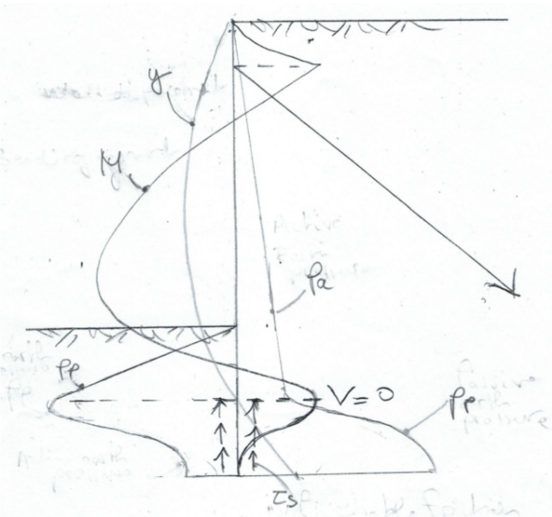


Figure D.7 – Depth at which shear force applied to embedded wall is zero

Key: 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

D.12 Determination of the anchor length to prevent interaction between anchors and retaining structures

(1) <PER> Potential interaction between a retaining structure and any deadman anchors used to stablize it may be ignored when the passive wedge mobilized by the anchor does not intersect with the active wedge acting on the structure.

(2) <PER> 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 may be 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 may be 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 423. 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 may be justified that interaction may be neglected.

NOTE 424. 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 425. 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.

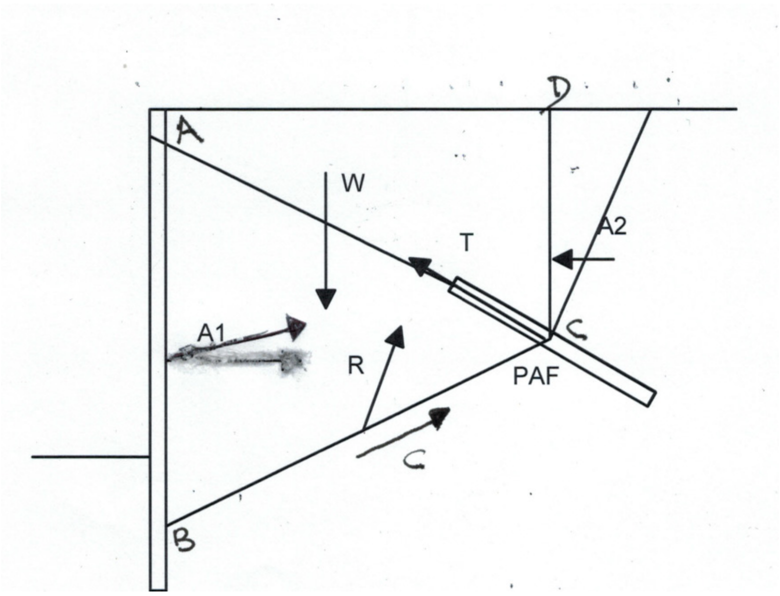


Figure D.8 – Determination of anchor length to prevent interaction with retaining structure

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

(3) <PER> For grouted anchors, the resulting force exerted in the ground may be assumed to act in the middle of the fixed anchor length.

NOTE 426. This assumption is relevant in standard ground conditions for which friction may be considered as uniformly distributed along the anchored length.

(4) <REQ> 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) <REQ> The equivalent free length shall be consistent with the fixed anchor length along which friction is considered when verifying the bearing capacity of the micropile according to Clause 6

Annex E

(informative)

Anchors

E.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 8 regarding anchors.

NOTE 427. 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 Annex covers:

– layout of anchors

E.3 Example for anchor design models

(1) <RCM> The free anchor length should be determined during the design of the anchored structure.

NOTE 428. Examples of design models for anchored structures are given in Annexes A and D.

E.4 Layout of anchors

(1) <RCM> The layout of anchors should take into account the proximity of the load-bearing stratum and the execution.

NOTE 429. Examples of the configuration of anchors are given in **Figure E.1**, **Figure E.2**, and Figure E.3.

NOTE 430. 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.

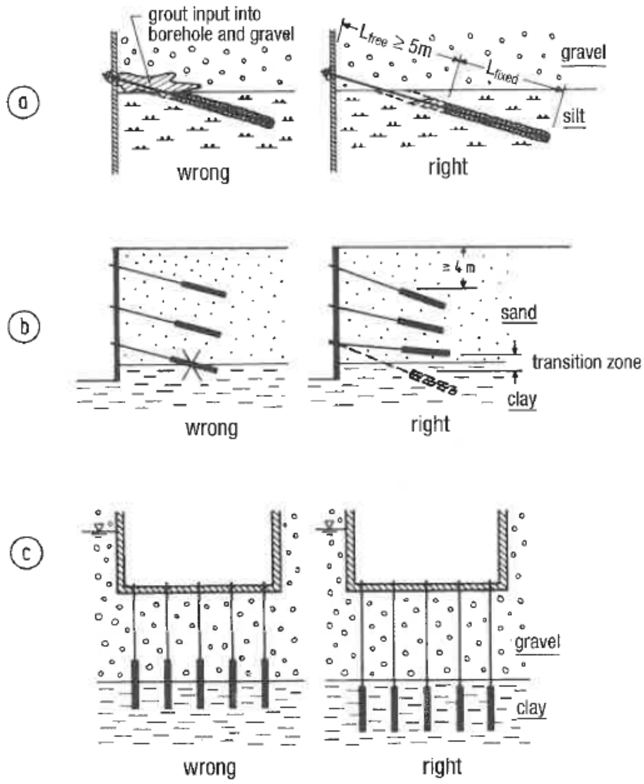


Fig. 23. Lay-out of grout bodies in stratified ground

Figure E.1 – Examples of good and bad anchor configurations in stratified ground

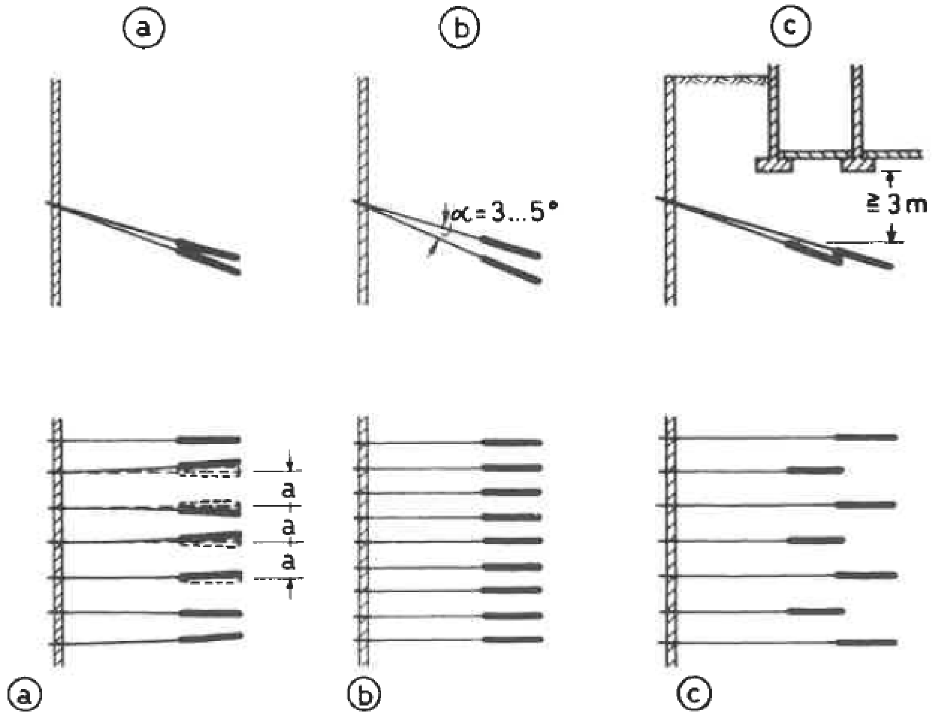


Figure E.2 – Examples of good and bad spreading and staggering of anchors: (upper) section, (lower) plan; (a) wrong; (b)-(c) right

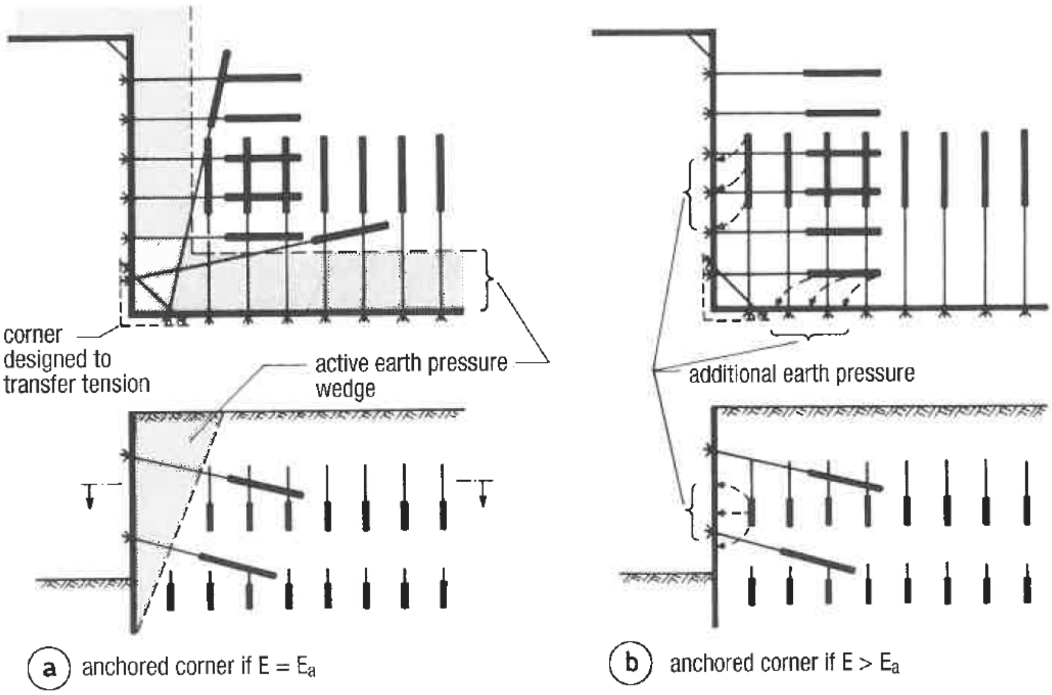


Figure E.3 – Examples of anchoring a protruding wall corner

<Drafting NOTE: *E* = earth pressure; E_a = active earth pressure>

Annex F

(informative)

Reinforced fill structures

<Drafting note: PT6 have had discussions on this Annex, and to what extent it is useful in its current format for EN 1997. PT6 therefore would like to have opinions on this annex from the NSBs>

F.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 9 for reinforced fill structures.

NOTE 431. 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) <PER> Slip surface analysis using the method of slices may be used for verifying internal and compound stability.

(2) <PER> In the case of reinforced slopes, the horizontal interslice forces may be ignored only if (3) is applied as well.

(3) <PER> 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) <RCM> 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) <RCM>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)).

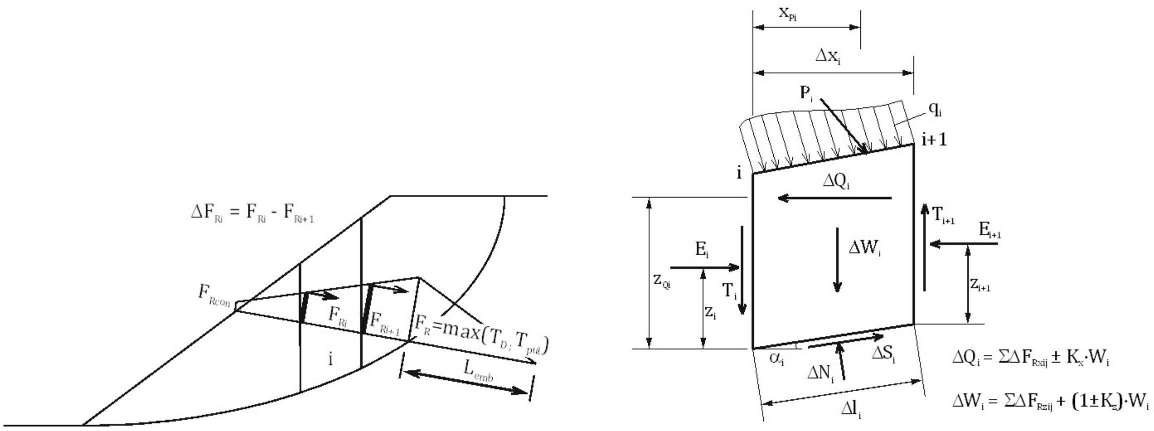


Figure F.1 – Forces from reinforcing element – implementation into method of slices

F.3.2 Coherent gravity method

(1) <PER> The coherent gravity method may be used for direct calculation of the load in each layer of soil reinforcements for internal stability check.

(2) <PER> The coherent gravity method may be used for non-extensible reinforcement that develops its tensile design strength at a strain < 1%.

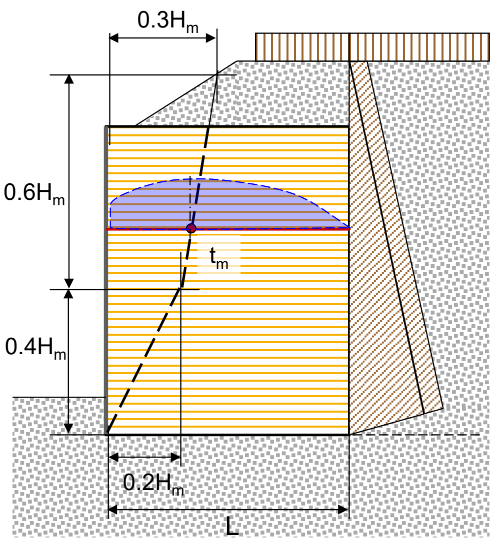


Figure F.2 – Coherent gravity method

(3) <PER> The coherent gravity method may be use

(4) <RCM> The stress state within the reinforced soil block should be taken to be K_0 at the effective ground surface reducing to K_a at a depth of 6 m.

(5) <RCM> 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. 1):

$$T_{\rm j} = T_{\rm p,j} + T_{\rm s,j} + T_{\rm f,j} = K\sigma_{\rm v,j}S_{\rm v,j} + T_{\rm s,j} + T_{\rm f,j} = K\left(\frac{R_{\rm v,j}}{L_{\rm j} - 2e_{\rm j}}\right)S_{\rm v,j} + T_{\rm s,j} + T_{\rm f,j} \qquad \text{(F.1)}$$

where:

$T_{p,j}$ 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 *j*th layer of reinforcement;

$\sigma_{v,j}$ is the vertical stress on the *j*th layer of reinforcements;

$S_{v,j}$ is the vertical spacing of the reinforcements at the *j*th 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 *j*th 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 *j*th layer of reinforcement

(6) <RCM> The line of maximum tension in the reinforcement should be assumed as indicated on Figure F.2.

(7) <REQ> 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 432. Detailed calculation procedure of coherent gravity method can be found in NF P 94 270.

F.3.3 Tie-back wedge method

(1) <PER> 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) <PER> The tie-back wedge method may be used for extensible reinforcement that develops its tensile design strength at a strain > 1%.

(3) <RCM> The stress state within the reinforced soil block should be taken to be K_a.

(4) <RCM> The verification of tensile resistance of a reinforcing element should comply with F.3b and Formula (F.1) with K equal to K_a.

NOTE 433. The Tie-back method is based on a rectangular shaped reinforced soil block and earth pressure coefficients with limited wall slopes (< 20° from vertical).

(5) <RCM> The stability of a series of potential straight line failure planes forming wedges through the reinforced soil block should also be checked accounting for beneficial effect from the tensile resistance within each reinforcement layer that crosses the failure plane (see Figure F.3).

(6) <REQ> The tensile resistance of each reinforcing element shall comply with 9.6.2.

NOTE 434. Detailed calculation procedure can be found in BS 8006-1.

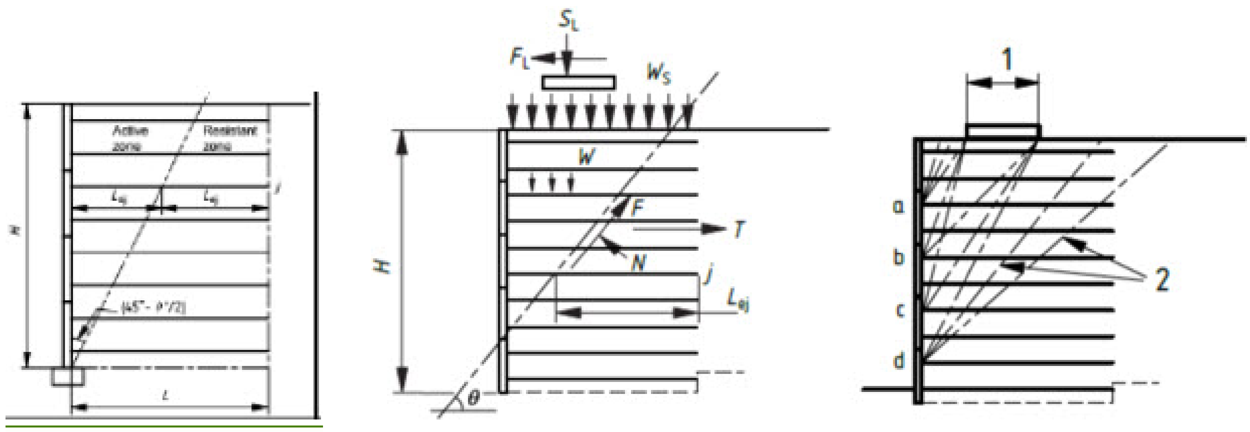


Figure F.3 – Tie-back wedge method

F.3.4 Multi-part wedge method

(1) <PER> The multi-part wedge method may be used for internal and compound stability check.

(2) <RCM> 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) <RCM> 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.

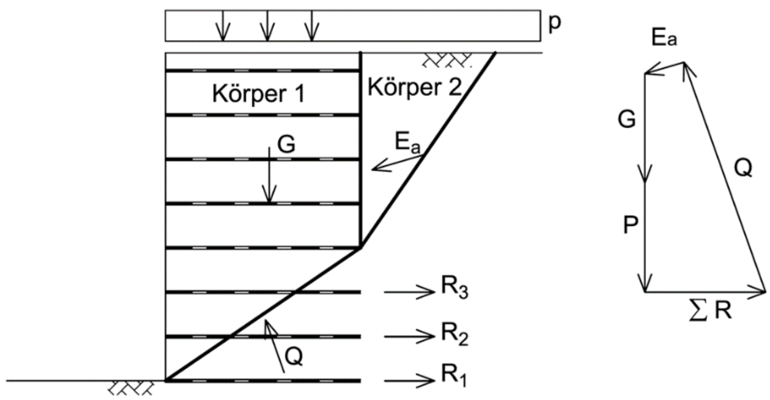


Figure F.4 – Two-part wedge method

F.4 Calculation models for reinforced embankment bases

F.4.1 Resistance to transverse sliding

(1) <RCM> The lateral sliding stability of the embankment should be determined by examining any preferential slip surfaces that pass above the basal reinforcement layers

(2) <RCM> 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;

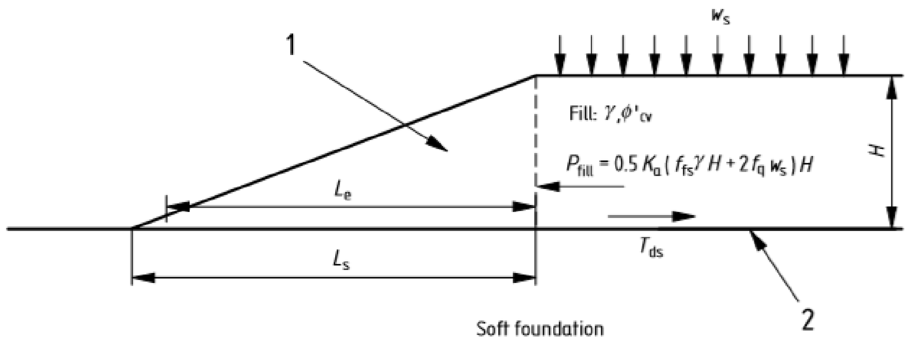
H is the height of the embankment; and

W_{s} is the surcharge load.

(3) <REQ> The tensile resistance of the reinforcing elements shall be greater than the lateral thrust.

NOTE 435. The lateral trust may be reduced by sliding resistance along the bottom of the reinforcement layers beneath the embankment side slope.

(4) <REQ> 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).



Key

1 Embankment

2 Reinforcement

Figure F.5 – Calculation model to determine resistance to sliding

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F.4.2 Resistance to foundation extrusion

(1) <RCM> 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) <RCM> The side slope of the embankment should be long enough to develop resistance to prevent the mobilisation of the outward shear stresses in the foundation soils (see Figure F.6).

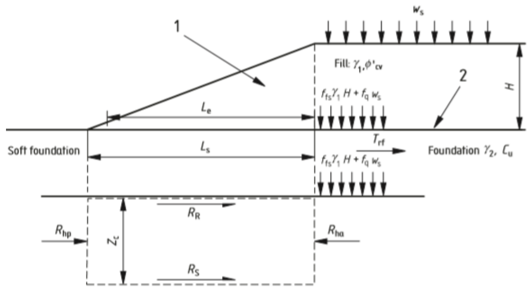

Figure F.6 – Calculation model to determine resistance to extrusion

(3) <RCM> The minimum side slope length required should be determined using Formula (F. 3):

$$L_{\mathrm{e}}=\frac{(\gamma H+W_{\mathrm{s}}-4c_{\mathrm{u}})z_{\mathrm{c}}}{(1+\alpha'_{\mathrm{ds}})c_{\mathrm{u}}} \tag{F.3}$$

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) <RCM> Basal reinforcement should be designed to transfer the load from the embankment onto the discrete inclusions (see Figure F.7).

(2) <RCM> 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) <REQ> The tensile force F_{LTP} shall be smaller than tensile resistance in the reinforcement determined from isochronous creep curves for specified limiting strain for an analysed limit state.

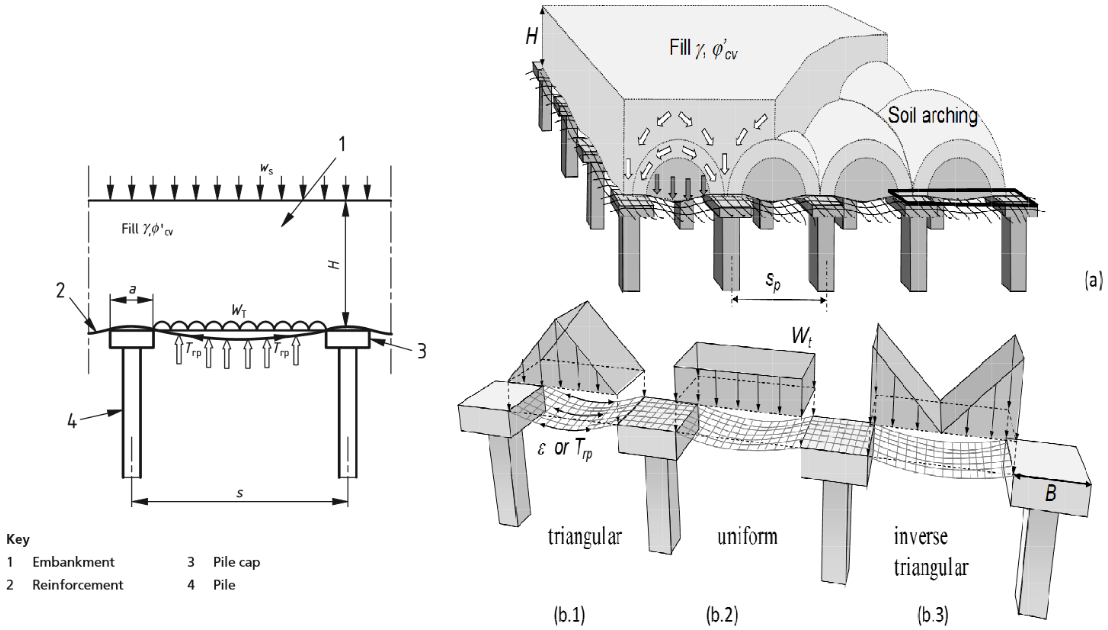

Figure F.7 – Schematic concept of a load transfer platform over discrete inclusions

F.5.2 Hewlett and Randolph method

(4) <REQ> In the Hewlett and Randolph method, the surcharge on the load transfer platform (W_{T} in Figure F.7) shall be assumed to be constant.

(5) <RCM> For geosynthetic reinforcement that allows some deformation, the tensile force F_{LTP} in a reinforcing element should be determined from Formula (F. 4F. 4F. 4F. 4):

$$F_{\mathrm{LTP}} = \frac{W_T\left(s_{\mathrm{p}} - B\right)}{2B}\sqrt{1 + \frac{1}{6\varepsilon}} \qquad (\mathrm{F}.4)$$

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 436. This formula assumes that there is no support from underlying low bearing strata.

NOTE 437. Detailed information about the Hewlett and Randolph method can be found in BS 8006-1.

F.5.3 EBGEO method

(1) <REQ> In the EBGEO method, the surcharge on the load transfer platform (W_{T} in Figure F.7) shall be assumed to be triangular.

(2) <RCM> The determination of surcharges and resistances of individual system elements should be determined by iterative calculation procedure.

NOTE 438. Details of the calculation procedure can be found in EBGEO.

F.5.4 Concentric Arches method

(1) <REQ> In the Concentric Arches method, the surcharge on the load transfer platform (W_T in Figure F.7) shall be assumed to have a shape of inverse triangle or uniform load with respect to embankment height and subsoil resistance support.

(2) <RCM> Surcharges and resistances of individual system elements should be determined by an iterative calculation procedure.

NOTE 439. Details of the calculation procedure can be found in CUR 226.

F.6 Calculation models for embankments over voids

(1) <PER> 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) <RCM> The design void diameter should be assumed based on previous experience of the site conditions.

(3) <RCM> 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 440. 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.

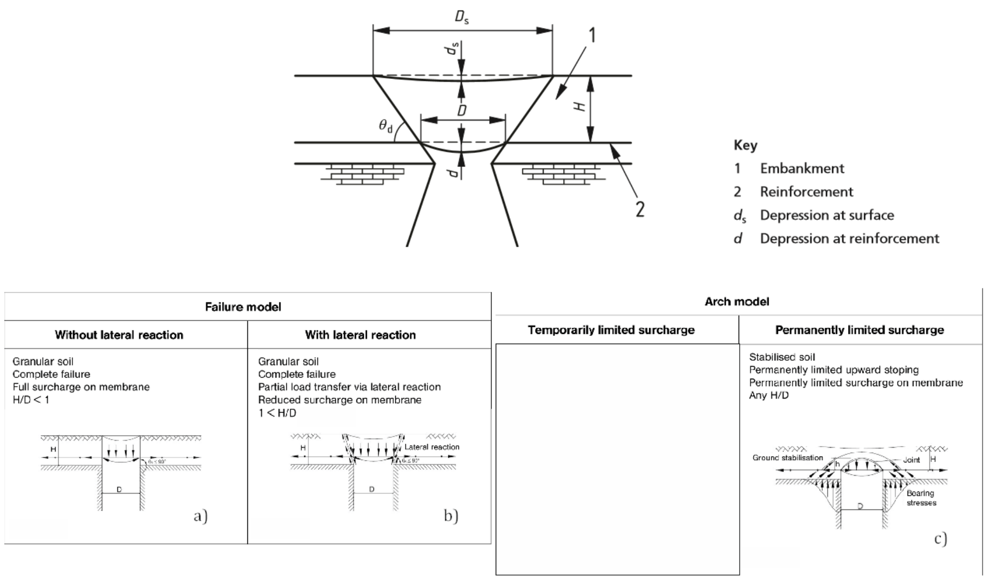

Figure F.8 – Parameters required for Formula (F.6)

(4) <REQ> Provided the deformed shape of the geosynthetic reinforcement is parabolic, the strain in the reinforcement layer ε shall be determined from Formula (F. 5):

$$\varepsilon = \frac{8}{3}\left(\frac{d_s}{D_s}\right) \tag{F.5}$$

where:

d_s is the deformation at the surface; and

D_s is the diameter of the depression at the surface,

(5) <PER> 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) <RCM> For cases b and c shown in Figure F.8, more complex calculation procedures should be followed to determine the force F_{vo}.

NOTE 441. For further details, see EBGEO.

(7) <PER> 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) <REQ> 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) <RCM> 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 (see Figure F.9).

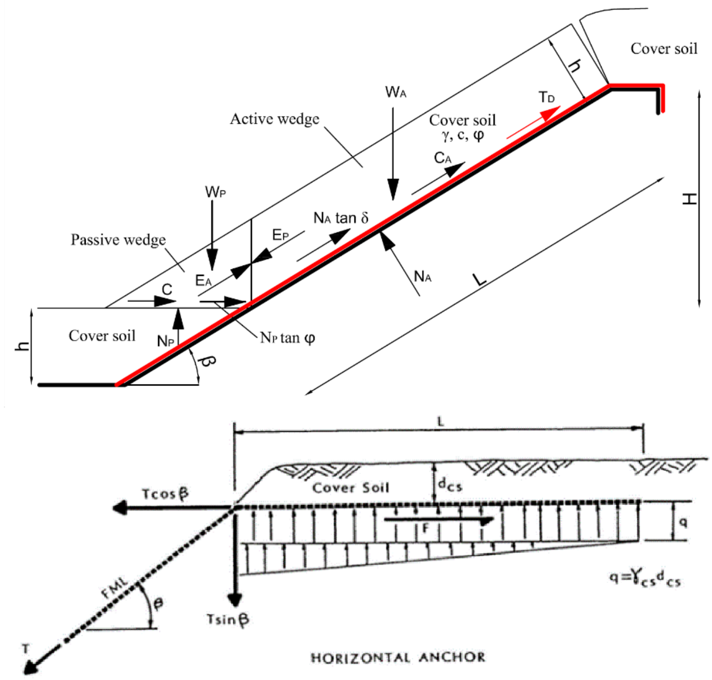

Figure F.9 – Forces acting on a veneer system

(2) <RCM>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 442. Veneer systems can be made up of multiple synthetic and mineral layers with different frictional characteristics.

(3) <RCM> The tensile force T_{ven} required to hold the veneer system on the slope without water should be determined from Formula (F. 6):

$$T_{ven} = W_A \sin\beta - W_A \cos\beta \tan\delta - C_A - \frac{C_P + W_P \tan\varphi}{\cos\beta - \sin\beta \tan\varphi} \qquad \text{(F. 7)}$$

where the symbols are defined in Figure F.9a.

(4) <RCM> The tensile force F_{ven} shall be smaller than tensile resistance in reinforcing element for an analysed limit state.

(5) <RCM> The stability of the horizontal anchorage at the top of the veneer (see Figure F.9b) without water should be verified using Formula (F. 8):

$$T_{\text{ven}} \leq \left(\frac{T_{\text{ven}} \sin\beta}{L_{\text{ds}}} + \gamma_{\text{cs}} d_{\text{cs}}\right) f_{\text{ds}} L_{\text{ds}} \qquad \text{(F. 8)}$$

where the symbols are defined in Figure F.9b.

(6) <RCM> When water is present or a different shape of anchorage is used, Formulae (F.8) and (F.9) should be amended accordingly.

Additional details on calculation procedure are given by Rimoldi (2018).

Annex G

(informative)

Ground improvement

G.1 Use of this Informative Annex

(1) This Informative Annex provides additional guidance to that given in Clause 11 for ground improvement.

NOTE 443. 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 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

Table G.1 – Examples of diffused ground improvement techniques

Method	Technique	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 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
	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 147131
	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

Method	Technique	Class	Description	Execution Standard
	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
	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
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 AII	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

Table G.2 – Examples of discrete ground improvement techniques

Method	Technique	Class	Description	Execution Standard
	Dry methods	BII	Mechanical disaggregation of soils while introducing a dry binder pneumatically and commonly cement. Most usually executed in	EN 14679

Method	Technique	Class	Description	Execution Standard
Mixing Methods			soft to very soft clays and silts. Land and marine based rigs available to considerable depths.	
	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. For cohesive soil, this increases the shear strength, and at the same time, it stabilises early settlement to reduce the consolidation settlement rate. For granular soils, it increases the relative density, thus enhances the shear strength. 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. For cohesive soil, this increases the shear strength, and at the same time, it stabilises early settlement to reduce the consolidation settlement rate. For granular soils, it increases the relative density, thus enhances the shear strength. Land and marine based rigs available to considerable depths.	EN 14713
	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 improve strength and accelerate drainage. Land and marine based rigs available.	None
	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, 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
	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

Method	Technique	Class	Description	Execution Standard
	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 and densification effort to the existing ground.	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 densification effort to the existing ground.	None
	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. For cohesive soil, this increases the shear strength, and at the same time, it stabilises early settlement to reduce the consolidation settlement rate. For granular soils, it increases the relative density, thus enhances the shear strength. Land and marine based rigs available to considerable depths.	None

G.4 Use of stress envelope to determine acceptable limit states

(1) <REQ> In case of using a calculation method based on deriving the principal stresses it shall be verified that the design values of effects of actions do not exceed the states of stress defined in Figure G.1.

(2) <RCM> It should be assumed that the body can crack if the principal tensile stress exceeds 10% of $f_{m,d}$.

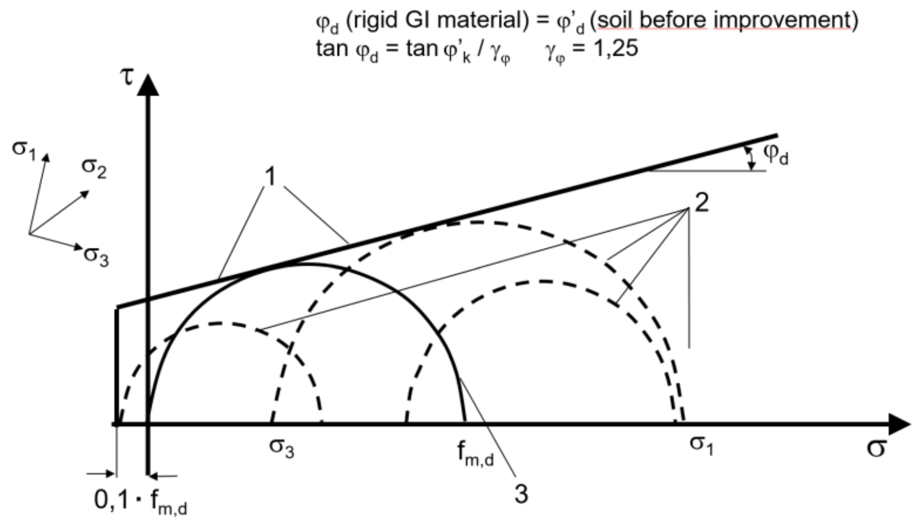

Figure G.1 – Allowable stresses in rigid ground improvement material

φ_d (strengthened soil) = φ'_d (unimproved soil)

tan φ_d = tan φ_k / γ_φ

Key: 1 – Envelope for allowed states of stress

2 - Examples for states of stress σ_1, σ_3, allowed as the design values of effects of actions

3 - State of stress in a uniaxial compression test: $\sigma_3 = 0$, $\sigma_1 = f_{m,d}$

(3) <REQ> For Class BII rigid inclusions subjected to eccentricities, eccentricities shall be considered in the verification of the internal bearing capacity according to 11.5.3.2(2) and resulting stresses within the cross section shall be verified to be within the stress envelope gin in Figure G.3

NOTE 444. It is assumed that the element can crack if the principal tensile stress exceeds 10% of f_{md}.

(4) <PER> As a simplification, compressive and shear stresses may be analysed separately during design of the strengthened soil.

<REQ> It shall be verified that design values of normal stress do not exceed 0.7 $f_{m,d}$ and design values of shear stress do not exceed 0.2 $f_{m,d}$ (see Figure G.1).

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