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DS/EN 1997-3:2025 DK NA:202X

**National Annex to: Eurocode 7 – Geotechnical design-
Part 3: Geotechnical structures**

**Nationalt Anneks til: Eurocode 7 – Geoteknik -
Del 3: Geotekniske konstruktioner**

Foreword

Published [måned og år].

This National Annex (NA) to DS/EN 1997-3:2025 forms part of the Danish national annexes to the second generation of Eurocodes.

This NA applies when it is implemented in the Building Regulations (Bygningsreglementet 2018 – BR18) as an appendix.

During the transition period stated in the Building Regulations, both the first generation and the second generation of the Eurocodes and NAs may be applied.

This NA cannot be used in conjunction with the first generation of Eurocodes.

This NA lays down the conditions for the Danish implementation of EN 1997-3:2025 for construction works in conformity with the Danish Building Regulations; Bygningsreglementet 2018 (BR18). Other parties may bring this NA into force by reference.

The Eurocodes establish the principle that certain parameters are left open for national determination. These parameters are referred to as *Nationally Determined Parameters* (NDP) and specifies the nationally applicable values, classes or methods. Furthermore, the NA may provide *Non-Contradictory, Complementary Information* (NCCI), i.e. additional guidance and alternative methods or values that support the application of the Eurocode without altering or contradicting its provisions.

Further, this National Annex includes NOTES. Notes have the status of guidance and provide additional information intended to assist the understanding or use of text of the national annex.

In conformity with the Danish Building regulations, the verbal forms used in this Eurocode and NA are to be understood as follows:

- “shall” expresses a requirement to be followed
- “should” expresses a recommendation. Alternative approaches could be used if technically justified
- “may” expresses permission within the limits of the Danish Building Regulations
- “can” expresses a possibility or capability
- “is/are” expresses a statement of certainty or a fact

This NA includes:

- Foreword;
- An overview of clauses with possible Nationally Determined Parameters (NDPs) and of Non-Contradictory Complementary Information (NCCI);
- National choice to national Determined Parameters (NDP), and Non-Contradictory Complementary Information (NCCI)

Valid versions of the NAs as well as previous versions, addenda can be found at www.bygningsreglementet.dk.

Overview of possible Nationally Determined Parameters (NDPs) and Non-Contradictory Complementary Information (NCCI)

The list below identifies the clauses where Nationally Determined Parameters (NDPs) are possible, and whether a national choice has been made, and if an informative annex is applicable or not applicable. Furthermore, clauses providing complementary information (NCCI) are identified in the list below.

Clause	Subject	Nationally Determined Parameter	Non-Contradictory Complementary Information
4.2.7.2(1)	Minimum extent of field investigation	National choice	
4.3.4(1)	Other materials	Unchanged	
4.6.3(1)	Partial factors	National choice	
5.2.4.2(4)-(6)	Permanent and variable actions		Complementary information
5.2.4.4(4)	Environmental influences		Complementary information
5.2.7.2(1)	Minimum extent of field investigation	National choice	
5.3.3(1)	Other materials	Unchanged	
5.5.4(4)	Settlement		Complementary information
5.6.1(3)	General		Complementary information
5.6.1(5)	General	National choice	
5.6.3(5)	Bearing failure	National choice	
5.6.3(6)	Bearing failure		Complementary information
5.6.3(7)	Bearing failure	National choice	
5.6.6(1) – 3 choices	Partial factors	National choice	
5.6.6(3)	Partial factors	National choice	
5.7.5(1)	Physical gap beneath the foundation	National choice	
6.1(2)	Scope and field of application	Unchanged	
6.2.7.2(1)	Minimum extent of field investigation	National choice	
6.3.7(1)	Other materials	Unchanged	

6.5.3.1(2)	Calculation	National choice	
6.5.3.3(5)	Testing	Unchanged	
6.5.3.3(8)	Testing	National choice	
6.5.3.3(10)	Testing	Unchanged	
6.5.3.3(11)	Testing	Unchanged	
6.5.6(3)	Resistance of piled rafts	Unchanged	
6.5.6(4)	Resistance of piled rafts	Unchanged	
6.5.7.2(1)	Displacement of piled foundations, Displacement of single piles		Complementary information
6.5.8(4)	Confirmation of pile design by testing or comparable experience	National choice	
6.6.2.1(2) – 2 choices	Axial compressive resistance	National choice	
6.6.2.4.2(1)	Model Pile Method	National choice	
6.6.2.4.3(1)	Testing	National choice	
6.6.2.5(6)	Structural resistance		Complementary information
6.6.3(4)	Ultimate limit state for pile groups and piled rafts	National choice	
6.6.3(5)	Ultimate limit state for pile groups and piled rafts	National choice	
6.6.4.1(1) – NOTE 1	Single piles	National choice	
6.6.4.1(1) – NOTE 2	Single piles	Unchanged	
6.6.4.2(1) – 2 choices	Pile groups and piled rafts	National choice	
6.7(4)	Serviceability limit states	National choice	
6.9.3(6)	Test proof load	National choice	
6.9.4(1)	Static load test		Complementary information
7.2.7.2(1)	Minimum extent of field investigation	Unchanged	
7.3.7(1)	Other materials	Unchanged	
7.4.1(1)	General	National choice	

7.5.3.2(2)	Active earth pressure	National choice	
7.5.3.2(5)	Active earth pressure	National choice	
7.6.2(6)	Overall stability	National choice	
7.6.7(2)	Supporting elements	Unchanged	Complementary information
7.6.9(1) – 2 choices	Partial factors	National choice	
8.2.7.2(1)	Minimum extent of field investigation	Unchanged	
8.3.4(1)	Other materials	Unchanged	
8.6.2(1)	Geotechnical resistance	National choice	
8.6.2(4)	Geotechnical resistance		Complementary information
8.6.2(6)	Geotechnical resistance		Complementary information
8.6.2(7)	Geotechnical resistance	National choice	
8.6.2(8)	Geotechnical resistance	Unchanged	
8.6.2(9)	Geotechnical resistance	National choice	
8.6.4(1) – 2 choices	Partial factors	National choice	
8.7.2(4)	Geotechnical resistance	Unchanged	
8.7.3(1)	Partial factors	Unchanged	
8.9.1(2) – 2 choices	General	National choice	Complementary information
8.9.3(1)	Sustainability tests	National choice	
9.2.7.2(1)	Minimum extent of field investigation	National choice	
9.3.3(6)	Geosynthetics	National choice	
9.3.4(16) – 2 choices	Steel	Unchanged	
9.3.7(1)	Other materials	Unchanged	
9.6.2.1(1)	Geosynthetics	National choice	
9.6.2.3(1)	Polymeric coated steel woven wire mesh	Unchanged	
9.6.7(1) – 2 choices	Partial factors	National choice	

10	Soil nailed structures	Not applicable	
10.3.2.2(2)	Durability	Awaiting	
10.3.7.1(11) – 2 choices	Steel wire mesh facings	Awaiting	
10.3.9(1)	Other materials	Awaiting	
10.6.2(2)	Pull-out resistance of soil nails	Awaiting	
10.6.2(3)	Pull-out resistance of soil nails	Awaiting	
10.6.2(6)	Pull-out resistance of soil nails	Awaiting	
10.6.2(7)	Pull-out resistance of soil nails	Awaiting	
10.6.5(3)	Structural resistance of steel wire mesh facings	Awaiting	
10.6.7(1) – 2 choices	Partial factors	Awaiting	
10.9.1(4)	Testing, General	Awaiting	
10.9.2(1) – 3 choices	Investigation, suitability and acceptance test for soil nails	Awaiting	
10.9.2(8) – 2 choices	Investigation, suitability and acceptance test for soil nails	Awaiting	
10.9.2(10) – 2 choices	Investigation, suitability and acceptance tests for soil nails	Awaiting	
11	Rock bolts and rock surface support	Not applicable	
11.3.3.2(3)	Durability	Awaiting	
11.3.3.2(8)	Durability	Awaiting	
11.3.5(1)	Other materials	Awaiting	
11.6.2(1)	Interface resistance of rock bolts (by calculation)	Awaiting	
11.6.3(1)	Interface resistance of rock bolts (by testing)	Awaiting	
11.6.5(1)	Partial factors	Awaiting	
11.9.1(4)	Testing, General	Awaiting	
11.9.2(1) – 3 choices	Investigation, suitability and acceptance tests for rock bolts	Awaiting	
11.9.2(4) – 2 choices	Investigation, suitability and acceptance tests for rock bolts	Awaiting	

11.9.2(5) – 2 choices	Investigation, suitability and acceptance tests for rock bolts	Awaiting	
11.9.3(1)	Control test	Awaiting	
12	Ground improvement	Not applicable	
12.2.2(2)	Geometrical properties	Awaiting	
12.2.7.2(1)	Minimum extent of field testing	Awaiting	
12.5.4(2) – 2 choices	Unconfined compressive strength of Class II material	Awaiting	
12.5.4(5)	Unconfined compressive strength of Class II material	Awaiting	
12.5.4(6) – 2 choices	Unconfined compressive strength of Class II material	Awaiting	
12.6.1(2)	Ultimate limits states, General	Awaiting	
12.6.3(1)	Resistance of discrete (Class B) ground improvement	Awaiting	
12.6.6(1) – 2 choices	Partial factors	Awaiting	
12.9.2.1(1)	Material control tests, General	Awaiting	
12.9.2.2(2)	Unconfined compressive strength	Awaiting	
13.2.7.2(1)	Minimum extent of field investigation	Unchanged	
13.3(4)	Materials	Unchanged	Complementary information
13.8.3(1)	Monitoring	Unchanged	
Annex A	Slopes, cuttings, and embankments	Informative with changes in annex A NA	
Annex A NA	Slopes, cuttings, and embankments	Informative	
Annex B	Spread foundations	Not applicable except annex B.2, B.3 and B.6-B16	
Annex B.2	Scope and field of application	Informative	
Annex B.3	Checklists	Informative	
B.5(3)	Calculation model for bearing resistance on ground underlain by a weaker layer	National choice, see annex B.5 NA	
Annex B.6-B.16		Informative, see annex B.1 NA (2)	

Annex B NA	Spread foundations	Normative	
<u>Annex B.1 NA</u>	<u>Use of this annex</u>	<u>Normative</u>	
<u>Annex B.4 NA</u>	<u>Calculation model for bearing resistance using soil parameters</u>	<u>Normative</u>	
<u>Annex B.5 NA</u>	<u>Calculation model for bearing resistance on ground underlain by a weaker layer</u>	<u>Normative</u>	
Annex C	Piled foundations	Not applicable except C.6, C.7, C.8, C.10, C.11, C.12, C.13 and C.14.	
Annex C.6	Axial pile resistance based on CPT profiles	Informative, see annex C.1 NA (2)	
Annex C.7	Axial pile resistance from Ménard pressuremeter profiles	Informative, see annex C.1 NA (2)	
Annex C.8	Axial pile resistance based on empirical tables	Informative, see annex C.1 NA (2)	
Annex C.10	Pile groups subject to axial tension functions	Informative, see annex C.1 NA (2)	
Annex C.11	Pile groups subject to axial tension functions	Informative, see annex C.1 NA (2)	
Annex C.12	Calculation model for single pile lateral displacement using load transfer functions	Informative, see annex C.1 NA (2)	
Annex C.13	Buckling and second order effects	Informative	
C.13.3.3(2)	Initial transverse deflection	Unchanged	
C.13.3.3(4)	Initial transverse deflection	Unchanged	
C.13.5.3(3) – 3 choices	Steel piles	Unchanged	
C.13.5.3(4)	Steel piles	Unchanged	
Annex C.14	Determination of axial pile resistance under cyclic loading	Informative, see annex C.1 NA (2)	
Annex C NA	Piled foundations		
Annex C.1 NA	Use of this annex	Normative	
Annex C.2 NA	Scope and field of application	Informative	
Annex C.3 NA	Pile classification	Informative	

Annex C.4 NA	Pile resistance based on ground parameters	Normative	
Annex C.5 NA	Pile resistance based on pile driving formulae	Normative	
Annex C.6 NA	Axial pile resistance based on CPT profiles	Informative	
Annex C.9 NA	Downdrag due to vertical ground movements	Normative	
Annex C.15 NA	Static load tests	Informative	
Annex D	Retaining structures	Not applicable except D.3, D.6, D.7, D.8, D.9, D.10, D.11, D.12 and D.13	
Annex D.3, D.6, D.7, D.9, D.10.1, D.11, D.12 and D.13		Informative, See Annex D.1 NA (2)	
D.8(2)	Limit equilibrium models	National choice	
Annex D NA	Retaining structures		
Annex D.1 NA	Use of this annex	Normative	
Annex D.2 NA	Scope and field of application	Informative	
Annex D.2.A NA	General	Normative	
Annex D.4 NA	Calculation model to determine at-rest values of earth pressure	Informative	
Annex D.5 NA	Earth pressures due to compaction	Normative	
Annex D.10 NA	Calculation model to determine intermediate values of earth pressure	Informative	
Annex E	Anchors	Informative with changes in annex E.5 NA, figure E.3 NA	
Annex E.5 NA	Anchors	Informative	
Annex F	Reinforced fill structures	Informative with changes in annex F.4 NA	

Annex F.4 NA	Calculation models for reinforcement of embankments on weak soil	Informative	
Annex G	Soil nailed structures	Not applicable	
Annex H	Rock Bolts and rock surface support	Not applicable	
Annex I	Ground improvement	Not applicable	
Annex J	Ground water control	<u>Informative with changes in annex J.3 NA</u>	
Annex J.3 NA	Examples of ground water control measures	Informative	
Annex K	Thermoactive geostructures design	Informative	

NOTE

Unchanged: Recommendations in the Eurocode to be followed.

National choice: A National Determined Parameter is chosen and defined.

Complementary information: (Non-contradictory) complementary information on how to use the Eurocode.

Normative: The Annex shall be used.

Informative: The Annex may be used.

Informative, see Annex X.1 NA (2): The Annex may be used accordingly to conditions given in Annex X.1. NA (2)

Not applicable: The Annex cannot be used.

No further information: The Eurocode allows further information. No further information is given.

National Determined Parameter (NDP) and Non-contradictory complementary Information (NCCI)

4.2.7.2(1) Minimum extent of field investigation

(NDP)

See Table 4.1 NA

Table 4.1 NA — Minimum depth of field investigation of slopes, cuttings, and embankments

Application	Minimum depth	Illustration
Cuttings	$d_{min} = 2,0 h$	
Embankments ^a	$d_{min} = \max \left\{ \begin{array}{l} 1,2 h \\ h_{com} + 2 m \end{array} \right.$	
a	h_{com} is the depth to the bottom of the compressible soil layers or of the zone of influence. h_{com} is measured from the same position as d_{min} .	

4.6.3(1) Partial factors

(NDP)

See Table 4.2 NA. Only the notes are modified.

Table 4.2 NA — 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
Overall stability	Action	γ_F	VC3 ^a
	Ground properties ^c	γ_M	M2 ^b
Bearing resistance	See Clause 5		
a	Values of the partial factors for actions (VC3) are given in NA to DS/EN 1990, Table A.1.8 NA and Table A.2.10 NA.		
b	Values of the partial factors for ground properties (M2) are given in NA to DS/EN 1997-1, Table 4.8 NA.		
c	Also includes ground properties of Class AI ground improvement (see Clause 12).		

5.2.4.2(6) Permanent and variable actions

(NCCI)

Medium- to very high-plasticity clays with $IP > 15\%$ may undergo significant volume changes as a result of variations in water content caused by wetting and drying. This may induce vertical and horizontal movements of spread foundations. Such effects can be mitigated through appropriate foundation design measures, including increased foundation depth, reinforcement, drainage, membranes and restrictions on trees and other vegetation in the vicinity of the foundation.

5.2.4.4(4) Environmental influences

(NCCI)

Frost-safe depth for foundations may normally be set at 0,9 m for conventional buildings and at 1,2 m for detached structures. The depth may be reduced by insulation.

5.2.7.2(1) Minimum extent of field investigation

(NDP)

See Table 5.1 NA

Table 5.1 NA — Minimum depth of field investigation for spread foundations

Application	Minimum depth ^{a,b}	Illustration
Square or circular spread foundation	$d_{min} = \max \left\{ \frac{3B}{6} \right\}$	
Strip spread foundation	$d_{min} = \max \left\{ \frac{5B}{6} \right\}$	
Raft or structure supported by a group of spread foundations	$d_{min} = \max \left\{ \frac{1,5B}{6} \right\}$	
a	B is related to the foundation system behaviour: the width of the shallow foundation for a single or a strip foundation, the width of the foundation group for a structure supported by many shallow foundations (to take into account group effects).	
b	The specified minimum depths of field investigation of 6 m applies to foundations for multistorey buildings and civil engineering projects. For structures in GCC1 the minimum depth of 6 m can be reduced to 2 m.	

5.5.4 (4) Settlement

(NCCI)

This depth may normally be taken as the depth at which the effective vertical stress due to the foundation load is 20 % of the effective overburden stress.

5.6.1(3) General

(NCCI)

When designing spread foundations, the vertical bearing resistance must be verified in both the short-term and long-term conditions, corresponding to undrained and drained failure in the ground, respectively.

However, for foundations on fine (cohesive) soils, it will be sufficient to verify undrained failure, except for:

Structures in GCC3

Heavily pre-consolidated clays with $c_{u,rep} > 350 \text{ kN/m}^2$

Foundations near sloping terrain or excavations.

5.6.1(5) General

(NDP)

The bearing resistance of spread foundations must not be verified by prescriptive rules.

5.6.3(5) Bearing failure

(NDP)

Foundations are considered to be highly eccentrically loaded when the load eccentricity exceeds 0,3 times the foundation width ($e > 0,3 B$). Design specification for bearing resistance for that situation appears in Annex B.4.6 NA.

5.6.3(6) Bearing failure

(NCCI)

The calculation formulas for highly eccentrically loaded foundations in Annex B.4.6 NA cannot be used uncritically. Highly eccentric loading of foundations can only be permitted for short-term horizontal loads, e.g. wind acting parallel to the short side of the foundation (B').

Unusual load combinations (e.g. horizontal load T acting in the opposite direction of the eccentricity) and special geometric designs of foundations will require numerical modelling.

5.6.3(7) Bearing failure

(NDP)

The values of ΔB and ΔD is 0,1 m.

Smaller values of ΔB and ΔD , including 0,0 m, may be used if it is specified that the foundation width must be controlled during the foundation work.

5.6.6(1) Partial factors

(NDP NOTE 1)

The Material Factor Approach (MFA) is used, see Table 5.2 NA

(NDP NOTE 2)

See Table 5.2 NA

(NDP NOTE 3)

Only the single combination (c) is used, see Table 5.2 NA

Table 5.2 NA — 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, either both combinations (a) and (b) or the single combination (c)			Resistance Factor Approach, either combination (d) or (e) ^c	
			(a)	(b)	(c) ^d	(d)	(e)
Overall stability	See Clause 4						
Bearing and sliding resistance	Actions, Effects of actions	γ_F			VC1 ^a		
	Ground properties	γ_M			M2 ^b		
	Bearing resistance	γ_{RN}	Not factored				
	Sliding resistance	γ_{RT}	Not factored				
	Passive resistance	$\gamma_{RT,face}$	Not factored				
a	Values of the partial factors for actions (VC1) are given in DS/EN 1990:2023 DK NA, Table A.1.8 NA and Table A.2.10 NA.						
b	Values of the partial factors for ground properties (M2) are given in DS/EN 1997-1:2024 DK NA Table 4.8 NA.						
c	Is not relevant, as the MFA is used in DK						
d	In this combination (c) the consequence factor on material properties is omitted						

5.6.6(3) Partial factors

(NDP)

As the Resistance Factor Approach (RFA) is not used for verification of spread foundations, the associated reduction factor for transient design situations, $k_{R,tr}$, is not relevant either.

5.7.5(1) Physical gap beneath the foundation

(NDP)

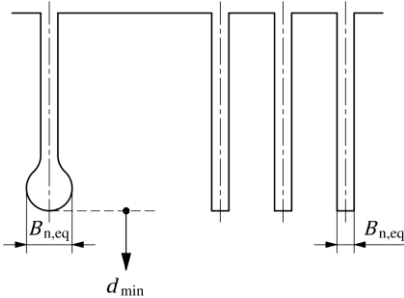
Table 5.3 is not used.

6.2.7.2(1) Minimum extend of field investigation

(NDP)

See Table 6.2 NA

Table 6.2 NA — Minimum depth of field investigation for piled foundations

Application	Minimum depth	Illustration
Single piled foundation	$d_{\min} = \max \{3 \text{ m}; 3B_{n,\text{eq}}\}$	
Pile groups or piled rafts in soils and in very weak and weak rock	$d_{\min} = \max \{5 \text{ m}; 3B_{n,\text{eq}}; p_{\text{group}}\}$	
Single piles, pile groups or piled rafts in strong rock	$d_{\min} = \max \{2 \text{ m}; 3B_{n,\text{eq}}\}$	

$B_{n,\text{eq}}$ 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_{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

6.5.3.1(2) Calculation

(NDP)

Both the Ground Model Method (GMM) and the Model Pile Method (MPM) can be used. When using the MPM the representative bearing capacity shall be determined according to DS/EN 1997-1:2024 DK NA, Annex A NA in consistency with DS/EN 1990:2023, Annex D.7.2.

The correlation factors in Table 6.6, 6.7 and 6.8 are not used.

6.5.3.3(8) Testing

(NDP)

If more than approximately 30% of the total pile resistance is provided in fine soils, dynamic impact testing should only be used if the results are calibrated against a site-specific static load test or against geotechnical resistance calculated in accordance with Annex C NA.

6.5.7.2(1) General

(NCCI)

If load tests are available, the settlement of the single pile can be estimated on the basis of the load-displacement curve. If the pile settlement for the service load is greater than the permissible project-specific

limit value for the supported structure, it will be necessary to reduce the service load (and the corresponding design load as well).

6.5.8(4) Confirmation of pile design by testing or comparable experience

(NDP)

see Table 6.3 NA

Table 6.3 NA — Minimum quantity of load testing for verification according to Table 6.4 NA

Type of load test	Confirmation of design by ultimate pile load tests
Static Load Test (SLT)	$\max\left(\begin{matrix} 1^a \\ 0,5 \% n_{total} \end{matrix}\right)$
Dynamic Impact Load Test (DLT)	$\max\left(\begin{matrix} 3 \\ 1,0 \% n_{total} \end{matrix}\right)$
NOTE	n_{total} = expected total number of piles per population (similar type, length and execution method in similar ground conditions). Number of SLT on screw piles appear from Annex C.4 NA
a	If only one pile load test is carried out, it shall be located where the most adverse ground conditions are expected to occur.

6.6.2.1(2) Axial compressive resistance

(NDP NOTE 1)

See Table 6.4 NA

(NDP NOTE 2)

Table 6.5 is not use. Model factors is given in Table 6.4 NA.

Table 6.4 NA — Model factor $\gamma_{Rd,pile}$ for verification by GMM and MPM

Verification based on	Assisted by	Model factor $\gamma_{Rd,pile}$
Calculation (from ground properties)	-	1,5
	DLT	1,4 (1,25)
	SLT	1,25 (1,1)
Pile Driving Formula (The Danish Driving Formula)	-	1,5
	DLT	1,4 (1,25)
	SLT	1,25 (1,1)
NOTE	DLT = Dynamic Impact Load Testing SLT = Static Load Testing Minimum number of DLT or SLT to justify the model factors according to Table 6.3 NA	

Numbers in brackets are for the tested piles

6.6.2.4.2(1) Model Pile Method

(NDP)

Table 6.6 is not used.

The characteristic bearing capacity by MPM shall be determined according to DS/EN 1997-1:2024 DK NA, Annex A NA in consistency with DS/EN 1990:2023, Annex D.7.2.

6.6.2.4.3(1) Testing

(NDP)

Table 6.7 and Table 6.8 are not used. The characteristic bearing capacity by MPM shall be determined according to DS/EN 1997-1:2024 DK NA Annex A NA in consistency with DS/EN 1990:2023, Annex D.7.2.

6.6.2.5(1) Structural resistance

(NCCI)

For screw piles the verification of the structural resistance shall include verification of the threads and/or helix plates according to the relevant structural codes taking design service life into account.

6.6.2.5(6) Structural resistance

(NCCI)

NOTE 3 The representative effective stiffness of reinforced concrete piles may be estimated by using the model as specified in DS/EN 1992-1-1:2023 DK NA, NCCI 0.4 (2).

NOTE 4 For traditional precast concrete piles buckling and second-order effects can be disregarded in soft soil with $c_{u,rep} > 10$ kPa, provided the mean design concrete compression stress does not exceed 10 MPa.

6.6.3(4) Ultimate limit state for pile groups and piled rafts

(NDP)

$$\gamma_{Rd,group} = 1,5$$

6.6.3(5) Ultimate limit state for pile groups and piled rafts

(NDP)

$$\gamma_{Rd,piled-raft} = 1,5$$

6.6.4.1(1) Single piles

(NDP NOTE 1)

See Table 6.9 NA.

Table 6.9 NA — Partial factors for the verification of ultimate resistance of single piles for fundamental (persistent and transient) design situations – Ground Model Method, Model Pile Method and Design by testing

Verification of	Partial factor on	Symbol	Material Factor Approach – both combinations		Resistance Factor Approach		
			(a) ^c	(b) ^d	Pile class		
Axial compressive resistance	Actions, Effects of actions ^a	γ_F, γ_E	Not used		All	VC1	
	Drag force	$\gamma_{F,drag}$				1,0	
	Ground Properties ^b	γ_M				Not factored	
	Base and shaft resistance in compression	γ_{Rb}, γ_{Rs}				Base	Shaft
					Full displacement	1,3	
					Partial displacement		
					Replacement		
	Total resistance in compression	γ_{Rc}			Full displacement		
					Partial displacement		
					Replacement		
Axial tensile resistance	Actions, Effects of actions ^a	γ_F, γ_E	Not used		All	VC1	
	Ground Properties ^b	γ_M				Not factored	
	Shaft resistance in tension	γ_{Rst}			Full displacement	1,3	
					Partial displacement		
					Replacement		
Transverse resistance	Actions, Effects of actions ^a	γ_F, γ_E	VC4	VC1	Not used	Not used	
	Transverse ground load	$\gamma_{F,tr}$					
	Ground Properties ^b	γ_M	M1	M2			
	Transverse resistance	γ_{Rtr}	Not factored				
^a Values of the partial factors for Verification Cases (VCs) 1, 3, and 4 are given in DS/EN 1990:2023 DK NA, Annex A NA.							

- | | |
|---|---|
| b | Values of the partial factors for Sets M1 and M2 are given in DS/EN 1997-1:2024 DK NA, 4.4.1.3, Table 4.8 NA. |
| c | Combination (a) only in case of Numerical modelling (FEM analysis). |
| d | In this combination (b) the consequence factor on the materials is omitted. |

Table 6.10 is not used. Partial factors are applied with MPM as with GMM and included in Table 6.9 NA

Table 6.11 is not used. Design by testing is included in Table 6.4 NA.

6.6.4.2(1) Pile groups and piled rafts

(NDP NOTE 1)

See Table A.12 NA.

Table 6.12 NA — Partial factors for the verification of ultimate resistance of pile groups and piled rafts for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Symbol	Material Factor Approach – Both combinations		Resistance Factor Approach
			(a)	(b)	
Vertical resistance	Actions, Effects of actions ^a	γ_F, γ_E	Not used		VC1
	Ground Properties ^b	γ_M			Not factored
	Vertical resistance	$\gamma_{R,group}$			1,3
		$\gamma_{R,piled-raft}$			1,3
Combined axial and transverse resistance (see EN 1997-1:2024, 8.2)	Actions, Effects of actions ^a	γ_F, γ_E	VC4	VC1	Not used
	Ground Properties ^b	γ_M	M1	M2	
	Compressive and transverse resistance	$\gamma_{R,group}$	Not factored		
a	Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in DS/EN 1990:2023 DK NA, Annex A NA				
b	Values of the partial factors for Sets M1 and M2 are given in DS/EN 1997-1:2024 NA, Table 4.8 NA				

(NDP NOTE 2)

The national choice of factor approach appears from Table 6.12 NA.

6.7(4) Serviceability limit states

(NDP)

Under certain conditions settlement calculation for piles loaded in compression can be replaced by an equivalent failure analysis according to Annex C.9.3 NA.

6.9.3(6) Test proof load

(NDP)

SLS load tests are not carried out, for which reason γ_{test} test is not relevant.

6.9.4(1) Static load tests

(NCCI)

The two commonly used types of static pile load tests are Maintained Load Test (MLT) and Constant Rate of Penetration Test (CRP-test).

The pile load tests considered in EN ISO 22477-1 and EN ISO 22477-2 are limited to MLTs.

As an alternative or supplement to a MLT, a CRP test can be carried out according to Annex C.15 NA. This method provides a continuous load-displacement curve that allows unambiguous determination of the ultimate pile resistance.

7.4.1(1) General

(NDP)

See Annex D.2.A.

7.5.3.2(2) Active earth pressure

(NDP)

Negative effective earth pressure (tension) is not allowed, i.e. $p'_{a,\min}$ is zero (0).

7.5.3.2(3) and (7) Active earth pressure

(NCCI)

The formulas in the code text regarding active earth pressure involving cohesion presupposes absolute values of the earth pressure coefficient K_c .

7.5.3.2(5) Active earth pressure

(NDP)

Negative total earth pressure (tension) is not allowed, i.e. $p_{a,\min}$ is zero (0).

7.6.2(6) Overall stability

(NDP)

$\gamma_{\text{Rd,retain}} = 1,0$ in all cases

7.6.7(2) Supporting elements

(NCCI NOTE 3)

$\gamma_F = \gamma_E$ (see DS/EN 1990:2023 DK NA, Table A.1.8 NA).

7.6.9(1) Partial factors

(NDP NOTE 1)

MFA is used for retaining structures

(NDP NOTE 2)

See Table 7.2 NA.

Table 7.2 NA — Partial factors for the verification of ground resistance against retaining structures for fundamental (persistent and transient) design situations

Verification of	Partial factor on	Sym- bol	Material Factor Approach –combinations (a), (b) and (c)			Resistance Factor Ap- proach	
			(a)	(b)	(c) ^d	(d)	(e)
Overall stability	See Clause 4						
Bearing/sliding resistance of gravity walls	Actions, Effects of actions	γ_F, γ_E	VC4 ^a	VC3 ^a	VC1 ^a		
	Ground proper- ties ^e	γ_M	M1 ^b	M2 ^b	M2 ^b		
	Bearing re- sistance	γ_{RN}	Not factored				
	Sliding re- sistance	γ_{RT}					
Bearing/rotational re- sistance of embedded walls Basal heave ^c	Actions, Effects of actions	γ_F, γ_E	VC4 ^a	VC3 ^a	VC1 ^a		
	Ground proper- ties	γ_M	M1 ^b	M2 ^b	M2 ^b		
	Vertical re- sistance, basal heave	γ_R	Not factored				
	Passive earth resistance	γ_{Re}					
a	Values of the partial factors for Verification Cases (VCs) 1, 3 and 4 are given in EN 1990:2023 DK NA, Annex A NA.						
b	Values of the partial factors for Sets M1 and M2 are given in DS/EN 1997-1:2024 DK NA, tabel 4.8 NA.						
c	For basal heave, see Annex D						
d	In combination (c) the consequence factor on the material properties is omitted						

8.6.2(1) Geotechnical resistance

(NDP)

Suitability tests are mandatory, and they shall be carried out before commencement of the subsequent acceptance tests.

Investigation tests can be carried out in advance of the suitability test.

The aim of investigation tests is (or can be):

To identify a realistic and robust proof load for suitability tests

To optimise (minimise) the bond length

To investigate behaviour and geotechnical ultimate resistance of new anchor types

To investigate potential geotechnical resistance of anchors in uncommon soil

To estimate the residual bearing capacity beyond failure (pull out)

To estimate reserve (robustness) of resistance between load at creep limit ($\alpha_1 = 2$ mm) and failure (pull out) for ductility evaluation

8.6.2(4) Geotechnical resistance

(NCCI)

The number of suitability tests by Test Method 1 shall correspond to at least 2 % of the expected number of production anchors, with a minimum of 3 anchors.

Suitability tests may be upgraded to investigation tests if they are continued until failure, but then supplementary suitability tests on production anchors must be carried out.

The criteria for production anchors to belong to the same population as the test anchors are same ground conditions in the bond zone such as:

soil type (fine or coarse soil, or rock)

strength

normal effective stress (for anchors in coarse soil), depending on e.g. water level and depth below ground level.

Requirements and recommendations on the similarities between test anchors and production anchors regarding characteristics of the installation procedure, borehole diameter, inclination, tendon and fixed length appear from DS/EN ISO 22477-5, clause 6.2 Test anchors.

Requirements on selection of location of test anchors appear from DS/EN ISO 22477-5 clause 6.1 Test location.

8.6.2(6) Geotechnical resistance

(NCCI)

The geotechnical ultimate limit state resistance of a grout-ed test anchor $R_{am,ULS}$ is identified as the load relating to the physical pull out resistance ($R_{am,\alpha ULS}$) defined by a vertical asymptote of a creep rate versus proof load diagram or, where such is difficult to determine, by a creep rate of 5 mm.

If that load is not reached ($P_p < R_{am,\alpha ULS}$), the Investigation Test has failed, and the $R_{am,ULS}$ has not been determined, even though it is defined as the minimum of $R_{am,\alpha ULS}$ and P_p in formula (8.2).

8.6.2(7) Geotechnical resistance

(NDP)

The values of α_1 are given in Table 8.4 NA.

8.6.2(9) Geotechnical resistance

(NDP)

No value is stated for ξ_{ULS} as the possible design value of the anchor force E_d , corresponding to the design value of the geotechnical resistance at the ultimate limit state $R_{ad,ULS}$, is limited by the possible proof load P_p for the suitability test according to formula (8.12):

$$E_d \leq \frac{P_p}{\xi_{a,ULS,test} \cdot \gamma_{Ra,ULS}} \quad (8.12)$$

8.6.4(1) Partial factors

(NDP NOTE 1)

Only Test Method 1 is used.

(NDP NOTE 2)

See Table 8.2 NA.

Table 8.2 NA — 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	
			Test Method 1	Test Method 3
Geotechnical resistance of an anchor	Geotechnical resistance at the ultimate limit state	$\gamma_{Ra,ULS}$	1,3 ^{a,b}	Not used
a	See Formula (8.5)			
b	See Formulae (8.12) and (8.14)			

8.9.1(2) General

(NDP)

Test method 1 is used.

See Table 8.4 NA.

Table 8.4 NA — Limiting criteria for investigation, suitability and acceptance tests at the ultimate and serviceability states

Test method _c	Parameter _a	Anchor type	Investigation test α_{ULS}	Suitability test		Acceptance test	
				α_{ULS}	α_{SLS}	α_{ULS}	α_{SLS}
				(8.12) ^b	(8.13) ^b	(8.14) ^b	(8.15) ^b
1	α_1	All	5 mm ^d	2 mm ^e	Not used	2 mm ^e	Not used
a Creep rate per log cycle of time. b Reference to applicable formula. c For a description of the test methods, see DS/EN ISO 22477-5:2018. Test method 1 is mandatory. d Failure (physical pull-out resistance) is defined by a vertical asymptote of a creep rate versus proof load diagram or, where such is difficult to determine, by a creep rate of 5 mm. e Limit creep rate for test with extended observation period.							

8.9.1(2) General

(NCCI)

By long term tests (over several days), load loss measurements, ΔN , (by locked anchor), may be used as an alternative to measurements of creep by retained load. The load loss measurements can be converted to equivalent creep, Δs , based on a free length corresponding to the apparent free length, L_{app} , determined from the unloading of the anchor at the last load cycle before lock-off.

Equivalent creep:

$$\Delta s = \Delta N * \frac{L_{free}}{E_t * A_t} \quad (8.a \text{ NA})$$

where ΔN is the load loss

L_{free} is L_{app}

E_t is the elastic modulus of the tendon

A_t is the cross-sectional area of the tendon

NOTE For ground anchors with different free lengths, load loss measurements cannot be compared, since the load loss is dependent on the free length.

8.9.3(1) Suitability tests

(NDP)

$$\xi_{a,ULS,test} = 1,1$$

$\xi_{a,SLS,test}$ is not relevant as Test Method 3 is not used.

9.2.7.2(1) Minimum extent of field investigation

(NDP)

The most adverse minimum depth $d_{\min}$ calculated based on the principles from Table 4.1 (Height of slope), Table 5.1 (width of reinforced earth structure) and Table 7.1 (height of supported ground) shall be governing, as presented in Table 9.1.

9.3.3(6) Geosynthetics

(NDP)

Table 9.a NA — η_{cr} , damage parameter for deformation under long term loading (creep)

Material	Damage parameter, η_{cr}
Steel	0,8
Polyester (PETP)	0,4
Polypropylene (PP)	0,2
Polyamide (PA)	0,35
Polyethylene	0,2

After "Nordic guidelines for reinforced soils and fills", and "Håndbok V220 Geoteknikk i vegbygging"

Table 9.b NA — η_{dmg} , damage parameter for damages sustained during construction

Material	Damage parameter, η_{dmg}
Clay / Silt	0.91
Sand	0.83
Gravel (natural)	0.77
Gravel (crushed)	0.72
Crushed rock	0.67

After "Nordic guidelines for reinforced soils and fills", and "Håndbok V220 Geoteknikk i vegbygging".

η_{w} , damage parameter for deterioration under UV radiation (sun)

If geosynthetics are exposed to sunlight (UV radiation) under installation or storage for a maximum of 12 hours, no damage parameter is applied. Thus, $\eta_{\text{w}} = 1,0$.

If the geosynthetics are exposed for longer than 12 hours, the strength of the material must be documented after EN 12224 (accelerated weathering index test). If it can be documented that the loss of strength is less than 5% or statistically insignificant, no damage parameter is applied. Thus, $\eta_{\text{w}} = 1,0$. However, it must be ensured that the geosynthetics are covered within one month (30 days).

If the geosynthetics reflect a larger loss of strength after EN 12224, the maximum allowable exposure time and the damage factor from Table 9.c NA (After ISO/TS 20432) must be applied.

Table 9.c NA — η_w , damage parameter for deterioration under UV radiation (sun)

Strength after exposure, cf. EN 12224	Maximum allow exposure (un-covered)	Damage parameter, η_w
> 80%	1 month ^a	The ration between the tensile strength of exposed vs. non-exposed material: $\eta_w = \text{exposed strength} / \text{intact strength} < 1,0$
60% to 80%	2 weeks	0,8
<60%	1 day	1
Untested material	1 day	1
^a longer exposure time may be allowed dependent on-site location and season, however never more than four months.		

If it is anticipated that the geosynthetics are exposed to sunlight for more than one month, the materials must be tested according to EN 12224.

Table 9.d NA — η_{ch} damage parameter for chemical deterioration

pH interval	Design life (years)	Temperature (°C)	η_{ch}
4 – 9	25	25	1,0
4 – 8	100	25	0,83
8 – 9	100	25	0,76
4 – 9	25	35	0,71

After ISO/TS 20432.

η_{dyn} damage parameter for dynamic loading

η_{dyn} is calculated after Table 9.e NA based on the ratio between static and dynamic loading $\zeta = 2 \sigma_{dyn,k} / \sigma_{stat,k}$. Only permanent loads are included in $\sigma_{stat,k}$.

Table 9.e NA — η_{dyn} damage parameter for dynamic loading

Interval	η_{dyn}	Remark
$\zeta = 0$	1,0	Static load
$0 < \zeta < 1$	1,0 til 0,66	Linear interpolation of ζ
$\zeta = 1$	0,66	

After “Recommendations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements – EBGE0”

9.6.2.1(1) Geosynthetics

(NDP)

The model factor is $\gamma_{Rd,gs} = 1,0$.

9.6.7(1) Partial factors

(NDP NOTE 1)

Material Factor Approach shall be used for verification of failure mechanisms unless otherwise stated Clause 4, Clause 5 and Clause 7.

(NDP NOTE 2)

See Table 9.4 NA.

Table 9.4 NA — Partial factors for the verification of resistance of reinforced fill structures for fundamental (persistent and transient) design situations

Verification of	Partial factor on		Symbol	Material Factor Approach	Resistance Factor Approach
Overall and compound failure mechanisms	See Clause 4				
Bearing resistance and sliding	See Clause 5				
Overturning	See Clause 7				
Internal failure mechanisms	Actions, Effect of actions		γ_F, γ_E	VC3 ^a	-
	Ground properties		γ_M	M2 ^b	-
Pull-out and direct shear	Pull-out resistance of reinforcing elements		$\gamma_{R,po}$	Not factored	-
	Resistance to direct shear along interface		$\gamma_{R,ds}$	Not factored	-
Rupture of reinforcing elements	Tensile resistance of	geosynthetic reinforcing elements	$\gamma_{M,gs}$	1,3	
		structural steel per EN 10025-2 or EN 10025-4	γ_{M0}	specified in EN 1993-1-1	
			γ_{M2}	specified in EN 1993-1-1	
		reinforcing steel per EN 10080	γ_s	specified in EN 1992-1-1	
			γ_t	1,3	
		polymeric coated steel wire mesh reinforcing elements	$\gamma_{M,pwm}$	1,3	
Rupture of connections to facing	Tensile resistance at connection	Reinforcing elements	$\gamma_{R,con,el}$	As specified above for rupture of reinforcing elements	
		Connectors	$\gamma_{R,con,c}$	As specified in the relevant material Eurocode	
		Facing elements	$\gamma_{R,con,f}$	As specified in the relevant material Eurocode	
a	Values of the partial factors for Verification Cases (VCs) 3 and 4 are given in DS/EN 1990:2023 DK NA, Annex A NA.				
b	Values of the partial factors for sets M1 and M2 are given in DS/EN 1997-1:2024 DK NA, table 4.8 NA.				

13.3(4) Materials

(NCCI)

DS/INF 466 may be applied for membranes of clay, bentonite and polymer.

Annex A NA Slopes, cuttings and embankments (informative)

This informative Annex A NA provides following changes to the informative annex A.

A.3 NA Calculation models for analysing the stability of soil and fill

Table A.1: Note a is deleted.

Annex B NA Spread foundations (Normative)

B.1 NA Use of this annex (normative)

(1) This annex provides complementary provisions to the general rules in Clause 5 regarding spread foundations.

(2) The provisions given in the informative annexes B.6 to B.16 are not in accordance with established Danish geotechnical practice. Accordingly, these annexes may only be used if it can be demonstrated - based on reliable documentation or relevant experience - that their use yields results for Danish conditions that are on the safe side for the limit states considered.

B.4 NA Calculation model for bearing resistance using soil parameters (normative)

B.4.1 NA General

(1) The design vertical bearing resistance of a foundation shall be verified using appropriate drainage conditions, see 5.6.1(3). Further guidance is found in (NCCI) to 5.6.1(3).

(2) The effects of the following should be considered:

The strength of the soil, generally represented by the design values of c_u , c' and φ' ;

The eccentricity and inclination of design actions;

The shape and depth of the foundation;

The slope of the ground surface;

Ground water pressures and hydraulic gradients;

The variability of the soil, especially layering.

(3) A general definition of competent (load bearing) soils cannot be given. Examples of deposits that cannot be regarded as competent without special measures include gyttja, peat, post-glacial clay, topsoil, uncontrolled fill, and disturbed or frozen soil.

(4) In formulas specified in B.4, design values of loads and strengths shall be used, according to the Material Factor Approach (MFA).

(5) The equations for the design vertical bearing resistance specified in clauses B.4.2 to B.4.5 apply to a horizontal foundation base and ground surface, the same effective stress from overburden on each side of the foundation and with the soil strength represented by the values of c_u , c' , φ' and γ' , that are assumed constant for the ground volume governing the limit state.

(6) Any effect from embedment depth and resistance above the foundations base is disregarded.

(7) For coarse soil the effective angle of friction φ' is equal to the plane strain value of the effective angle of friction, see DS/EN 1997-2 DK NA E.4.5, formula (E.a NA). For fine soil, there is no equivalent correction.

(8) The formulas given for calculation of the bearing resistance in B.4.2 to B.4.5 apply to loads acting eccentrically with an eccentricity, e , not exceeding $0,30 B$. For strong eccentric loading, $e > 0,30 B$, see B.4.6.

(9) The foundation is loaded with a vertical load, N , normal to the base and a horizontal load $T = \sqrt{T_B^2 + T_L^2}$ at the base.

NOTE 1 Subscripts B and L denote the foundation effective width, B' , and length, L' , respectively.

NOTE 2 For a horizontal load T not aligned with the short side of the foundation B' , the bearing resistance is determined through a general procedure by two independent calculations: one for load component T_B parallel to the short side and one for load component T_L parallel to the long side as presented in sub clause B.4.3 and B.4.5 below.

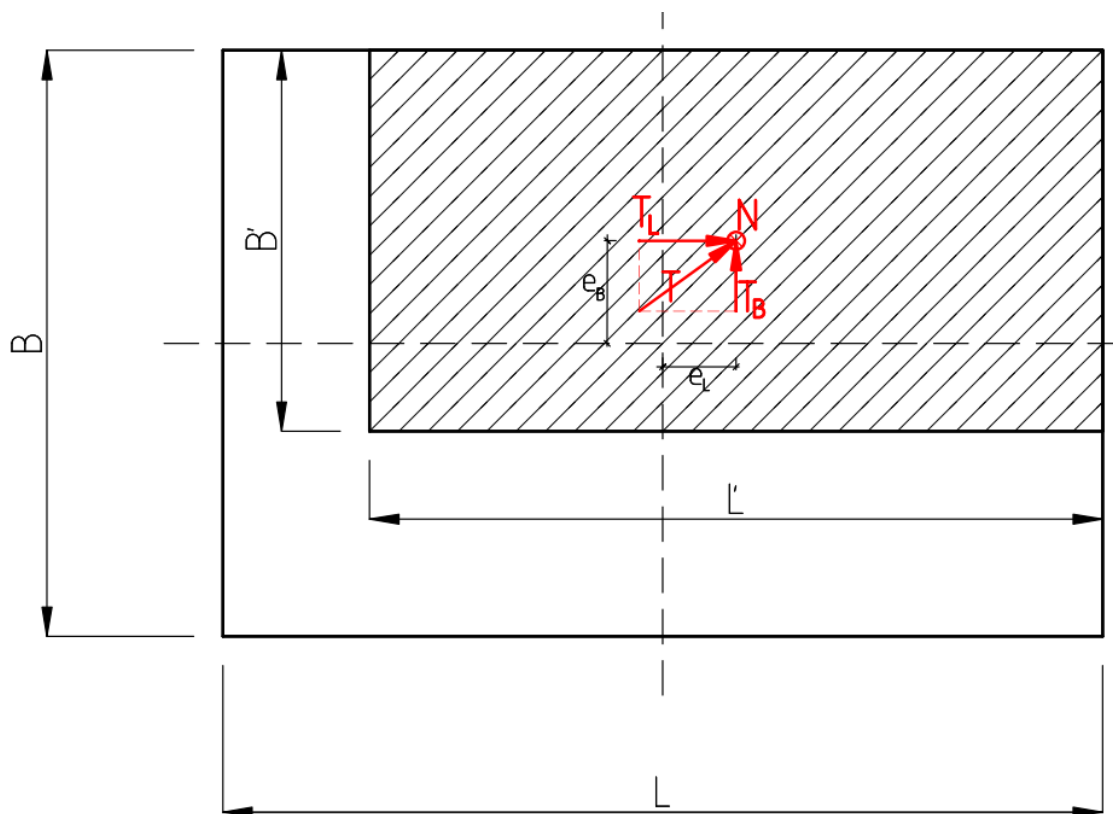


Figure B.4.a NA - Plan of foundation with excentric load

(10) The sliding resistance of a foundation shall be investigated according to 5.5.3 in DS/EN 1997-3. If the sliding resistance is combined with other stabilizing forces (e.g. passive earth pressure), all stabilizing effects shall be assessed, taking strain compatibility into account

B.4.2 NA Drained bearing resistance. Basic formula

(1) For the basic situation with T predominantly aligned with the shortest side, B' , the drained bearing resistance $\frac{R_N}{A'}$ normal to the foundation base may be determined from effective stress analysis as:

$$\frac{R_N}{A'} = 0,5\gamma'B'N_\gamma s_{\gamma B} i_{\gamma B} + q'N_q s_{qB} i_{qB} + c'N_c s_{cB} i_{cB} \quad (\text{B.4.a NA})$$

where A' is the effective plan area of the foundation;

B' is the effective foundation width;

c' is the soil's effective cohesion;

q' is the effective stress from overburden at foundation level;

γ' is the effective weight density below foundation base;

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

$i_{\gamma B}$, i_{qB} and i_{cB} are non-dimensional factors accounting for load inclination;

$s_{\gamma B}$, s_{qB} and s_{cB} are non-dimensional factors accounting for the shape of the foundation.

(2) The drained bearing resistance factors may be calculated from:

$$N_\gamma = \frac{1}{4} \left((N_q - 1) \cos \varphi' \right)^{\frac{3}{2}} \text{ for a rough base } \delta > \frac{\varphi'}{2} \quad (\text{B.4.b NA})$$

$$N_q = e^{\pi \tan \varphi'} \tan^2 \left(45 + \frac{\varphi'}{2} \right) \quad (\text{B.4.c NA})$$

$$N_c = (N_q - 1) \cot \varphi' \quad (\text{B.4.d NA})$$

where φ' is the plane, effective peak angle of friction (in degrees);

δ is the angle of interface friction between the foundation and the ground.;

(3) For the basic situation with T aligned with B' , the shape- and inclination factors may be calculated from:

$$s_{\gamma B} = 1 - 0,4 \left(\frac{B'}{L'} \right) \quad (\text{B.4.e NA})$$

$$s_{qB} = s_{cB} = 1 + 0,2 \left(\frac{B'}{L'} \right) \quad (\text{B.4.f NA})$$

$$i_{qB} = i_{cB} = \left(1 - \frac{T_B}{N + A' c' \cot(\varphi')} \right)^2 \quad (\text{B.4.g NA})$$

$$i_{\gamma B} = i_{qB}^2 \quad (\text{B.4.h NA})$$

NOTE 1 It is conservative to assume $T_B = T$ regardless of the true direction of T , meaning that the formulas given above can be used as a conservative approach, if it is assumed that the resultant T is parallel to the shortest side of the foundation, B' , and pointing in the same direction as the eccentricity, e_B .

B.4.3 NA Drained bearing resistance. Extended formula

(1) For the situation where T is not parallel to B' , the following procedure may be followed.

NOTE 1 The procedure suggested below is a simplification of the procedure described by Brinch Hansen, J: "A revised and extended formula for bearing capacity", The Danish Geotechnical Institute Bulletin No. 28 (1970)

(2) In addition to the shape and inclination factors in the direction of B' as defined in B.4.2, the shape and inclination factors in the direction of L' may be calculated as follows:

$$s_{\gamma L} = 1 - 0,4 \left(\frac{B'}{L'} \right) \quad (\text{B.4.i NA})$$

$$s_{qL} = s_{cL} = 1 + 0,2 \left(\frac{B'}{L'} \right) \quad (\text{B.4.j NA})$$

$$i_{qL} = i_{cL} = \left(1 - \frac{T_L}{N + A' c' \cot(\varphi')} \right)^2 \quad (\text{B.4.k NA})$$

$$i_{\gamma L} = i_{qL}^2 \quad (\text{B.4.l NA})$$

(3) Two bearing resistance formulas are considered (failure in B' direction and L' direction):

B-direction:

$$\frac{R_N^B}{A'} = \underbrace{0,5 \gamma' B' N_{\gamma} s_{\gamma B} i_{\gamma B}}_{\text{"}\gamma\text{"-term}} + \underbrace{q' N_q s_{qB} i_{qB}}_{\text{"}q\text{"-term}} + \underbrace{c' N_c s_{cB} i_{cB}}_{\text{"}c\text{"-term}} \quad (\text{B.4.m NA})$$

L-direction:

$$\frac{R_N^L}{A'} = 0,5 \gamma' L' N_{\gamma} s_{\gamma L} i_{\gamma L} + q' N_q s_{qL} i_{qL} + c' N_c s_{cL} i_{cL} \quad (\text{B.4.n NA})$$

(4) The governing direction is found from:

If $i_{\gamma B} B \leq i_{\gamma L} L$, the B-direction governs

If $i_{\gamma B} B \geq i_{\gamma L} L$, the L-direction governs

Accordingly, the first term (γ -term) of the bearing resistance formula shall be taken from (B.4.m NA) in the former case and from (B.4.n NA) in the latter.

The q -term and c -term shall be taken as the smallest numerical values of (B.4.m NA) and (B.4.n NA)

(5) The final bearing resistance is then calculated as:

$$\frac{R_{N,final}}{A'} = \text{"}\gamma\text{"} + \text{"}q\text{"} + \text{"}c\text{"} \quad (\text{B.4.o NA})$$

where

- " γ " is γ -term from (B.4.i NA) if $i_{\gamma B'} B \leq i_{\gamma L'} L$ and from (B.4.j NA) if $i_{\gamma B'} B \geq i_{\gamma L'} L$;
- " q " is smallest numerical value of q -term from (B.4.m NA) or (B.4.n NA);
- " c " is smallest numerical value of c -term from (B.4.m NA) or (B.4.n NA).

(6) If T_B or T_L is pointing in the opposite direction of the eccentricity (e_B or e_L) the following load inclination factors apply:

$$i_q = i_c = i_\gamma = 1,0 \quad (\text{B.4.p NA})$$

B.4.4 NA Undrained bearing resistance. Basic formula

(1) For the basic situation with T predominantly aligned with the shortest side, B' , the undrained bearing resistance $\frac{R_{Nu}}{A'}$ may be determined from total stress analysis as:

$$\frac{R_{Nu}}{A'} = (\pi + 2) c_u s_{cB} i_{cB} + q \quad (\text{B.4.q NA})$$

where c_u is the undrained shear strength;

q is the total stress from overburden at foundation level.

(2) The shape and inclination factors in (B.4.q NA) may be determined from:

$$s_{cB} = 1 + 0,2 \left(\frac{B'}{L'} \right) \quad (\text{B.4.r NA})$$

$$i_{cB} = 0,5 \left(1 + \sqrt{1 - \frac{T_B}{A' c_u}} \right), T_B \leq A' c_u \quad (\text{B.4.s NA})$$

NOTE It is conservative to assume that $T_B = T$ regardless of the true direction of T , meaning that the formulas given above can be used as a conservative approach, if it is assumed that the resultant T is parallel to the shortest side of the foundation and pointing in the same direction as the eccentricity, e_B .

(3) For eccentrically loaded foundations on fine (cohesive) soil, potential destabilizing water pressure acting on the un-loaded part of the foundation shall be taken into account.

NOTE This can most conveniently be done by assuming hydrostatic water pressure at the foundation base and replace the total stress from overburden, q , with the effective stress from overburden, q' , in (B.4.q NA) which provides $\frac{R_N}{A'}$.

(4) If T_B is pointing in the opposite direction of the eccentricity (e_B) the following load inclination factor applies:

$$i_{cB} = 1,0 \quad (\text{B.4.t NA})$$

B.4.5 NA Undrained bearing resistance. Extended formula

(1) For the situation where T is not parallel to B' , the following procedure may be followed.

NOTE The procedure suggested below is a simplification of the procedure described by Brinch Hansen, J: "A revised and extended formula for bearing capacity", The Danish Geotechnical Institute Bulletin No. 28 (1970)

(2) The undrained bearing resistance in L-direction is:

$$\frac{R_{Nu}}{A'} = (\pi + 2) c_u s_{cL} i_{cL} + q \quad (\text{B.4.u NA})$$

(3) In addition to the shape and inclination factors in the direction of B' as defined in (B.4.r NA) and (B.4.s NA), the shape and inclination factors in the direction of L' may be calculated as follows:

$$s_{cL} = 1 + 0,2 \left(\frac{L'}{B'} \right) \quad (\text{B.4.v NA})$$

$$i_{cL} = 0,5 \left(1 + \sqrt{1 - \frac{T_L}{A' c_u}} \right), T_L \leq A' c_u \quad (\text{B.4.x NA})$$

(4) The final bearing resistance in the undrained case is then the smallest numerical value of (B.4.q NA) or (B.4.u NA).

(5) If T_L is pointing in the opposite direction of the eccentricity (e_L) the following load inclination factor applies:

$$i_{cL} = 1,0 \quad (\text{B.4.y NA})$$

B.4.6 NA Strong eccentricity

(1) For foundations that are strongly eccentrically loaded (i.e., with an eccentricity greater than 0,3 times the foundation width), it is necessary to evaluate the bearing resistance for a failure mechanism extending beneath the unloaded part of the foundation.

(2) For effective stress (drained) analysis this can be done by multiplying the γ -term and the c -term in the general bearing resistance formula (B.4.a NA) by 2 for the γ -term, and $(1,05 + \tan^3 \varphi')$ for the c -term, and neglecting the q -term:

$$\frac{R_N}{A'} = \gamma' B' N_\gamma s_{\gamma B} i_{\gamma B}^e + (1,05 + \tan^3 \varphi') N_c s_{cB} i_{cB}^e \quad (\text{B.4.z NA})$$

where special load inclination factors, $i_{\gamma B}^e$ and i_{cB}^e apply:

$$i_{\gamma B}^e = 1 + 3 \frac{T}{N} \quad (\text{B.4.æ NA})$$

$$i_{cB}^e = 1 + 4 \tan \varphi' \frac{T}{N} \quad (\text{B.4.ø NA})$$

(3) For total stress (undrained) analysis this can be done by multiplying the c_u term in (B.4.q NA) by 1,05, using load inclination factor $i_{cB}^e = 1$ and s_{cB} according to B.4.r NA, with the q -term neglected:

$$\frac{R_{Nu}}{A'} = 1,05 c_u s_{cB} i_{cB}^e \quad (\text{B.4.å NA})$$

NOTE 1 In the above it is presumed that T is acting in the same direction as the eccentricity (e_B) and parallel to the shortest side of the foundation, B'

NOTE 2 The procedure above is from Steen, O.: "Bæreevne af excentrisk belastede fundamenter". Danish Geotechnical Institute. IM 1969-1.

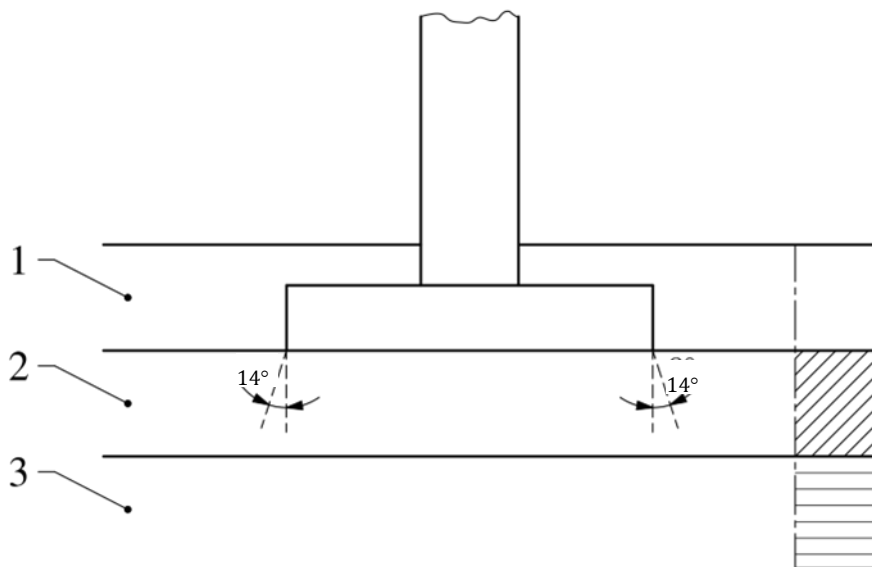
B.5 NA Calculation model for bearing resistance on ground underlain by a weaker layer (normative)

(1) In case of a firm layer underlain by a weaker layer, the bearing resistance analysis shall be supplemented by an analysis of failure due to punching shear (punch through).

(2) The bearing resistance R_{Nu} against punching shear of a strip, rectangular or circular spread foundation may be determined conservatively using Formulae (5.1) after projecting the effective foundation area to the top of the weaker fine soil layer as illustrated in Figure B.5.

NOTE 1 The maximum angle of foundation spread in the stronger coarse soil layer from the foundation edges is 1:4 (14°) (see Figure B.5).

NOTE 2 Formulae (B.4.a NA) and (B.4.q NA) are used to check for punching by projecting the effective area on top of the weak layer and by assuming that the weak layer extends indefinitely in depth. This is a conservative approach.



Key

- 1 overburden
- 2 stronger coarse soil layer
- 3 weaker fine soil layer

Figure B.5 NA — Foundation on a stronger layer over a weaker layer

NOTE 3 The overburden shall be taken as the overburden at the true foundation level as shown in Figure B.5.

Annex C NA Piled foundations

C.1 NA Use of this annex (normative)

(1) This Annex provides complementary provisions to the general rules in Clause 6 Piled foundations.

(2) The provisions given in the informative annexes C.7, C.8, C.10, C.11, C.12 and C.14 are not in accordance with established Danish geotechnical practice. Accordingly, these annexes may only be used if it can be demonstrated - based on reliable documentation or relevant experience - that their use yield results for Danish conditions that are on the safe side for the limit states considered.

C.2 NA Scope and field of application (informative)

(1) This Informative Annex provides an example of pile classification based on Table 6.1 and calculation models for:

axial pile resistance based on

- ground parameters;
- impact driving resistance;
- CPT profiles;
- MPM profiles;
- empirical tables;

downdrag due to vertical ground movements;

pile groups subject to axial tension;

single pile settlement and lateral displacement using load transfer functions;

buckling and second order effect;

axial pile resistance under cyclic loading

C.3 NA Pile classification (informative)

(1) Table C.1 NA gives examples of piles classified according to Table 6.1.

Table C.1 NA — Examples of piles in different classes

Pile type	Class	Example
Displacement piles	Full displacement	Driven cast-in-situ concrete piles Driven 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 steel H-section piles (plugged) Driven timber piles Augered cast-in-situ concrete piles
	Partial displacement ^a	Driven open-ended tubular steel piles (unplugged) Driven steel H-section piles (unplugged) Driven steel sheet piles Threaded cast-in-situ concrete piles ^c Precast threaded concrete screw piles ^b Steel threaded screw piles ^b Steel helix screw piles ^b
Replacement piles	Replacement	Bored cast-in-situ concrete piles installed using continuous flight auger (CFA) Cased continuous flight auger piles (CCFA) Bored piles with permanent casing Bored piles with temporary casing Bored piles with slurry or polymer support Bored piles excavated without support Bored or drilled steel tubular piles Micropiles Barrettes Diaphragm walls
a	A partial displacement pile is a pile which displaces less than the contour of the pile. Unplugged H-profiles and screw piles are examples of partial displacement piles.	
b	A screw pile is a prefabricated pile, fully or partly threaded, or with one or more discrete helix plates, which is installed by screwing the pile into the bearing strata with a penetration per revolution corresponding to the pitch.	
c	A threaded pile is not a screw pile, unless it is screwed into the ground.	

C.4 NA Pile resistance based on ground parameters (normative)

C.4.1 Impact driven piles

C.4.1.1 Shaft resistance

(1) For total stress analysis, the value of unit shaft resistance, q_s in fine (cohesive) soils may be derived from:

$$q_s = \alpha c_u \quad (\text{C.4.a NA})$$

where c_u is the undrained shear strength of the ground in the geotechnical unit being considered;

α is an empirical adhesion factor.

(2) The adhesion factor, α is an empirical factor that depends on the set-up effects of soil strength after installation, the interface friction between the pile and the ground, and the method of execution.

(3) For driven displacement piles in fine (cohesive) soils, the adhesion factor may be determined by:

$$\alpha = m r \quad (\text{C.4.b NA})$$

(4) The material factor, m , is:

$$m = \begin{cases} 1,0 & \text{for concrete} \\ 1,0 & \text{for timber} \\ 0,7 & \text{for steel} \end{cases}$$

(5) The regeneration factor, r , depends on the strength of the fine soil to the effect that r decreases with increasing strength. Where a direct determination is not carried out, the regeneration factor for fine soil may be taken as $r = 0,4$, valid for $c_u \leq 500 \text{ kN/m}^2$.

NOTE: Normally, the regeneration factor $r = 0,4$ can be expected 4 weeks after installation.

(6) In case of effective stress analyses, the value of unit shaft resistance, q_s in coarse (frictional) soils may be derived from:

$$q_s = \beta \sigma'_v \quad (\text{C.4.c NA})$$

where σ'_v is the vertical effective stress at the centre of the geotechnical unit being considered;

β is an empirical coefficient.

(7) The empirical coefficient, β , depends on the earth pressure coefficient and the interface friction between the pile and the ground. Where no direct determination is carried out, the following values of β apply for coarse soil:

	0,6	for full displacement piles in compression (piles of concrete, piles of timber, closed-ended tubular steel piles and steel H-section piles with soil plugs)
$\beta =$	0,3	for partial displacement piles in compression (steel sheet piles, open-ended tubular steel piles and steel H-section piles without soil plugs)
	0,2	for displacement piles in tension

(8) If piles are subjected to downdrag in a serviceability limit state, resistance contributions from the settling layers shall only be taken into account in the ultimate limit state to the extent that it can be justified that the shaft resistance in these layers is mobilised at failure.

C.4.1.2 Base resistance

(1) For total stress analysis, the value of unit base resistance q_b in fine (cohesive) soils may be derived from:

$$q_b = N_c c_{ub} \quad (\text{C.4.d NA})$$

where c_{ub} is the undrained shear strength of the ground at the pile base

N_c is a bearing resistance factor. $N_c = 9$

(2) For impact driven displacement piles with the base (pile toe) in hard clay till, defined by undrained shear strength, $c_u > 300 \text{ kN/m}^2$, $N_c = 18$ can be used. For clay till with $150 \text{ kN/m}^2 \leq c_u \leq 300 \text{ kN/m}^2$ linear interpolation between $N_c = 9$ and $N_c = 18$ can be used.

(3) For effective stress analysis, a preliminary rough estimate of unit base resistance q_b in coarse (frictional) soils for driven, full displacement piles may be estimated from

$$q_b = 2 N_q \sigma'_{vb} \quad (\text{C.4.e NA})$$

where σ'_{vb} is the vertical effective stress at pile base

N_q is the bearing resistance factor from Appendix B.4(4)

NOTE For open-ended tubular steel piles and steel H-section piles further recommendations for base resistance in coarse soil may be found in the literature.

(4) For effective stress analysis, the value of the unit base resistance q_b , should be limited to 10 MPa.

(5) When determining the pile base resistance, the strength of the soil layers both above and below the pile base level shall be considered.

(6) For driven, open-ended tubular steel piles or steel H-section piles, the bearing capacity shall be verified for both plugged and unplugged conditions, and the lower value shall be adopted.

(7) For impact driven piles with the base (pile toe) in coarse soil (sand and gravel), the geostatic calculation of the base resistance is considered too unreliable to be used for the final determination of the compressive resistance. In such cases, the "Danish pile driving formula" (see C.5.1 NA) may be applied.

C.4.1.3 Testing

(1) Pile driving should normally be initiated by test driving of approximately 5-10 % of the number of piles (but not less than 2-3 piles), recording the entire driving sequence. Test piles should be placed near the geotechnical investigation points. For the remaining piles, it is usually sufficient to record the driving sequence on the final part of the driving - typically over the last 1-2 m. When a follower is used, the driving sequence shall be recorded both before and after the follower has been mounted.

C.4.1.4 Execution considerations

(1) For all driven piles, the driving shall be recorded in a driving record containing all the relevant information about the driving (number, type, and dimension of the pile, pile rig type, hammer type, hammer weight and drop height, use of follower, ground level, toe level at the end of driving as well as notes on special conditions such as driving breaks, suspicion of a broken pile, pile head crushing etc.).

(2) Bearing resistance contributions shall be determined taking into account the effect of any pre-drilling for driven piles, bitumen cover as well as heave of adjacent piles during installation of driven piles.

(3) For pile groups, where pile heave may occur, it shall be verified by levelling of the pile top, whether heave is present. If the heave exceeds a given project-specific limit value, the piles shall be redriven, or the base resistance shall be disregarded.

NOTE: The project-specific limit value for pile heave will depend on the deformation tolerance of the supported structure.

(4) Piles in limestone and chalk require special measures when assessing the bearing resistance.

NOTE When piles are driven into limestone or chalk, the structure of these deposits may change significantly, and any potential strength recovery in the limestone/chalk may be uncertain. Consequently, the shaft resistance may be independent of previously measured in situ strengths. It may be necessary to use a combination of geostatic calculation, bearing resistance determined based on driving resistance, and/or dynamic load tests correlated with static load tests.

C.4.2 Screw piles

C.4.2.1 General

(1) For screw piles, geostatic calculations may be used for a preliminary rough estimate of the axial pile resistance.

(2) The unit resistance of screw piles, which are categorised as partial displacement piles, can in general be estimated from formulas in C.4.1 taking pile geometry and additions in C.4.2.2 and C.4.2.3 into account. Effect of pile geometry, including changes in shaft diameter, couplings, sleeves, or other local discontinuities, that may influence the pile resistance, shall be considered.

(3) Three principal types of screw piles are covered by this sub clause:

Piles with continuous threads along the full length of the shaft

Piles with continuous threads along a portion of the shaft, typically the lower section)

Piles with one or more discrete helix plates.

NOTE Combinations of the above types may occur, where a screw pile includes both continuous threads and discrete helix plates.

(4) Resistance of screw piles with multiple helix plates should be checked with two separate calculations to determine the lowest pile resistance:

Resistance from the individual plates, each giving an end-bearing contribution

Where a screw pile is fitted with multiple, relatively closely spaced helix plates, the bearing capacity may be governed by the shaft resistance on the circumscribed cylinder, as well as by the bearing resistance of the lowest or uppermost helix plate under compression or tension loading, respectively.

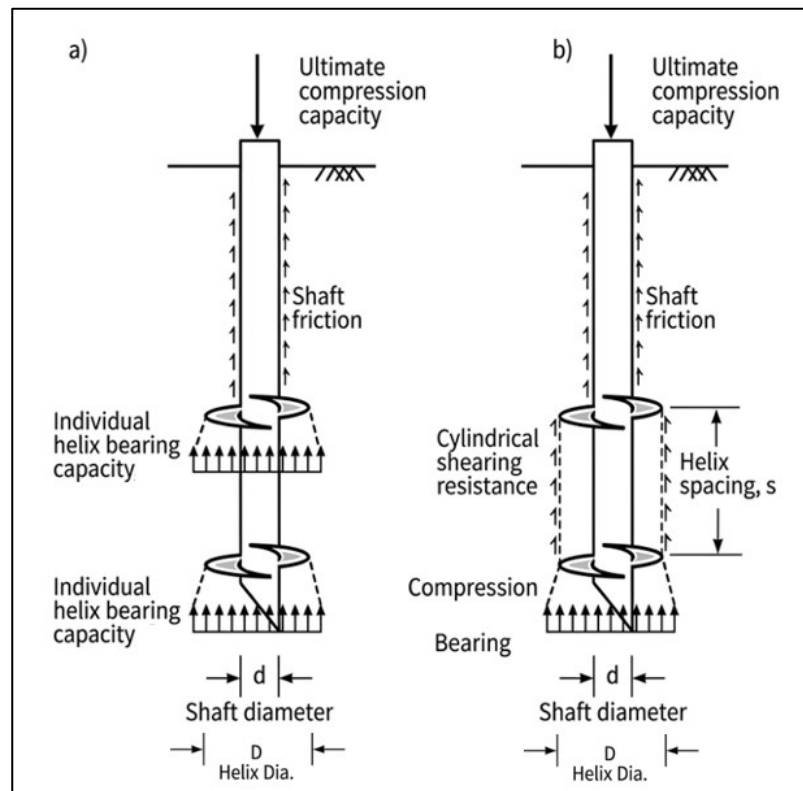


Figure C.4.a NA — Screw pile with helix plates in compression.

(5) For screw piles the verification of the structural resistance shall include verification of the threads and/or helix plates according to the relevant structural codes taking design service life into account.

C.4.2.2 Shaft resistance

(1) In case of effective stress analyses, the value of unit shaft resistance, q_s in coarse (frictional) soils may be derived from formula (C.4.c NA). The empirical β -coefficient depends on the degree of soil displacement along the pile.

(2) A displacement factor, k , is defined as the ratio between the diameter of the shaft, d and the diameter of the threads or helix plates, D :

$$k = d/D \quad (\text{C.4.f NA})$$

NOTE For the shaft friction above the threads or helix plates $k = 1,0$.

(3) For screw piles the empirical coefficient, β is:

$$b = \begin{cases} 0,6 & \text{for compression piles, where } k \geq 0,7 \\ 0,3 & \text{for compression piles, where } k < 0,7 \\ 0,2 & \text{for tension piles (regardless of } k) \end{cases}$$

(4) The shaft resistance of the tapered (conical) part of tension piles shall be disregarded.

C.4.2.3 Base resistance

(1) A preliminary rough estimate of unit base resistance in coarse (frictional) soils may be estimated as:

$$q_b = 1,5 N_q \sigma'_{vb} \quad (\text{C.4.g NA})$$

where σ'_{vb} is the vertical effective stress at pile base

N_q is the bearing capacity factor from Annex B.4(4)

(2) For both total and effective stress analysis, the value of the unit base resistance q_b , should be limited to 2 MPa.

(3) When determining the base resistance of the helix plates, the strength of the soil layers both above and below the pile base level shall be considered.

C.4.2.4 Testing

(1) For projects involving screw piles, a number of test piles shall be installed and installation records used for evaluating the ground model. With reference to the geotechnical soil investigation report and preliminary geostatic calculations, the test piles shall be evenly distributed across the site while ensuring that the expected weakest soil conditions are adequately captured.

(2) Initially, 10 % of the total number of working piles per population should be installed as test piles.

NOTE A population is a number of same type of piles, installed in similar ground conditions with similar installation parameters (Advancement Ratio and rotation speed).

(3) The final axial pile resistance of the screw piles shall be determined based on static pile load test. These piles shall be selected among the test piles installed with the smallest torque and crowd force.

(4) For compression piles 2 % of each population (minimum two piles) shall be tested.

(5) For tension piles 4 % of each population (minimum four piles) shall be tested.

C.4.2.5 Execution considerations

(1) The resistance of screw piles is highly dependent on execution, which is expected to be specified in the next revision of EN 12699 Displacement piles. Below mentioned requirements and recommendations may be due to updating after publishing of the final new version of EN 12699.

(2) Continuous installation records shall be obtained for all piles.

(3) Installation records shall as a minimum include:

Pile data (pile no., -type and -length, pitch and toe level)

Torque

Penetration Per Rotation (PPR)

Advancement Ratio (AR)

Rotation speed, Rounds Per Minute (RPM)

Axial installation force (crowd force)

(4) Measurement equipment shall be calibrated and reliable.

(5) For screw piles it is of vital importance that the penetration per rotation (PPR) matches the pitch of the thread from (below) the boundary of the ground layer which is supposed to contribute to the bearing resistance of the screw pile. This corresponds to an Advancement Ratio (AR) of 1,0. If the pile is “over screwed” (like an auger), the AR becomes < 1,0. However, even if $AR < 1,0$ the pile may still be considered a screw pile and have some bearing resistance.

NOTE $AR = PPR / \text{pitch}$, where pitch is the distance between the threads on a continuous helical pile.

(6) Screw piles should be installed with an AR between 0,85 and 1,15 at the depth where the threads or helix plates engage the load-bearing strata.

(7) If the installation torque is meant to be used as a quantitative measure of the bearing resistance, the piles must be installed with similar rotational speed and advancement ratio in the same type of soil. Furthermore, the torque should be measured using a direct measurement system.

(8) To determine the friction correlation factor K_c between installation torque and compressive bearing capacity, minimum three static load test on piles from same population but with different final installation torques must be carried out.

(9) For qualitative use (e.g. identifying boundary layers, increase or decrease of soil strength, high resistance from low resistance) the torque can be measured using an indirect measurement system like a hydraulic pressure. However, this transformation should be calibrated at relevant rotation speeds and oil temperatures.

(10) When penetrating the threads or helix plates into the bearing stratum, the rotational speed should be less than 20 rounds per minute (RPM).

C.4.3 Bored piles

C.4.3.1 Shaft resistance

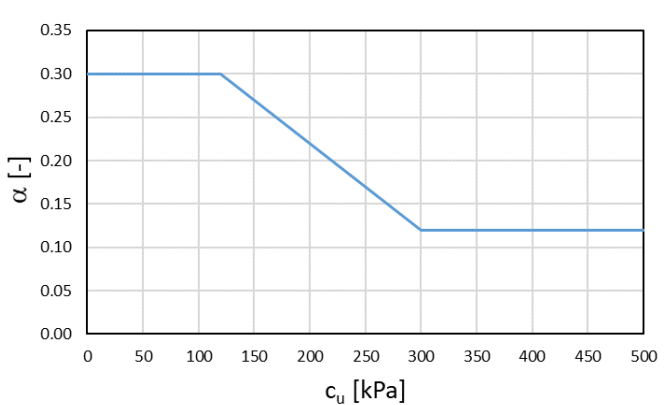
(1) For total stress analysis, the value of unit shaft resistance, q_s in fine (cohesive) soils may be derived from:

$$q_s = \alpha c_u \quad (\text{C.4.h NA})$$

where c_u is the undrained shear strength of the ground in the geotechnical unit being considered

α is an adhesion factor, see Table C.2 NA

Table C.2 NA — α for bored piles

Undrained shear strength c_u [kPa]	α	
< 120	0,3	
120 – 300	0,3 – 0,12	
300 – 500	0,12	

NOTE The value of c_u must be limited to 500 kPa. Between $120 < c_u < 300$ linear interpolations of α can be used.

(2) In case of effective stress analysis, the value of unit shaft resistance q_s in coarse (frictional) soils may be de-rived from:

$$q_s = \beta \sigma'_v \quad (\text{C.4.i NA})$$

where σ'_v is the vertical effective stress at the centre of the geotechnical unit being considered;

β is an empirical coefficient that depends on earth pressure coefficient and interface friction between the pile and soil. In the absence of further documentation, the following values of β may be applied:

0,3 for compression piles

0,2 for tension piles

(3) For uncased excavations stabilized with fluids (slurry) during construction, the unit shaft resistance values provided in C.4.3.1 shall be reduced by a factor of 0,7.

C.4.3.2 Base resistance

(1) For total stress analysis, the value of unit base resistance q_b in fine (cohesive) soils may be derived following provisions for base resistance given in Annex C.4.1, Formula (C.4.d NA)

(2) For effective stress analysis, the value of unit base resistance q_b in coarse (frictional) soils may be derived from

$$q_b = 1,2 N_q \sigma'_{vb} \quad (\text{C.4.j NA})$$

where σ'_{vb} is the vertical effective stress at pile base

N_q is the bearing capacity factor from Annex B.4(4)

(3) For both total and effective stress analysis, the value of the unit base resistance q_b , shall be limited to 2 MPa.

C.4.3.3 Execution considerations

(1) The axial resistance of bored piles is highly dependent on execution, and the provisions given in C.4.3.1 and C.4.3.2 assume that the work is carried out in accordance with DS/EN 1536.

(2) Drilling records shall be obtained for all bored piles.

(3) Drilling records shall as a minimum include:

- Pile data (pile no., -diameter and -length)
- Top and toe level of pile, and terrain level at pile
- Drilling equipment (drilling rig, casing, stabilizing fluids etc.)
- Registration of the drilled soil per 1 m of borehole as well as depth for layer boundaries and ground-water level
- Documentation of casting of concrete (concrete mix, casting method, concrete volume etc.)

(4) The unit shaft resistance of bored piles is highly dependent on the method of stabilization of the borehole (casing, support fluid, etc.).

(5) The unit base resistance of bored piles is highly dependent on a thorough cleaning of the bottom of the borehole before casting concrete, to ensure full contact between intact soil and the pile and to minimize settlements when loading piles. The settlements can be reduced by base grouting or installation of flat jacks at the pile base.

C.5 NA Pile resistance based on pile driving formulae (normative)

C.5.1 The Danish Pile Driving Formula

(1) For piles driven into coarse (frictional) soil, the compressive resistance, $R_{c,rep}$, can be determined using the "Danish Pile Driving Formula":

$$R_{c,rep} = R_{dyn,m} \quad (C.5.a \text{ NA})$$

$$R_{dyn,m} = \frac{\eta h G}{s + 0,5 s_0} \quad (C.5.b \text{ NA})$$

$$s_0 = \sqrt{\frac{2\eta h G L_p}{AE}} \quad (C.5.c \text{ NA})$$

$$\eta = \eta_0 (1 - \mu \cdot \tan \theta) \quad (C.5.d \text{ NA})$$

where

η	is the efficiency factor
η_0	is the efficiency factor for vertical leader
μ	is the friction coefficient between hammer and leader
	($\mu \sim 0,1 - 0,4$ depending on winch, leader, hammer etc.)

θ	is the inclination of the leader
G	is the weight of the drop hammer
h	is the vertical component of the drop height
s	is the permanent set of pile per blow
L_n	is the pile length
A	is the cross-sectional area of the pile
E	is the modulus of elasticity of the pile.

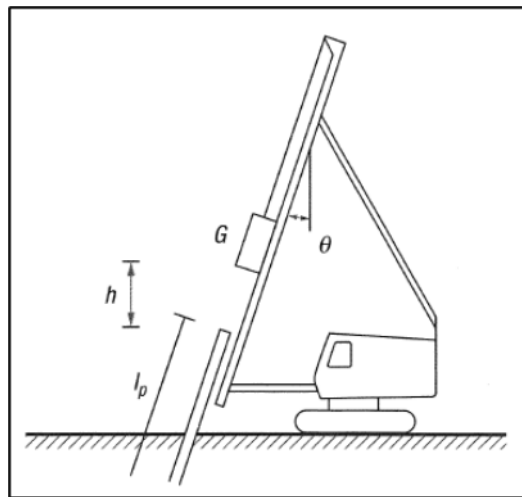


Figure C.5.a NA — Illustration of drop height and inclination of leader

(2) The formula assumes the use of the following values of the moduli of elasticity:

Concrete piles $E = 20 \cdot 10^6 \quad \text{kN/m}^2$

Timber piles $E = 10 \cdot 10^6 \quad \text{kN/m}^2$

Steel piles $E = 210 \cdot 10^6 \quad \text{kN/m}^2$

(3) For piles shorter than 20 times the pile width, the mean value of the actual pile length and 20 times the pile width should be inserted in the pile driving formula. For timber piles, the mean diameter is used for calculating the area A . For steel piles, A is the cross-sectional area of the steel.

(4) Driving resistances are determined based on the driving result at the end of a normal, uninterrupted driving process, or from re-strike after a short driving break.

(5) If another formula than the Danish Pile Driving Formula is applied to determine the bearing resistance of compression piles, the validity of the formula is to be based either on recognised documentation or static load tests on the same pile type, of similar length and cross-section, and in similar soil conditions.

C.5.2 Execution consideration

(1) The pile shall be driven in such a way that damage is avoided.

(2) The risk of damaging the pile during the driving may be reduced significantly by not using a greater effective drop height ηh , or by using a smaller ratio of the hammer weight (G) to the weight of the pile (G_p) than the values given in table C.5.a NA.

Table C.5.a NA — Limiting criteria to prevent damage of pile

	Max ηh	Min G/G_p
Reinforced concrete piles	approx. 1 m	approx. 0,8
Steel piles	approx. 2 m	approx. 1,5
Timber piles	approx. 4 m	approx. 1,0

(3) By “hard” driving (corresponding to $s < 0,1 s_0$) the driving shall be adjusted to avoid damaging the piling material.

(4) “Soft” driving of concrete piles, corresponding to a set $s > 0,02$ m (less than 10 blows per 0,2 m pile settlement), shall be avoided, if necessary, by reducing the drop height. The requirement does not apply, if the pile penetrates partly or fully by its own weight and the weight of the hammer.

C.6 NA Axial pile resistance for impact driven piles based on CPT profiles (informative)

C.6.1 Base resistance

(1) A preliminary rough estimate of unit base resistance q_b in coarse (frictional) soils may be estimated following the principals given in Jardine, R. et al, 2005, *ICP design methods for driven piles in sands and clays*, from which the expressions below are reproduced.

(2) For driven closed ended, squared or rectangular piles q_b may be estimated from:

$$q_b = c_b q_{c,mean} \quad (\text{C.6.a NA})$$

where $q_{c,mean}$ is the mean (average) cone resistance at the pile base

c_b is an empirical cone factor for base resistance

NOTE 1 The empirical factor c_b is 0,7 for driven closed-ended, squared or rectangular piles with a pile width up to 500 mm.

NOTE 2 $q_{c,mean}$ is calculated as an arithmetic average in the zone from $1,5 B_{eq}$ above the pile tip to $1,5 B_{eq}$ below the pile tip. Cone resistances are limited to values between 0,7 to 1,3 times the average q_c . The average of the remaining values is then used as $q_{c,mean}$

NOTE 3 $q_{c,mean}$ is according to Bustamante and Gianeselli, 1982, *Pile bearing capacity prediction by means of static penetrometer CPT*. Proceedings of the 2nd European symposium on penetration testing, pp. 493-500.

(3) For driven closed-ended, circular piles q_b may be estimated from:

$$q_b = q_{c,mean} [1 - 0,5 \log(D/D_{CPT})] \quad (\text{C.6.b NA})$$

where $q_{c,mean}$ is the mean (average) cone resistance at the pile base
 D is pile diameter
 D_{CPT} is 0,036m (Diameter of CPT probe)

NOTE A lower limit of $q_b = 0.3q_{c,mean}$ is suggested for piles with $D > 0,90$ m.

(4) For effective stress analysis, the value of the unit base resistance q_b , should be limited to 15 MPa.

(5) For open-ended tubular steel piles and steel H-section piles further recommendations for base resistance in coarse soil can be found in Jardine, R et al. (2005).

(6) For piles installed from an excavated depth that is deeper than that from which the cone penetration tests were executed, the value of $q_{c,mean}$ in Formulae (C.6.a NA) should be reduced accordingly.

(7) For impact driven piles with the base (pile toe) in coarse soil (sand and gravel), the geostatic calculation of the base resistance from CPT (C.6.a NA and C.6.b NA) is considered too unreliable to be used for the final determination of the compressive resistance. In such cases, the "Danish pile driving formula" (see C.5.1 NA) may be applied.

C.9 NA Downdrag due to vertical ground movements (normative)

C.9.1 NA General

(1) The drag force caused by downdrag shall be classified as a permanent action.

NOTE 1 'Downdrag' is the term used to describe relative movement between settling ground and the pile shaft. A drag force occurs where the ground settlement exceeds the pile settlement.

NOTE 2 Pile settlement due to downdrag continues until the combination of imposed actions from the structure and the drag force come into equilibrium with the mobilised pile resistance. The neutral point marks the boundary between forces that act downwards and upwards along the pile shaft.

(2) Potential downdrag shall be included in the verification of serviceability limit states.

NOTE Section C.9.3 provides a prescriptive approach for assessing the impact of downdrag on piles, which under the specified conditions can serve as an alternative to an explicit verification of the serviceability limit state.

(3) Potential downdrag shall be included in the verification of ultimate limit states.

NOTE 1 Typically, pile settlement at the ultimate limit state (ground failure) exceeds the settlement needed to fully mobilize drag forces (negative skin friction). Therefore, drag forces normally can be disregarded in the verification of the ultimate limit state.

NOTE 2 If drag forces are disregarded in the ultimate limit state, see NOTE 1, positive pile shaft resistance from all layers can be utilized in ultimate limit state, provided compatibility of displacements is documented, see DS/EN 1997-3, clause 6.5.3.1(4) NOTE.

(4) Potential drag forces shall be included in the structural design of piles.

C.9.3 NA Simplified approach for analysis of piles subjected to downdrag

(1) Representative values of the drag force F_{neg} should be determined for the full thickness of the settling strata, i.e. for the neutral point located at the base of the settling strata.

(2) If (1) is fulfilled the following analysis for piles subjected to downdrag in GCC2 as well as for end bearing piles transmitting the load to the ground, which can be characterized as competent in consideration of pile load and possible load from fills, an equivalent failure calculation can replace an explicit settlement calculation, when verifying the serviceability limit state:

$$F_{cd,ULS} + \gamma_{F,drag} F_{neg} \leq \frac{R_{b,rep} + R_{s,rep}}{\sqrt{\gamma_{Rc} \cdot \gamma_{Rd,pile}}} \quad (\text{C.9.a NA})$$

where	$F_{cd,ULS}$	is the design axial compression load in the ultimate limit state with the square root of partial factors for the relevant ULS load combination without contributions from downdrag.
	F_{neg}	is the representative drag force on the pile from for the full thickness of the settling soil.
	$F_{b,rep}, F_{s,rep}$	is part of the calculated bearing resistance of the pile, originating from the competent strata below the settling soil layer. For impact driven end bearing piles in coarse soil, $R_b + R_s$ shall be replaced by R_{dyn} minus any shaft resistance from the settling strata during driving.
	$\gamma_{Rd,pile}$	is the model factor accounting for uncertainty in the resistance model for a piled foundation, see Table 6.4 NA or 6.5 NA.
	γ_{Rc}	is the resistance factor, see Table 6.9 NA.
	$\gamma_{F,drag}$	is the partial factor on the drag force, see Table 6.9 NA.

NOTE Late glacial and older deposits are usually considered to be competent in this context. There are, however, several important exceptions – e.g. late glacial soft soil deposits, inter-glacial marine and marsh deposits as well as high and very high plasticity clays and chalk.

(3) To determine the representative drag force, F_{neg} , the upper (superior) values of soil strength values should be used.

NOTE The maximum drag force can be determined as the smaller of the two following values:

The geostatically calculated skin friction in the deposits above the competent strata according to C.4, using regeneration factor $r = 1,0$, see C.4.1.1 (5) and formula (C.4.b NA). For bored piles $\alpha = 1,0$

The settlement-generating actions within the volume, which is limited downwards by surfaces inclined 1H:2V intersecting the pile cross-section at the base of the settling strata. Only the part of the settlement-generating action which corresponds to the settlements that will occur in the settling strata after driving of the pile shall be taken into consideration.

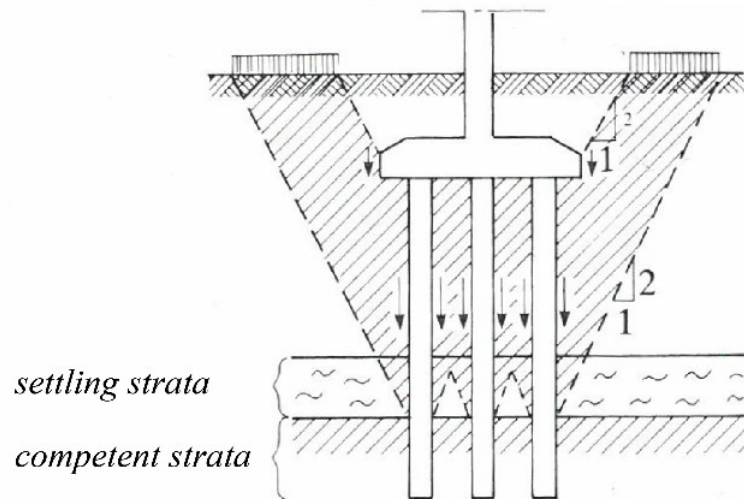


Figure C.9.a NA — Volume to be taken into consideration.

(4) By experience, downdrag may be reduced by coating prefabricated piles in and above the settling strata with bitumen. For concrete piles cast in smooth formwork and coated with 1 mm bitumen with penetration 70-100 under controlled (dry) conditions, the geostatically calculated downdrag (see above) in a serviceability limit state is assumed to be reduced to less than 10 kN per m² concrete surface. Not less than 25 % of the full value should be taken into account without closer investigation due to the risk of damaging the bitumen layer during installation.

NOTE The effect of the bitumen depends on how fast the shear stresses along the bitumen layer build up. The specified guidelines for its effect are only applicable when the stresses build up slowly in connection with the settling (or heave) of the surrounding soil, and not for shear stresses from dynamic actions (including dynamic impact pile tests).

(5) Specification of bitumen shall be according to DS/EN 12591.

C.15 NA Static load tests (informative)

C.15.1 Constant Rate of Penetration test (CRP-test)

(1) As an alternative to a maintained pile load test procedure described in DS/EN ISO 22477-1 for compression piles, a faster test method using Constant Rate of Penetration (CRP) in the last part of the test may be used to determine the geotechnical compression resistance of a pile at the ultimate limit state, $R_{c,m}$ according to the loading procedure stated below:

Define a target load, F_{p1} , for the first load cycle as either 50 % of the estimated ultimate resistance or a relevant service load.

- a) Apply a datum load of maximum 5 % of F_{p1} .
- b) Apply load at a constant loading rate of 4 % of the target load F_{p1} per minute until reaching the target load.
- c) At the target load F_{p1} , maintain the load constant for a minimum of 30 minutes. A deformation rate of $< 0,2$ mm/hour at the end of this load step is desired. Record deformation every minute.
- d) Unload to the datum load over a period of 5 minutes. Record deformation every minute
- e) Maintain the datum load for 10 minutes. Record deformation every minute.
- f) Reload to the load level specified in c) over a period of 5 minutes. Record deformation every minute.
- g) Continue with the constant loading rate described in b). Record deformation every minute.
When the deformation rate reaches 20 mm/hour, continue the load step at this constant deformation rate until the failure criterion is met.
The failure criterion is a permanent settlement after unloading = max (40 mm; 0,1 D) if no peak load has been identified before that, where D is the pile diameter or equivalent diameter for a squared pile, $D_{\text{equ}} = a\sqrt{4/\pi}$, where a is the side length. For a screw pile D is the diameter of the thread or helix plate.
The test may be terminated if the load drops significantly under constant deformation rate or if the maximum capacity of the test setup is reached.
- h) Unload to the datum load over 5 minutes. Record deformation every minute for 30 minutes.
- i) Throughout the entire static load test, regular displacement measurements of the reaction device shall be performed.

(2) The procedure described above may also be used as an alternative to pile load test procedure described in DS/EN ISO 22477-2 for tension piles.

Annex D NA Retaining structure (normative)

D.1 NA Use of this annex (normative)

(1) This annex provides complementary provisions to the general rules in Clause 7, retaining structures.

(2) The provisions given in the informative annexes D.3, D.6, D.7, D.9, D.10.1, D.11, D.12 and D.13 in DS/EN 1997-3 are not in accordance with established Danish geotechnical practice. Accordingly, these annexes may only be applied if it can be demonstrated - based on reliable documentation or relevant experience - that their use yield results for Danish conditions that are on the safe side for the limit states considered.

D.2 Scope and field of application (informative)

(1) This informative annex covers:

limit values of earth pressures;

at rest values of earth pressures;

compaction effects;

additional earth pressures induced by thermal effects for integral bridges;

general principles and application of calculation models; limit equilibrium models, beam on springs, numerical models;

movements to develop active and passive earth pressures;

numerical continuum models;

vertical equilibrium of embedded walls

basal heave

interaction between anchors and retaining structures.

D.2.A NA General (normative)

(1) Earth pressure on structures may be determined according to the theory of plasticity, provided that the displacements are sufficient to develop the limit state earth pressures.

NOTE Important examples of structural types, where the displacement during failure is not necessarily sufficient to mobilise limiting values for earth pressure, are:

Walls with internal struts (if modelled as yielding);

Walls with large axial forces which may fail by instability or buckling;

Brittle / unreinforced walls;

Walls with heavily prestressed ground anchors (if modelled as yielding), particularly when there are multiple anchor levels;

(2) If a wall or its support structures (i.e. compression struts) will fail in a brittle manner - i.e. without prior deformation and without any bearing capacity reserve - the at-rest earth pressure shall be applied in the ultimate limit state, ULS, including any additional pressure from compaction.

(3) In cases as described in D.2.A(1) and (2) at-rest values of earth pressure – inclusive potential compaction pressures - shall be used and the effect of actions factored by $\gamma_E = 1,2$ k_F in VC4. Partial factor on soil strength is $\gamma_M = 1,0$.

NOTE The background for this requirement is that, if the wall turns out to be stronger and stiffer than assumed in the design of the wall itself — so strong that it is in fact capable of resisting the at-rest earth pressure and the com-paction pressure without significant deflection — then a force corresponding to the at-rest and compaction pressure will be transferred to the supports. The supports must therefore be designed to resist this force.

(4) The partial factors of DS/EN 1997-1 DK NA, Table 4.8 NA provide the required Danish safety for structures subject to earth pressure, when the associated analysis is based on the following:

At the toe of the wall - where the failure surfaces from the front and back meet - a discontinuity of the failure line can occur. This corresponds to the fact that the two failure surfaces do not necessarily have coinciding rotation centres.

The failure surface can consist of a line failure, zone failure, or a combination hereof.

(5) The roughness between the structure and the surrounding soil (the angle of shearing resistance of the wall δ and adhesion a) may normally be taken as equal to the corresponding effective strength parameters of the ground (φ' and c') unless the surface is treated with bitumen or similar products, see also DS/EN 1997-3, 7.5.2. It shall, however, be verified that the degree of roughness used in the earth pressure calculation is in accordance with the vertical equilibrium condition of the structure.

(6) When investigating the ultimate limit states, the most unfavourable, realistic combination of water tables - including differential water pressure - which may occur, shall be used for the structure.

NOTE 1 In this connection, attention is drawn to the fact that it is the water pressures on the failure surface which are governing.

NOTE 2 Holes in the structure are usually not sufficient to prevent the build-up of a water pressure on the failure surface behind the structure.

NOTE 3 It should be considered whether an impermeable wall may change the water table conditions, e.g. by cutting off the natural runoff, whereby a high water table behind the wall may build up.

NOTE 4 Water pressure in fine (cohesive) soil cannot be fully documented from water table observations in stand-pipes. For structures retaining fine soil, a water table at the top of the fine soil should normally be assumed, see DS/EN 1997-1 DK NA, NCCI for 6.4 (2), (6) and (7).

D.4 NA Calculation model to determine at-rest values of earth pressure (informative)

(1) In addition to 7.5.4, the at-rest earth pressure coefficient K_0 in soils may be determined only for unloading stress paths from:

$$K_0 = (1 - \sin\varphi)\sqrt{OCR} \times (1 + \sin\beta) \leq K_{py} \quad (\text{D.1 NA})$$

where φ is the soil's angle of friction;
 OCR is the over-consolidation ratio at depth z_0 (equal to $\sigma'_{v,max} / \sigma'_v$);
 $\sigma'_{v,max}$ is the maximum effective overburden pressure at depth z_0 ;

- σ'_v is the current effective overburden pressure at depth z_0 ;
 β is the inclination of the ground surface above the horizontal; and
 K_{py} is the passive earth pressure coefficient.

NOTE Formula (D.18) can lead to unrealistic values of K_0 close to the ground surface, where the vertical stress is low.

(2) Formula (D.18) should not be used for very high values of OCR or in circumstances involving geological reloading.

(3) The direction of the resulting force should be assumed to be parallel to the ground surface.

D.5 NA Earth pressures due to compaction (normative)

(1) The effective compaction earth pressure normal to the wall face (p'_c) at a depth (z) below ground surface may be determined from:

$$p'_c = \begin{cases} K_{py,0} \bar{\gamma}_c z & \text{for } z \leq z_{c,min} \\ p'_{c,max} & \text{for } z_{c,min} \leq z \leq z_{c,max} \\ K_0 \bar{\gamma}_c z & \text{for } z \geq z_{c,max} \end{cases} \quad (D.2 \text{ NA})$$

$$z_{c,min} = \frac{p'_{c,max}}{\bar{\gamma}_c K_{py,0}} \quad (D.3 \text{ NA})$$

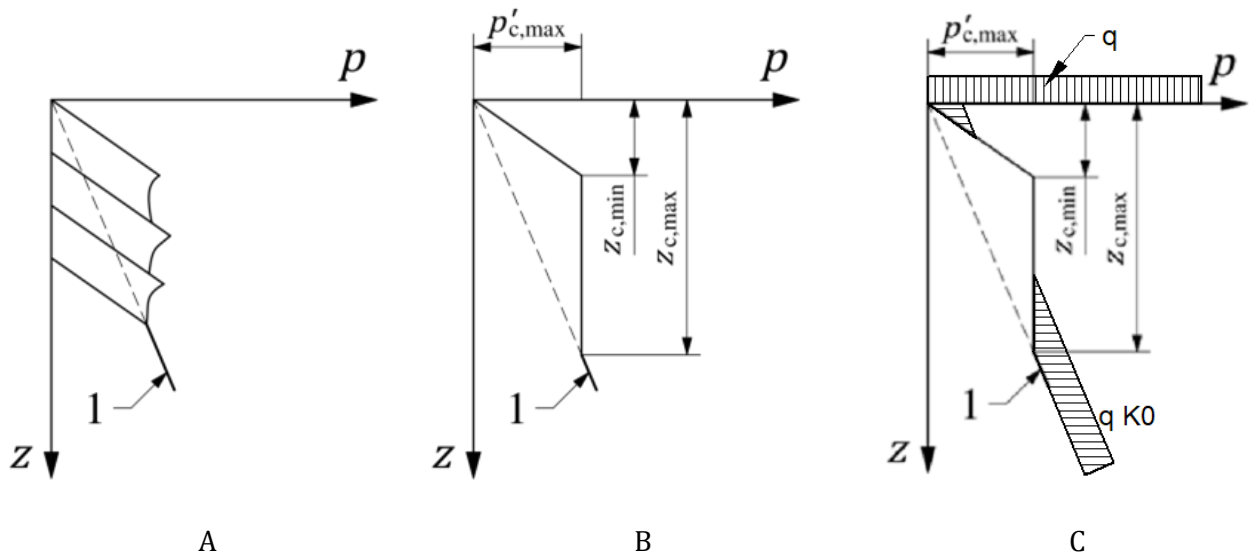
$$z_{c,max} = \frac{p'_{c,max}}{\bar{\gamma}_c K_0} \quad (D.4 \text{ NA})$$

- where $p'_{c,max}$ is the maximum horizontal earth pressure due to compaction;
 $\bar{\gamma}_c$ is the average weight density of the ground over depth $z_{c,max}$;
 $K_{py,0}$ 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; and
 $z_{c,max}$ is the maximum depth at which p'_c applies.

NOTE Measurements indicate that additional pressures depend on the applied compaction energy, the soil moisture content, the thickness of the compacted layers and the travel pattern of the compaction machinery. Horizontal pressure normal to the wall in a layer can be reduced when the next layer is placed and compacted. When backfilling is complete, the additional pressure normally acts only on the upper part of the wall.

(1) For non-yielding walls, compaction pressure may be represented by the bi-linear profile shown in Figure D.4(B).

NOTE Compaction pressures from soil placement in layers, more realistically produces a distribution similar to that shown in Figure D.4(A).



Key

- A compaction earth pressure
- B simplified profile for non-yielding wall with compaction pressure
- C effect of surface load in combination with additional pressure from compaction
- 1 K_0 line

Figure D.4 NA — Distribution of compaction earth pressure (A), simplified profile for non-yielding wall (B) and with succeeding surface load after compaction (C)

(2) The value of the maximum compaction earth pressure $p'_{c,max}$ may be taken from Table D.1 NA.

Table D.1 NA — 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)
	$b \leq 1,0$ m	$b \geq 2,5$ m	
Non-yielding	40	25	15
NOTE 1 Use interpolation for intermediate values of b			

D.8(2) Limit equilibrium models

(NPD)

The values of λ_a and λ_p are 0,1 to 0,5 % and 1 to 5 %, respectively.

D.10 NA Calculation model to determine intermediate values of earth pressure (informative)

D.10.1 General (informative)

No change

D.10.2 NA Movement to develop active and passive earth pressures

(1) The horizontal movement, v , required to mobilize active or passive earth pressure in the soil behind or in front of a retaining wall should be considered.

NOTE The magnitude of movement depends - among other factors - on the density of the soil.

(2) The order of magnitude values of the relative movement, v/h , are given in Table D.2:

Table D.2 NA — Order of magnitude values of the ratio v/h

	Loose soil v/h %	Dense soil v/h %
Active earth pressure	0,5	0,1
Passive earth pressure	5	1
NOTE 1	v is the horizontal wall movement required to mobilize limit earth pressure	
NOTE 2	h is the wall height on active and passive side.	
NOTE 3	The values given assume a vertical wall and a horizontal ground surface	

Annex E.5 NA Anchors (informative)

This informative Annex E.5 NA provides following changes to figure E.2 and E.3 in the informative annex E.5.

Figure E.2: Figure E.2 is replaced by Figure E.2 NA. Only change in Figure E.2 NA is a slight improvement of the layout of the key.

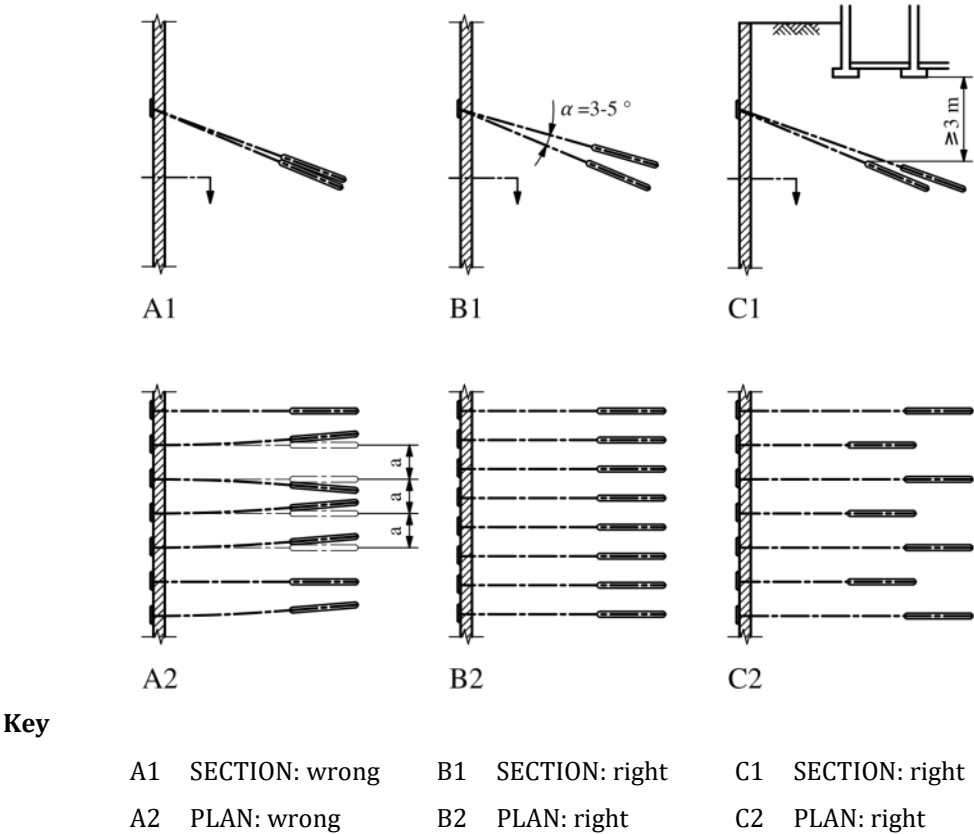
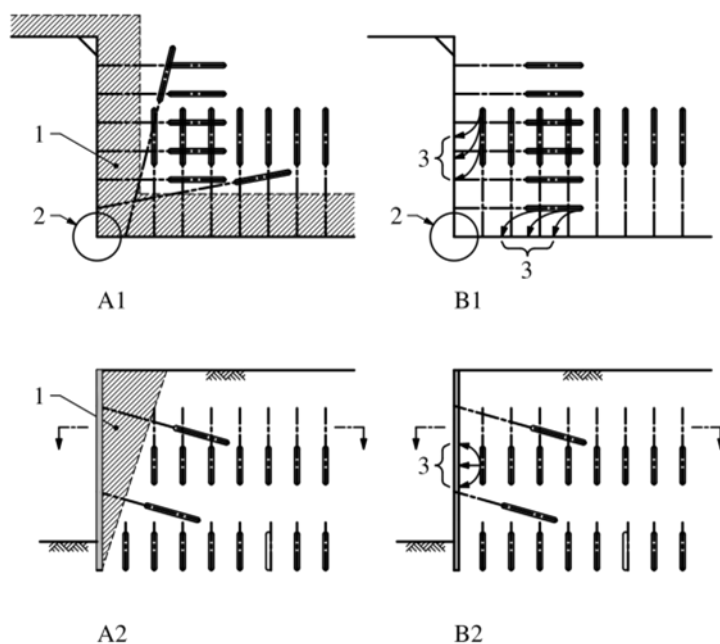


Figure E.1 NA — Examples of good and bad spreading and staggering of anchors

Figure E.3: Figure E.3 is replaced by Figure E.3 NA. Only change in Figure E.3 NA is a slight improvement of the layout of the key.



Key

A1	Plan: $E = E_a$	B1	Plan: $E > E_a$
A2	Section $E = E_a$	B2	Section: $E > E_a$
1	Active earth pressure wedge		
2	Corner designed to transfer tension		
3	Additional earth pressure		

Figure E.2 NA — Examples of anchoring a protruding wall corner

Annex F.4 NA Reinforced fill structures(informative)

This informative Annex F.4 NA provides following changes to the informative annex F.4.

F.4 Calculation models for reinforcement of embankments on weak soil (informative)

In the title of F.4 the term “basal” is deleted

Annex J.3 NA Ground water control

J.3 NA Examples of groundwater control measures (informative)

NOTE Table J.1 gives examples of groundwater control measures.

Table J.1 NA — Examples of groundwater control measures NA

Class	Measures	Examples
1	Impermeable barriers ^a	Sheet pile walls, diaphragm walls, slurry walls, grouted sealing slabs, plastic and geosynthetic barriers.
2	Reduction of hydraulic conductivity	Grouting, soil mixing, use of natural clay layer, build-in membrane of clay.
3	Dewatering/infiltration	Drains, well points, bored wells and other dewatering systems.
a	The term “impermeable” is project specific. A project defined low value of the hydraulic conductivity is considered as impermeable.	