TC250/SC7/EG7: Pile Design

(draft) Final Report 2015

1. WHICH CLAUSES IN THE CURRENT EN 1997-1 AND -2 ARE RELEVANT TO YOUR EG'S TOPIC OF INTEREST?

In the first instance Section 7 “Pile Foundations” of EN 1997-1 has been relevant for the work of EG 7 ´Pile Design´. Further some clauses of EN 1997-1, section 1, 2 and 3 have to be considered providing additional information for the design of deep foundations.

In EN 1997-2 also some appendices include calculation methods for pile foundations (e.g. D.7) and therefore are of interest for EG 7.

2. WHICH OF THOSE CLAUSES SHOULD REMAIN UNCHANGED IN THE NEXT EDITION OF EUROCODE 7?

In order to make the outcome of EG 7´s efforts as effective and valuable as possible EG 7 has been working on the present text of section 7 “Pile Foundations” since summer 2013 after a preparatory 2-year-period period of more general and fundamental discussions and considerations since 2011. The process of elaborating which clauses could be deleted, which clauses could remain unchanged and which clauses or even subsections should be added in the next edition of Eurocode 7 has been ongoing in a systematic manner. The section has been discussed with all delegates clause by clause whereas new subsections need more time to be elaborated and discussed.

The section 7 “Pile Foundations” of EN 1997-1 has been restructured by EG 7 and attached to the new structure which was elaborated during the SC7-Vienna meeting.

The elaborated status (2015-10-31) of the revised section ´Pile foundations´ elaborated and discussed in EG 7 is attached to this final report. It looks quite promising that the aim to come up with a more user-friendly version of the section “Pile Foundations” will be reached as the clauses already revised could be focused on the essential principals.

The work of EG 7 was focused on:

a) Editing, changing and improving text of EN 1997-1, section 7 (mainly editorial task, e.g. cancel doublings, resetting mistakes). Aim was to make the document much clearer (modifications are marked in the attached document).

b) Review of values for correlation factors, partial factors, model factors and action factors and their combination in each country as this was considered to be of major interest for further revision of EN 1997-1.

c) Comparing and identification of proposals for harmonizing different design approaches for pile design.

d) Comparing calculation methods for pile foundations for selcected applications like axially loaded piles, downdrag etc.

The new structure and the elaborated status of the revised version (draft) of section ´Pile foundations´ is attached in a ´two-column´-layout providing the original text with the recommended changes on the left side and the improved proposal on the right side (status of 2015-10-31). Due to time restrictions the process of discussing all clauses in detail couldn´t been finished completely, but main aspects were addressed and documented in the attached version.

3. WHICH OF THOSE CLAUSES SHOULD BE DELETED FROM THE NEXT EDITION OF EUROCODE 7? AND WHY?

Based on evaluation by EG 7 a proposal was elaborated for making the text of section 7 shorter, especially cancelling textbook-like passages and doublings. Please take reference to issue 2 and the attached document (draft revised section 7 prepared by EG 7).

4. WHICH OF THOSE CLAUSES SHOULD BE CHANGED IN THE NEXT EDITION OF EUROCODE 7? WHAT CHANGES SHOULD BE MADE? AND WHY?

Several modifications are proposed by EG 7. Please take reference to issue 2 and the attached document (draft revised section 7 prepared by EG 7) for more details.

5. WHAT NEW CLAUSES SHOULD BE ADDED ON YOUR TOPIC IN THE NEXT EDITION OF EUROCODE 7? AND WHY?

It was agreed that aspects which are presently not covered appropriate should be included in future version of EC 7. The most relevant aspects were identified as follows:

- downdrag / negative skin friction
- ground heave
- design of pile groups

- design of piled rafts including cases where piles are used as ´settlement-reducer´ (concerning ´hard inclusions´ relation to EG ´Ground Improvement´ necessary
- dynamic / cyclic loads on piles
- seismic design of pile foundations (strong relation to EC 8 necessary)
- transversally loaded piles, especially by horizontally moving ground
- serviceability of pile foundations

Concrete proposals for new claused were elaborated e.g. for the design of piles for negative skin friction (downdrag) and other issues like negative lateral thrust on piles due to soil movement. These clauses were integrated into the attached revised version of section 7.

ACTIVE MEMBERSHIP

Name	Position*	Country
Christian Moormann	Convenor	Germany
Chris Raison	Secretary	UK
Gary Axelsson		Sweden
Ioan Boldurean		Romania
Sébastien Burlon		France
Josif Josioivski		Rep. of Mazedonia
Boleslaw Klosinski		Poland
Jan Kos		Czech Republic
Ole Møller		Denmark
Panicos Papadopoulos		Cyprus
Krzystof Sahajda		Poland
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Frits van Tol		Netherlands
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Appendix

The draft status (2015-10-31) of the revised section of EN 1997-1, section 7 ´Pile foundations´ elaborated by EG 7 is attached to this final report documenting the modifications proposed and the new clauses elaborated by EG 7. These proposals and recommendations might be considered as a starting point by the Project Team which will be mandated with writing the section ´Pile Foundations´ for next generation of EN 1997.

Report prepared by:

Christian Moormann

31.10.2015

2015-10-31/CM

Eurocode 7 ´Geotechnical design´ - Part 1: General rules - Section 7 – Pile foundations

Proposal for modifications elaborated by TC250 / SC7 /EG7

Doc EG7-D160 Rev. 10

´Dirty´ Version with all modifications elaborated by EG 7	´Clean´ Version of clauses for which modifications were finally accepted and agreed on by EG 7
New Structure: **1 General** [existing §7.1] **2 Limit states [existing §7.2]** **3 Actions and design situations [existing §7.3]** **3.1 General [existing §7.3.1]** **3.2 Dynamic and cyclic loading** 3.3 Actions due to ground displacement [existing §7.3.2] 3.3.1 General 3.3.2 Downdrag (negative skin friction) 3.3.3 Heave 3.3.4 Transverse loading **4 Design methods and design considerations** [NEW, incorporating existing §7.6] 4.1 Design by calculation [NEW] 4.1.1 General [NEW] 4.1.2 Single piles [NEW] 4.1.2.1 Axially loaded piles [NEW, incorporating existing §7.6] 4.1.2.2 Transversely loaded piles [NEW, incorporating	

existing §7.7] 4.1.3 Pile Groups [NEW] 4.1.3.1 Axially loaded piles [NEW] 4.1.3.2 Transversely loaded piles [NEW] 4.1.4 Piled Rafts [NEW] 4.1.5 Pile resistance due to cyclic, dynamic and impact loads [NEW] 4.2 Design by testing [NEW, incorporating existing §7.6] 4.2.1 General [NEW] 4.2.2 Axially loaded piles [NEW, incorporating existing §7.6] 4.2.2.1 Ultimate resistance from static load tests 4.2.2.2 Ultimate resistance from dynamic impact tests 4.2.2.3 Ultimate resistance by applying pile driving formulae 4.2.2.4 Ultimate resistance from wave equation analysis 4.2.2.5 Re-driving 4.2.3 Transversely loaded piles 4.2.4 Pile resistance due to cyclic, dynamic and impact loads **5 Ultimate limit state design [NEW]** 5.1 General [NEW] 5.2 Single piles [NEW] 5.2.1 Axially loaded piles [NEW, incorporating existing §7.6] 5.2.2 Transversely loaded piles [NEW, incorporating existing §7.7] 5.3 Pile Groups [NEW] 5.3.1 Axially loaded piles [NEW]	

5.3.2 Transversely loaded piles [NEW] (5.4 Piled Rafts) [NEW] **6 Serviceability limit state design [NEW]** 6.1 General [NEW] 6.2 Single standing piles [NEW] 6.2.1 Axially loaded piles [NEW] 6.2.2 Transversely loaded piles [NEW] 6.3 Pile Groups [NEW] 6.3.1 Axially loaded piles [NEW] 6.3.2 Transversely loaded piles [NEW] (6.4 Piled Rafts) [NEW] **7 Testing and Instrumentation** 7.1 General [existing §7.5.1] 7.2 Static load tests [existing §7.5.2] 7.2.1 Loading procedure 7.2.2 Trial piles 7.2.3 Working piles 7.3 Dynamic load tests [existing §7.5.3] 7.4 Load test report [existing §7.5.4] **8 Structural design [existing §7.8]** **9 Execution (supervision, monitoring and maintenance) [NEW, incorporating existing §7.9]**	

1 General	1 General
[existing Part 1, § 7.1] **A 4.1.1 Scope and general requirements** (1)P The provisions of this Section apply to ~~end-bearing piles, friction piles, tension piles~~ axially and transversely loaded piles installed by driving, by jacking, by vibration, and by screwing or boring with or without grouting. A(1) This section also applies to ´bored piles´ which are formed as ~~slurry~~ diaphragm wall elements, according to EN 1538~~6~~. ~~(~~(2) A pile is a slender (l/D ≥ 5) monolithic structural element used for transferring loads into ground without need for confining support by the soil. Other systems like stone columns, jet grouting columns, deep soil mixing columns etc. may be designed according to this section. ~~The provisions of this Section should not be applied directly to the design of piles that are intended as settlement reducers, such as in some piled raft foundations~~. (3)P The construction of pile foundations is based on European Execution Standards EN 1536 (Bored Piles), EN 12699 (Displacement Piles) and EN 14199 (Micropiles).~~The following standards shall apply to the execution of piles:~~ ~~— EN 1536:1999, for bored piles~~ ~~— EN 12063:2000, for sheet pile walls,~~ ~~— EN 12699:2000, for displacement piles.~~ ~~— EN 14199;2012 for micropiles~~ ~~NOTE EN 14199 Execution of special geotechnical works - Micro-piles is~~	**1.1 Scope and general requirements** (1)P The provisions of this Section apply to axially and transversely loaded piles installed by driving, by jacking, by vibration, and by screwing or boring with or without grouting. (2) This section also applies to ´bored piles´ which are formed as diaphragm wall elements, according to EN 1538. (3) A pile is a slender (normally l/D ≥ 5) monolithic structural element used for transferring loads into ground. Some aspects of other systems like stone columns, jet grouting columns, deep soil mixing columns etc. can be designed according to this section. (3)P The construction of pile foundations is based on European Execution Standards EN 1536 (Bored Piles), EN 12699 (Displacement Piles) and EN 14199 (Micropiles).

~~in preparation.~~	
~~PL:~~ *~~(3)P Delete EN 12063:2000 Sheet piling; add EN 14199 Micropiles. Delete the NOTE on Micropiles The Standard EN 14199 exists for many years!~~*	
~~A~~ 4.1.2 Classification in Geotechnical Categories	**1.2 Classification in Geotechnical Categories**
~~A~~ (1) ~~Structural measures with~~For classification of pile foundations ~~have to be usually classified~~ into ~~the~~ Geotechnical ~~Category~~ Categories take reference to Annex X.Y.~~GC 2 or into the Geotechnical Category GC 3~~.	(1) For classification of pile foundations into Geotechnical Categories take reference to Annex X.Y. *[FIN: Annex X.Y is needed.]*
~~A (2) A building measurestructure with piles has to be classified into the Geotechnical Category GC 3 when the following criteria are given:~~ ~~- significant cyclic, dynamic or impulsive effects in accordance with A 2.4.2.1 A (8b) and A 2.4.2.1 A (8c);~~ ~~- raked tension piles with an inclination smaller than 45°;~~ ~~- group of tension piles;~~ ~~- grouted pile systems (micropiles in accordance with EN 14199 and grouted displacement piles in accordance with EN 12699) as anchor elements;~~ ~~- determination of the pile resistance for tension from values of experience according to EN 1997-1:2009-09,7.6.3.3;~~ ~~- loading transversal to the pile axis due to lateral soil pressure or settlement bending;~~ ~~- heavily loaded piles in combination with very low allowed settlements;~~ ~~- piles with shaft grouting and/or toe grouting~~ ***~~F:~~*** *~~The classification in Geotechnical Categories has to be more general~~*	

~~*and has to be presented in a general section of Eurocode 7 (not a section devoted a specific geotechnical structure). It is important that this classification takes into account the various geotechnical structures.*~~ ~~*The definition of precise values in this classification is not always pertinent. For example, the value of 45° for the pile inclination is too precise since it seems to state that there is an important change of the behaviour between 44 and 45°.*~~ ~~***PL:** The intention of GC3 is to use it in case of "**very large and unusual structures; involved abnormal risks**" etc. (see 2.1 (21)). This regulation should not be too strict, better would be that "GC3 should be considered in the following cases: … " and left place for decision of the designer.*~~ ~~*In particular: raked tension piles with an inclination smaller than 45°; groups of tension piles, grouted pile systems (micropiles and grouted displacement piles) and piles with shaft grouting and/or toe grouting are routine, often used foundation structures and usually not need to be classified to GC3. Also the piles loaded transversal to their axis due to lateral soil pressure or settlement bending; exception unusual cases. It should be to decision of a designer. **The rule should be to use GC2, GC3 is for unusual situations.***~~ ~~*E.g. – are all anchors classified to GC 3? For anchoring piles and micropiles the GC3 is justified only in soft soils, in the vicinity of buildings etc. Base grouting is a routine construction procedure (e.g. in Poland 1000 to 2000 piles a year – nobody counts them now!). It has proven as an effective, economic technique and assuring high safety of a foundation.*~~ ~~*ADD TO THE LIST: - piled raft foundations of heavy loaded structures. (Not all!)*~~ ~~*A (1) and A (2) Structural measures ?? = structures (look at 2.1 (11))*~~	

2 Limit states	**2 Limit states**
[existing Part 1, § 7.2] (1)P The ~~following~~ limit states in accordance to 2.4.7 and 2.4.8 shall be considered ~~and an appropriate list shall be compiled:~~ ~~— loss of overall stability;~~ ~~— bearing resistance failure of the pile foundation;~~ ~~— uplift or insufficient tensile resistance of the pile foundation;~~ ~~— failure in the ground due to transverse loading of the pile foundation;~~ ~~— structural failure of the pile in compression, tension, bending, buckling or shear;~~ ~~— combined failure in the ground and in the pile foundation;~~ ~~— combined failure in the ground and in the structure;~~ ~~— excessive settlement;~~ ~~— excessive heave;~~ ~~— excessive lateral movement;~~ ~~— unacceptable vibrations.~~ *D: Comment: List is already included in section 2, there is no need for repeat.*	(1)P The limit states in accordance to 2.4.7 and 2.4.8 shall be considered. *[FIN: Almost similar check-list of different limit states should be kept (if not in this paragraph at least as an annex). Limit states mentioned in 2.4.7 and 2.4.8 are to general to cover all the aspects of pile foundation].*

3 Actions and design situations	3 Actions and design situations
3.1 General	**3.1 General**
[existing Part § 7.3.1] (1) The actions listed in 2.4.2(4) should be considered when selecting the design situations. ~~(2) Piles can be loaded axially and/or transversely.~~ (3)P Design situations shall be derived in accordance with 2.2. (4) An analysis of the interaction between structure, pile foundation and ground can be necessary to prove that the limit state requirements are met. ~~A~~ (1) The non-linearity of the resistance-settlement curve of axially loaded piles ~~has~~ may need to be considered in calculation of the structure. ~~For simplification it is possible to determine a spring constant from the secant through the resistance-settlement curve for the characteristic stress range.~~	(1) The actions listed in 2.4.2(4) should be considered when selecting the design situations. (2)P Design situations shall be derived in accordance with 2.2. (3) An analysis of the interaction between structure, pile foundation and ground can be necessary to prove that the limit state requirements are met. (4) The non-linearity of the resistance-settlement curve of axially loaded piles may need to be considered in calculation of the structure.
3.2 Dynamic and cyclic loading	**3.2 Dynamic and cyclic loading**
A (2) Cyclic and dynamic stresses on piles can cause a significant reduction of long-term bearing capacity and additional displacements. ***F:*** *Taking into account cyclic effects is important but every loading is cyclic since there is always permanent and transient loads. This aspect has to be more precise.*	(1) Cyclic and dynamic stresses on piles can cause a significant reduction of long-term bearing capacity and additional displacements. **New clauses to be elaborated for next revision of EN 1997**

3.3 Actions due to ground displacement	**3.3 Actions due to ground displacement**
3.3.1 General	**3.3.1 General**
[existing Part § 7.3.2.1] **3.3.1 General** (1)P Ground in which piles are located may be subject to displacement caused by consolidation, swelling, adjacent loads, creeping soil, landslides or earthquakes. Ground displacement can also be caused by construction/installation of piles, or nearby excavations or dewatering. Consideration shall be given to these phenomena as ~~they~~ can affect the piles by causing downdrag (negative skin friction), heave, ~~stretching,~~ transverse loading and displacement. (2) For these situations, the ~~design~~ most unfavourable values of the strength and stiffness of the moving ground should ~~usually~~ be used which in this case might be upper bound values. (3)P One of the two following approaches shall be adopted for design: — the ground displacement is treated as an action. An interaction analysis is then carried out to determine the forces, displacements and strains in the pile; — a~~n upper bound to the~~ force, which the ground could transmit to the pile shall be introduced as the design action. Evaluation of this force shall take account of the strength of the soil and the source of the load~~,~~. ~~represented by the weight or compression of the moving soil or the magnitude of disturbing actions~~.	**3.1 General** (1)P Ground in which piles are located may be subject to displacement caused by consolidation, swelling, adjacent loads, creeping soil, landslides or earthquakes. Ground displacement can also be caused by construction/installation of piles, or nearby excavations or dewatering. Consideration shall be given to these phenomena as can affect the piles by causing downdrag (negative skin friction), heave, transverse loading and displacement. (2) For these situations, the most unfavourable values of the strength and stiffness of the moving ground should be used which in this case might be upper bound values. (3)P One of the two following approaches shall be adopted for design: — the ground displacement is treated as an action. An interaction analysis is then carried out to determine the forces, displacements and strains in the pile; — a force, which the ground could transmit to the pile shall be introduced as the design action. Evaluation of this force shall take account of the strength of the soil and the source of the load.

3.3.2 Downdrag (negative skin friction)	3.3.2 Negative skin friction (downdrag)
[existing Part § 7.3.2.2] (1)P If ultimate limit state design calculations are carried out with the downdrag load as an action, its value shall be the maximum, which could be generated by the downward movement of the ground relative to the pile. ~~(2) Calculation of maximum downdrag loads should take account of the shear resistance at the interface between the soil and the pile shaft and downward movement of the ground due to self-weight compression and any surface load around the pile.~~ *D: Comment: Basic knowledge.* (3) An upper bound to the downdrag load on a group of piles may be calculated from the weight of the surcharge causing the movement and taking into account any changes in ground-water pressure due to ground-water lowering, consolidation or pile driving. (4) Where settlement of the ground after pile installation is expected to be small, an economic design may be obtained by treating the settlement of the ground as the action and carrying out an interaction analysis. ~~(5)P The design value of the settlement of the ground shall be derived taking account of material weight densities and compressibility in accordance with 2.4.3.~~ *D: Comment: Is regulated already in Section 2.* ~~(6) Interaction calculations should take account of the displacement of the pile relative to the surrounding moving ground, the shear resistance of the soil along the shaft of the pile, the weight of the soil and the~~	(1) Negative skin friction has to be regarded as a permanent action F_n, originating from relative axial movement between the ground and the pile, when the ground settles more than the pile. (2) The pile continues to settle until the actions from negative skin friction τ_n, together with the actions imposed on the pile by the superstructure, and the pile resistances resulting from the pile end bearing capacity and supporting skin friction q_s, are in equilibrium. (3) Negative skin friction should be taken into account for the justification at SLS and ULS. (4) Both for SLS and ULS, the characteristic values of negative skin friction should be estimated by considering the pile loading level with appropriate calculation models, see (5), accounting for relevant strain mechanisms between the pile and the soil surrounding. For simple cases approximates approach can be used, see (6). (5) An appropriate model to calculate negative skin friction is to take into account up to the neutral point marking the boundary between positive and negative skin friction. Two separate neutral points should be considered for the ULS and SLS (Figure 1). In the case of SLS the neutral point is the point of the theoretical zero relative movement between the pile and the settling soil considering total possible settlement (e.g. primary and secondary consolidation). In the case of the ULS the neutral point should be moved up relative to the SLS neutral point. To calculate the negative skin friction $\tau_{n,k}$ information are required on:

~~expected surface loads around each pile, which are the cause of the downdrag.~~

D: Comment: Is regulated in (1) resp. basic knowledge.

~~(7) Normally, downdrag and transient loading need not be considered simultaneously in load combinations.~~

D: Comment: ´Normally´ does not help a user.

PL: *I would not delete this statement – it is a useful guidance, decision should be up to designer.*

- pile settlements with depth;
- soil strata settlements with depth;
- the resulting relative movements and
- any mobilisation functions of $\tau_{n,k}$ and $q_{s,k}$.

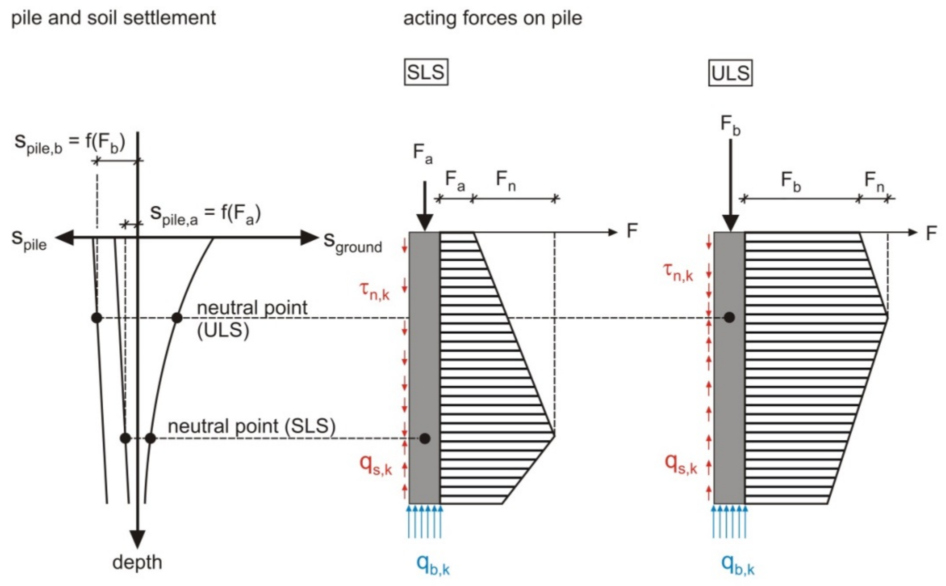

Figure 1 Evaluation of negative skin friction for ultimate limit state (ULS) and serviceability limit state (SLS)

Comparing the relative displacements from pile settlement s_{pile} and the settlement of the surrounding soil s_{ground} gives the location of the neutral point and thus the value of the characteristic action $F_{n,k}$ in the serviceability limit state and in the ultimate limit state.

(6) For simple cases the following approximate approaches can be used alternatively to (5):

a) In case that at ultimate limit state the settlements of the pile can be assumed to be greater than the settlement of the surrounding soil the neutral point can be assumed to be located at the ground surface resp. at top of the pile. Thus the pile resistance can be proven for the ultimate limit state without calculating a negative skin friction.

b) In case that at ultimate limit state the settlements of the pile has to be considered to be smaller than the settlement of the surrounding soil (example: “rock socket piles”) the neutral point can be assumed to be located at the bottom of the settling soil layer. Thus the pile resistance can be proven for the ultimate limit state by assuming characteristic values of negative skin friction for the settling soil layers.

c) For serviceability limit state the calculation of the neutral point can be substituted by assuming characteristic values of negative skin friction for the settling soil layers, thus considering the neutral point to be located at the bottom of the settling soil layers. This value $\tau_{n,k}$ should be an upper bound value and be determined on the safe side.

(7) Approaches for deriving the characteristic negative skin friction $\tau_{n,k}$ are given in Annex A.

(8) Normally, downdrag and transient loading need not be considered simultaneously in load combinations.

(9) The pile resistance has to be proven analysing the ultimate and

serviceability limit state:

a) Serviceability limit state (SLS): the characteristic action $F_{n,k}$(SLS) and the location of the neutral point have to be calculated by the deformation behaviour associated with the pile settlement s_{pile} and the settlements in the soft stratum s_{ground}. The design value of the effects is:

$$F_d = F_k = F_{G,k} + F_{n,k}(SLS) + F_{Q,rep} \tag{1}$$

Additionally it should be proven that the pile displacements are compatible with the supported structures.

b) Ultimate limit state (ULS): the characteristic action $F_{n,k}$(ULS) and the location of the neutral point have to be calculated by comparing the deformations associated with the pile settlement $s_{pile} = s_{ult}$in the ultimate limit state and the settlements in the soft stratum s_{ground}. The location of the neutral point is normally higher than in the serviceability limit state, because the pile settlement s_{ult} is greater than s(SLS). The design value of the effects is:

$$F_d = (F_{G,k} + F_{n,k}(ULS)) \cdot \gamma_G + F_{Q,rep} \cdot \gamma_Q \tag{2}$$

(10) For the structural analysis of the pile shaft the action resulting from negative skin friction at serviceability limit state, i.e. $F_{n,k}$(SLS), has to be considered as permanent characteristic action beside permanent and transient loading from the structure. The design value of the action relevant for the internal pile design has to be calculated as

$$F_d = (F_{G,k} + F_{n,k}\ (SLS)) \cdot \gamma_G + F_{Q,k} \cdot \gamma_{Q.} \tag{3}$$

Note: *To be discussed whether the same partial factor should be applied for structural loads and negative skin friction.*

Annex A (Informative)

Two principle approaches for deriving the characteristic negative skin friction $\tau_{n,k}$ are given in the literature dealing with negative skin friction:

– Using total stresses for cohesive soils

$$\tau_{n,k} = \alpha \cdot c_{u,k} \tag{A.1}$$

where:

α factor for specifying the value of the characteristic negative skin friction for cohesive soils;

$c_{u,k}$ characteristic value of the shear strength of the undrained soil.

Depending on the soil type and pile type the factor α generally ranges between 0.15 and 1.60, whereby α = 1 is often adopted in approximation, which is generally recommended for cohesive soils.

More detailed information on the value of α can be taken from [Literature].

– Using effective stresses for non-cohesive and cohesive soils:

$$\tau_{n,k} = K_0 \cdot \tan\delta_k \cdot \sigma'_v = \beta \cdot \sigma'_v \tag{A.2}$$

where:

σ'_v effective vertical stress;

K_0 coefficient of at-rest earth pressure;

δ_k characteristic value of the interface friction angle, whereby

$\delta_k = \varphi'_k$ for concrete cast in situ piles,
$\delta_k = 0.75\,\varphi'_k$ for concrete precast and steel piles,
but $K_0 \cdot \tan\delta_k \geq 0.25$

	φ'_k characteristic value of the friction angle; β factor for specifying the value of the characteristic negative skin friction for non-cohesive and cohesive soils. According to [Literature].the factor β generally ranges between 0.1 and 1.0, depending on soil type. For non-cohesive soils β = 0.20 to 0.30 is often used.
3.3.3 Heave	**3.3.3 Heave**
[existing Part § 7.3.2.3] (1)P In considering the effect of heave, or upward loads, which may be generated along the pile shaft, the movement of the ground shall ~~generally~~ be treated as an action. ~~NOTE 1 Expansion or heave of the ground can result from unloading, excavation, frost action or driving of adjacent piles. It can also be due to an increase of the ground-water content resulting from the removal of trees, cessation of abstraction from aquifers, prevention (by new construction) of evaporation and from accidents.~~ ~~NOTE 2 Heave may take place during construction, before piles are loaded by the structure, and may cause unacceptable uplift or structural failure of the piles.~~ *D: Comment: That is stuff for a textbook.* *NL: The present Eurocode 7 functions as a check list for designers and also fort he authorities. This is regarded as useful in NL. To my opinion deleting all these check lists requires a discussion on the level of projectteam in charge for*	(1)P In considering the effect of heave, or upward loads, which may be generated along the pile shaft, the movement of the ground shall be treated as an action.

<table>
<tr><td>redrafting EC7.</td><td></td></tr>
<tr><td>3.3.4 Transverse loading</td><td></td></tr>
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[existing Part § 7.3.2.4]

(1)P In the following cases consideration shall be given to transverse actions originating from ground movements or asymmetric loads around a pile:

~~(2) Consideration should be given to the following list of design situations, which may result in transverse actions on a pile:~~

— different amounts of surcharge on either side of a pile foundation (e.g. in or near an embankment);
— different levels of excavation on either side of a pile foundation (e.g. in or near a cutting);
— a pile foundation constructed in a creeping slope;
— inclined piles in settling ground;
— piles in a seismic region.

~~(3) Transverse loading should normally be evaluated by considering the interaction between the piles, treated as stiff or flexible beams, and the moving soil mass. When the horizontal deformation of weak soil layers is large and the piles are widely spaced, the resulting transverse loading of the piles depends mainly on the shear strength of the weak soil layers.~~

Comment: That is stuff for a textbook.
</td><td>
3.3.4 Transverse Loading (existing § 7.3.2.4)

(1) In the following cases consideration shall be given to transverse actions on a pile originating from ground movements or asymmetric loads around a pile:
- different amounts of surcharge on either side of a pile foundation (e.g. in or near an embankment or for piles supporting a retaining structure);
- different levels of excavation on either side of a pile foundation (e.g. in or near a cutting);
- a pile foundation constructed in a creeping slope or piles arranged for stabilization within a slope;
- inclined piles in settling ground;
- piles in a seismic region.

(2) Generally an interaction analysis shall be carried out to simulate the soil-pile interaction and to determine the displacements, forces and strains in the pile shaft. For the analysis the characteristic values of the ground parameters and of the pile material as well as characteristic values for the stresses should be used.

To consider the impact of the spatial soil variability as well as of the uncertainty in determining the shear
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</table>

strength and stiffness of the ground a range should be specified for these parameters and upper and lower values of this range should be considered for the analysis of the soil-pile interaction; this is of special relevance for multi-layered ground conditions.

The characteristic strains and forces calculated for the pile shaft shall be used as permanent characteristic actions for the structural design.

(3) In simple cases a force which the ground could transmit to the pile can be introduced to consider the transverse pressure on the pile caused by the ground movements. The transverse actions on a pile originating from ground displacement shall then be treated as a permanent action.

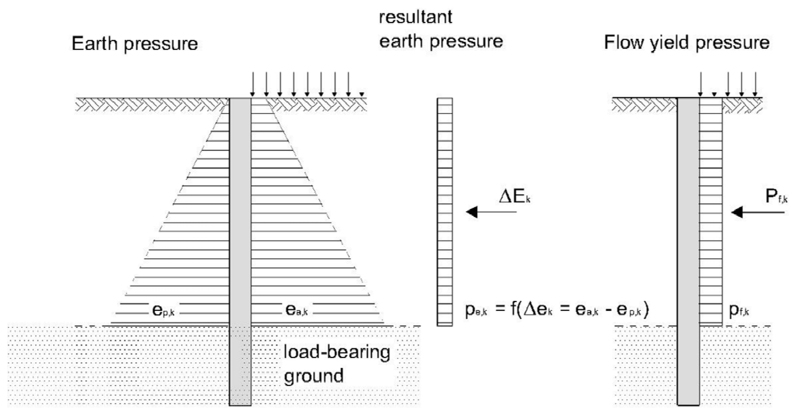
NOTE: The action resulting from ground movments transverse to the pile axis may then be determined e.g. by the following approaches:

– *the characteristic flow pressure* $p_{f,k}$ *and*

– *the characteristic resulting earth pressure* Δe_k

– *for inclined piles the weight of the overlaying soil.*

~~Approaches for calculating the characteristic flow pressure~~ ~~$p_{f,k}$~~ ~~and the characteristic resulting earth pressure~~ ~~Δe_k~~ ~~are documented in Annex A.~~

	 **Figure 1** Simplified approach to determine the action resulting from ground movements lateral to the pile axis
4 Design methods and design considerations	**4 Design methods and design considerations**
	4.1 Design by calculation
4.1 General	**4.1.1 General**
[NEW, incorporating existing Part 1, § 7.4] [as Section ´General´ in § 4.4 NEW?] (1)P The design shall be based on one of the following approaches: — the results of ~~static~~ load tests~~, which have been demonstrated, by means of calculations or otherwise, to be consistent with other~~	(1)P The design shall be based on one of the following approaches: — the results of load tests; — empirical or analytical calculation methods;

~~relevant experience~~; — empirical or analytical calculation methods ~~whose validity has been demonstrated by static load tests in comparable situations~~; — ~~the results of dynamic load tests whose validity has been demonstrated by static load tests in comparable situations~~; — the observed performance of a comparable pile foundation~~, provided that this approach is supported by the results of site investigation and ground testing~~. *Validation to address in specific subsection.* ***F:** The clause 7.4.1.1(P) has to be maintained. It defines the principles of pile design and these principles are not restrictions.* ***PL**: I propose to recommend testing of trial piles as the most rational (despite it is usually complicated).* ~~(2) Design values for parameters used in the calculations should be in general accordance with Section 3, but the results of load tests may also be taken into account in selecting parameter values.~~ *D: Comment: Clause is self-evident.* ~~(3) Static load tests may be carried out on trial piles, installed for test purposes only, before the design is finalised, or on working piles, which form part of the foundation.~~ *D: Comment: Clause is dispensable.*	— the observed performance of a comparable pile foundation. **[*FIN:* It seems that lot of "good text" are planned to be deleted. Even though some of the text is self evident, handbook stuff and related to execution standards, clauses are good in general level and should be kept*]*.**
~~4.4.2 Design considerations~~ [has to be modified] ~~(1)P The behaviour of individual piles and pile groups and the stiffness~~	

~~and strength of the structure connecting the piles shall be considered.~~ ~~(2)P In selecting calculation methods and parameter values and in using load test results, the duration and variation in time of the loading shall be considered.~~ ~~(3)P Planned future placement or removal of overburden or potential changes in the ground-water regime shall be considered, both in calculations and in the interpretation of load test results.~~ *D: Comment: Clauses are all self-evident.* ~~(4)P The choice of type of pile, including the quality of the pile material and the method of installation, shall take into account:~~ ~~— the ground and ground-water conditions on the site, including the presence or possibility of obstructions in the ground;~~ ~~— the stresses generated in the pile during installation;~~ ~~— the possibility of preserving and checking the integrity of the pile being installed;~~ ~~— the effect of the method and sequence of pile installation on piles, which have already been installed and on adjacent structures or services;~~ ~~— the tolerances within, which the pile can be installed reliably;~~ ~~— the deleterious effects of chemicals in the ground;~~ ~~— the possibility of connecting different ground-water regimes;~~ ~~— the handling and transportation of piles;~~ ~~— the effects of pile construction on neighbouring buildings.~~ *D+NL: Comment: Like a textbook. Probably to be discussed whether*	

some issues should be kept as a check-list specific for piles. ~~(5) In considering the aspects listed above, the following items should receive attention:~~ ~~— the spacing of the piles in pile groups;~~ ~~— displacement or vibration of adjacent structures due to pile installation;~~ ~~— the type of hammer or vibrator used;~~ ~~— the dynamic stresses in the pile during driving;~~ ~~— for those types of bored pile where a fluid is used inside the borehole, the need to keep the pressure of the fluid at a level to ensure that the borehole will not collapse and that hydraulic failure of the base will not occur;~~ ~~— cleaning of the base and sometimes the shaft of the borehole, especially under bentonite, to remove remoulded materials;~~ ~~— local instability of a shaft during concreting, which may cause a soil inclusion within the pile;~~ ~~— ingress of soil or water into the section of a cast-in-situ pile and possible disturbance of wet concrete by the flow of water through it;~~ ~~— the effect of unsaturated sand layers around a pile extracting water from the concrete;~~ ~~— the retarding influence of chemicals in the soil;~~ ~~— soil compaction due to the driving of displacement piles;~~ ~~— soil disturbance due to the boring of a pile shaft.~~ *Comment: Clauses are subject of codes for execution of piles.*	

4.1.2 Single standing piles	**4.1.2 Single standing piles**
4.1.2.1 Axially loaded piles	**4.1.2.1 Axially loaded piles**
Ultimate compressive resistance from ground test results [7.6.2.3] *NL: we think that a limited number of calculation methods must be introduced in this paragraph. E.g. the so-called direct CPT method as used in Belgium, France and The Netherlands and the method based on the Cone Pressiometer as used in France.* *Those methods are now presented in the national NCCI 's (Non Conflicting Complementary Information). But as the evaluation committee wants to limit the number of NCCI 's the calculation methods must be in the code* (1)P Methods for assessing the compressive resistance of a pile foundation from ground test results shall have been established from pile load tests and from comparable experience as defined in 1.5.2.*2.* ~~(2) A model factor may be introduced as described in 2.4.1(9) to ensure that the predicted compressive resistance is sufficiently safe.~~ *D: Comment: Might be shifted to an informative annex if necessary.* ***F:*** *The clause 7.6.2.4(2) has to maintained. The possibility to use a model factor is the chance to go to a harmonization. We have to define a first set of model factors and after each country can add a model factor to takie into account its own practice.* *I agree that the calibration or the choice of this model factor has to be presented in an annex.*	

(3)P The design compressive resistance of a pile, $R_{c;d}$, shall be derived from:

$$R_{c;d} = R_{b;d} + R_{s;d} \tag{7.6}$$

(4)P For each pile, $R_{b;d}$ and $R_{s;d}$ shall be obtained from:

$$R_{b;d} = R_{b;k}/\gamma_b \text{ and } R_{s;d} = R_{s;k}/\gamma_s \tag{7.7}$$

NOTE The values of the partial factors may be set by the National annex. The recommended values for persistent and transient situations are given in Tables A.6, A.7 and A.8.

(5)P The characteristic values $R_{b;k}$ and $R_{s;k}$ shall either be determined by:

$$R_{c;k} = \left(R_{b;k} + R_{s;k}\right) = \frac{R_{b;cal} + R_{s;cal}}{\xi} = \frac{R_{c;cal}}{\xi} = \mathrm{Min}\left\{\frac{\left(R_{c;cal}\right)_{mean}}{\xi_3}; \frac{\left(R_{c;cal}\right)_{min}}{\xi_4}\right\} \tag{7.8}$$

where ξ_3 and ξ_4 are correlation factors that depend on the number of profiles of tests, n, and are applied respectively:

— to the mean values
$(R_{c;cal})_{mean} = (R_{b;cal} + R_{s;cal})_{mean} = (R_{b;cal})_{mean} + (R_{s;cal})_{mean}$

— and to the lowest values
$(R_{c;cal})_{min} = (R_{b;cal} + R_{s;cal})_{min}$,

or by the method given in 7.6.2.3(8).

NOTE The values of the correlation factors may be set by the National annex. The recommended values are given in Table A.10. *[FIN: Correlation factors: It seems that there are lots of challenges to harmonize the correlation factors.* *Proposed Change:* *Together with the exact number of load tests or soil investigations to determine the correlation factor, there should be a percentage of all piles (tested) which also can determine the correlation factor. Number of piles can be something from few piles to hundreds of piles at the same location, so percentage is better when thinking of reliability.* *Typical situation in Finland with small bridges: there's only 4 piles (d=600…900mm diameter steel pipe piles) which continue as columns to bridge deck: every pile is tested with dynamic pile load test (100%). The lowest correlation factor is used (1.4 / 1.25) not the correlations based on number of piles (which would lead to 1.6 / 1.5)* *The quality (and quantity) of soil investigations and level of quality assurance should be able to consider when level safety (partial factors of resistance) are chosen].* ~~(6)P The systematic and random components of the variation in the ground shall be recognised in the interpretation of the ground tests and calculated resistances.~~ *D: Comment: Self-evident.* (7) For structures with sufficient stiffness and strength to transfer loads from "weak" to "strong" piles, the factors ξ_3 and ξ_4 may be divided by 1,1, provided that ξ_3 is never less than 1,0. (8) The characteristic values may be obtained by calculating: $R_{b;k} = A_b\, q_{b;k}$ and $R_{s;k} = \sum_i A_{s;i} \cdot q_{s;i;k}$ (7.9)	

where $q_{b;k}$ and $q_{s;i;k}$ are characteristic values of base resistance and shaft friction in the various strata, obtained from values of ground parameters. [D: Most common approach. Change order.] ~~NOTE If this alternative procedure is applied, the values of the partial factors γ_b and γ_s recommended in Annex A may need to be corrected by a model factor larger than 1,0. The value of the model factor may be set by the National annex.~~ *D: Comment: Does a ´may´-clause help?* ~~For micropiles to EN 14199 pile resistances from experiences should be used only in justified exceptional cases. Normally the pile toe resistance is not allowed to be considered.~~ ~~The normal case for micropiles are static pile load tests, see 7.6.2.2 A (1a).~~ *[Nordic end-bearing micropiles are a standard solution]* (9)P If Design Approach 3 is used, the characteristic values of ground parameters shall be determined according to 2.4.5. Partial factors shall then be applied to these characteristic values to obtain design values of the ground parameters for calculating the design values of the pile resistance. ~~(10) In assessing the validity of a model based on ground test results, the following items should be considered:~~ ~~— soil type, including grading, mineralogy, angularity, density, pre-consolidation, compressibility and permeability;~~ ~~— method of installation of the pile, including method of boring or driving;~~ ~~— length, diameter, material and shape of the shaft and of the base of the pile (e.g. enlarged base);~~ ~~— method of ground testing.~~	

D: Comment: Self-evident that this conditions should be comparable.	
Ultimate tensile resistance from ground test results [7.6.3.3] (1)P Methods for assessing the tensile resistance of a pile foundation from ground test results shall have been established from pile load tests and from comparable experience as defined in 1.5.2.2. A (1) For the determination of the tension pile resistances on the basis of experiences 7.6.2.3 applies analogously, in which in particular the included limitations of the methods have to be considered. For the decision not to execute pile load tests for tension piles, expertise and experience is required in geotechnical engineering. The decision to execute no load tests for tension piles has to be justified project-oriented on the basis of comparable tension pile load tests. ~~(2) A model factor may be introduced as described in 2.4.1(9) to ensure that the predicted tensile resistance is sufficiently safe.~~ (3)P The design value of tensile resistance of a pile, $R_{t;d}$, shall be derived from: $R_{t;d} = R_{t;k} / \gamma_{s;t}$ (7.15) where: $R_{t;k} = R_{s;k}$ (7.16) NOTE The values of the partial factor may be set by the National annex. The recommended values for persistent and transient situations are given in Tables A.6, A.7 and A.8 .	

(4)P The characteristic value $R_{t;k}$ shall either be determined by:

$$R_{t;k} = \text{Min}\left\{\frac{(R_{s;cal})_{mean}}{\xi_3};\frac{(R_{s;cal})_{min}}{\xi_4}\right\} \quad (7.17)$$

where ξ_3 and ξ_4 are correlation factors that depend on the number of profiles of tests, n, and are applied respectively to the mean $(R_{s;cal})_{mean}$ and to the lowest value $(R_{s;cal})_{min}$ of $R_{s;cal}$, or by the method given in 7.6.3.3(6).

NOTE The values of the correlation factors may be set by the National annex. The recommended values are given in Table A.10.

~~(5)P The systematic and random components of the variation in the ground shall be recognised in the interpretation of the calculated tensile resistance.~~

D: Comment: General comment, guilty for all sections. Shift to section 2.

(6) The characteristic value of tensile resistance may be obtained by calculating:

$$R_{t;k} = \sum_i A_{s;i} \cdot q_{s;i;k} \quad (7.18)$$

where $q_{s;i;k}$ are characteristic values of shaft friction in the various strata obtained from values of ground properties.

~~NOTE If this alternative procedure is applied, the value of the partial factor $\gamma_{s,t}$ recommended from Annex A, may need to be corrected by a model factor larger than 1,0. The value of the model factor may be set by the National annex.~~

D: Comment: not normative.

(7)P If Design Approach 3 is used, the characteristic values of ground parameters shall be determined according to 2.4.5; partial factors shall then be applied to these characteristic values to obtain design values of the ground parameters to calculate the design values of the pile resistance. *D: Comment: is design approach 3 still needed or chance for harmonization?.* ~~(8) The assessment of the validity of a model based on ground test results should be in accordance with 7.6.2.3(10).~~ *D: Comment: general recommendation with no specific relation here.*	
4.1.2.2 Transversely loaded piles	
[7.7.3 Transverse load resistance from ground test results and pile strength parameters] ~~(1)P The transverse resistance of a pile or pile group shall be calculated using a compatible set of structural effects of actions, ground reactions and displacements.~~ A (1) To estimate the lateral soil reaction in front of vertical piles, e.g. by the subgrade modulus or earth resistance, the parameters for stiffness and shear strength of the soil have to be specified; they have to be determined from field or laboratory tests, in case there are no local experiences existent and no pile load test performed. ~~(2)P The analysis of a transversely loaded pile shall include the possibility of structural failure of the pile in the ground, in accordance with 7.8.~~	

D: Comment: self-evident static proof. ~~(3) The calculation of the transverse resistance of a long slender pile may be carried out using the theory of a beam loaded at the top and supported by a deformable medium characterised by a horizontal modulus of subgrade reaction.~~ *D: Comment: see A(1).* A (3) The subgrade moduli of the involved soil layers may be applied by the equation A (7.20), when they only serve to determine the internal forces $k_{s,k} = E_{s,k} / D_s$ with $k_{s,k}$ the value of the subgrade modulus for a level of stress in soil with characteristic or representative interactions; $E_{s,k}$ the value of the constrained modulus for a level of stress in soil with characteristic or representative interactions; D_s diameter of the pile shaft The scope of the equation A (7.20) is limited by a computational maximum characteristic horizontal displacement of either 2,0 cm or 0,03 · D_s; the smaller of the values is decisive. *[FIN: where comes the limitations 2.0 cm or 0.03 x Ds. These seems to be very small values?]* Size and distribution along the pile in the soil of the subgrade modulus have to be determined from pile load tests, if the deformations of the pile foundation are important for the structural behavior of the structure and if there are no experiences. *[FIN: How to arrange and interpret the pile load test?]*	

(4)P The degree of freedom of rotation of the piles at the connection with the structure shall be taken into account when assessing the foundation's transverse resistance.	
4.1.3 Pile Groups	
4.1.3.1 Axially loaded piles	
[New]	
4.1.3.2 Transverselly loaded piles	
[New]	
4.1.4 Piled rafts	
[New]	
4.1.5 Pile resistance due to cyclic, dynamic and impact loads	
[New]	
4.2 Design by testing	
4.2.1 General	
[existing Part 1, § 7.5 renamed, has to be modified] **[7.5.1 General]** (1)P Pile load tests shall be carried out in the following situations: — when using a type of pile or installation method for which there is no comparable experience;	

— when the piles have not been tested under comparable soil and loading conditions; — when the piles will be subject to loading for which theory and experience do not provide sufficient confidence in the design. The pile testing procedure shall then provide loading similar to the anticipated loading; — when observations during the process of installation indicate pile behaviour that deviates strongly and unfavourably from the behaviour anticipated on the basis of the site investigation or experience, and when additional ground investigations do not clarify the reasons for this deviation. ***PL:*** *The Polish standard for pile design says differently: pile load tests should be done as a rule, with exceptions of small number of piles and so on, provided there are no objections regarding quality of piles. It is considered as "hard rules but good rules'. And it pays!* *I propose more strictly to recommend performing of load tests, particularly for piles in tension, and to require them in the listed situations.* ~~(2) Pile load tests may be used to:~~ ~~— assess the suitability of the construction method;~~ ~~— determine the response of a representative pile and the surrounding ground to load, both in terms of settlement and limit load;~~ ~~— to allow judgement of the overall pile foundation.~~ *Comment: That is like a textbook.* ~~(3) Where load tests are not practical due to difficulties in modelling the variation in the load (e.g. cyclic loading) very cautious design values for~~	

~~the material properties should be used.~~ (3) As far as cyclic loads has to be considered not only a static pile load test should be executed but also additionally the cyclic loads should be simulated by a test considering the amplitude and the number of cycles realistically for the serviceability limit state. *[NL: This is sometimes not practical and the need depends largely on the ratio of cyclic over static and this ratio depends of the pile typ.* *This issue requires more attention]* ***PL:*** *It is OK. But it should be somehow limited to 'serious' cyclic loadings (as was above). Take into account that any building is under cyclic loading from wind, and any bridge from traffic. Is the proposed requirement for each of these structures?* (4)P If one pile load test is carried out, it shall ~~normally~~ be located where the most adverse ground conditions are believed to occur. If this is not possible, an allowance shall be made when deriving the characteristic value of the compressive resistance. ~~(5)P If load tests are carried out on two or more test piles, the test locations shall be representative of the site of the pile foundation and one of the test piles shall be located where the most adverse ground conditions are believed to occur.~~ (5) In highly variable soil conditions within a construction area several pile load tests have to be performed, with which the specific resistance of the piles in the areas of different soil properties is evaluated reliably, see 7.6.2.2 A (10). ~~(6)P Between the installation of the test pile and the beginning of the load test, adequate time shall be allowed to ensure that the required strength of the pile material is achieved and the pore-water pressures have regained their initial values.~~	

~~(7) In some cases it can be necessary to record the pore-water pressures caused by pile installation and their subsequent dissipation in order to take a proper decision regarding the start of the load test.~~ *[D: Comment: That´s like a textbook and not concrete enough.]* *[NL:* Deleting is not the solution. Attention must be paid tot he occurence of excess pore pressures as the residual from the installation as the generated during (dynamic and quasi-static) load testing]	
4.2.2 Axially loaded piles	
4.2.2.1 Ultimate resistance from static load test	
[7.6.2.2 Ultimate compressive resistance from static load tests] ~~(1)P The manner in which load tests are carried out shall be in accordance with 7.5 and shall be specified in the Geotechnical Design Report.~~ *Comment: Already mentioned before.* For micropiles under compression in accordance to EN 14199 static pile load tests have to be performed – at at least 3% of the planned number of piles not less than n = 2 piles have to be tested. [NL: Refer to 7.5.2.2. *[FIN: Reference to EN14199 should be deleted (Micropile pile static load test reference to EN14199 is misleading. It's not obligatory to have static pile load test for micropiles according to EN14199. FIN, NOR rely on bedrock and no tests are done (end-bearing piles in bedrock)]*	

(2)P Trial piles to be tested in advance shall be installed in the same manner as the piles that will form the foundation and shall be founded in the same stratum. ~~(3) If the diameter of the trial pile differs from that of the working piles, the possible difference in performance of piles of different diameters should be considered in assessing the compressive resistance to be adopted.~~ *D: Comment: Does not help, relevant is clause (4).* (4) In ~~the case of a very large diameter pile, it is often impractical to carry out a load test on a full size trial pile. Load tests on smaller diameter trial piles may be considered~~ For bored piles with diameters D ≥ 0,8 m the pile-resistance can be determined by load tests on trial piles with smaller diameters than the building piles provided that: — the ratio of the trial pile diameter/working pile diameter is not less than 0,5; — the smaller diameter trial pile is fabricated and installed in the same way as the piles used for the foundation; — the trial pile is instrumented in such a manner that the base and shaft resistance can be derived separately from the measurements. ~~This approach should be used with caution for open-ended driven piles because of the influence of the diameter on the mobilisation of the compressive resistance of a soil plug in the pile.~~ *D: Comment: Should be limited to bored piles.* For micropiles to EN 14199 piles with same diameter and length like the planned working piles have to be proven by pile load tests. (5)P In the case of a pile foundation subjected to downdrag, this must be considered by planning and evaluation of the static pile load test.	

~~the pile resistance at failure, or at a displacement that equals the criterion for the verification of the ultimate limit state determined from the load test results, shall be corrected. The correction shall be achieved by subtracting the measured, or the most unfavourable, positive shaft resistance in the compressible stratum and in the strata above, where negative skin friction develops, from the loads measured at the pile head.~~

~~(6) During the load test of a pile subject to downdrag, positive shaft friction will develop along the total length of the pile and should be considered in accordance with 7.3.2.2(6). The maximum test load applied to the working pile should be in excess of the sum of the design external load plus twice the downdrag force.~~

D: Comment: textbook like.

(7)P When deriving the ultimate characteristic compressive resistance $R_{c;k}$ from values $R_{c;m}$ measured in one or several pile load tests, an allowance shall be made for the variability of the ground and the variability of the effect of pile installation.

(8)P For structures, which do not exhibit capacity to transfer loads from "weak" piles to "strong" piles, as a minimum, the following equation shall be satisfied:

$$R_{c;k} = \mathrm{Min}\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\} \quad (7.2)$$

where ξ_1 and ξ_2 are correlation factors related to the number of piles tested and are applied to the mean $(R_{c;m})_{mean}$ and the lowest $(R_{c;m})_{min}$ of $R_{c;m}$ respectively.

NOTE The values of the correlation factors may be set by the National annex. The recommended values are given in Table A.9.

German NDP according to 7.6.2.2 8(P)

The values of the correlation factors ξ_1 and ξ_2 for pile foundations for deriving the characteristic pile resistance in accordance with equation 7.2 are given in Table A. 7.1.

Tabelle A 7.1: *correlation factors ξ_i for deriving characteristic values of static pile load tests.*

n	**1**	**2**	**3**	**4**	**≥ 5**
ξ_1	1,35	1,25	1,15	1,05	1,00
ξ_2	1,35	1,15	1,00	1,00	1,00
n is he number of test load piles					

→ Chance for harmonization?

F: *Table A.7.1*

In order to define $\xi_{1\text{-}2}$ factors, we have to take into account the distance between the static load test and the piles of the project. If some piles are located at the static load pile (or very close), a lower value for ξ can be considered, maybe 1.2.

In France, ξ factors vary according to the number of pile load tests and the surface of the project. I can present this approach in Vienna if you want. This approach is extended to $\xi_{3\text{-}4}$.

(9) For structures having sufficient stiffness and strength to transfer loads from "weak" to "strong" piles, the values of ξ_1 and ξ_2 may be divided by 1,1, provided that ξ_1 is never less than 1,0.

~~(10)P The systematic and random components of the variations in the ground shall be recognised in the interpretation of pile load tests.~~

(10) If the ground in the construction area of the planned pile foundation has stronger changes in the stratigraphic sequence and in the soil conditions, only areas with same soil conditions are allowed to consider for the number n of test piles.

~~(11)P The records of the installation of the test pile(s) shall be checked and any deviation from the normal execution conditions shall be accounted for.~~

D: Comment: not clear how to handle this.

(12) The characteristic compressive resistance of the ground, $R_{c;k}$, may be derived from the characteristic values of the base resistance, $R_{b;k}$, and of the shaft resistance, $R_{s;k}$, such that:

$$R_{c;k} = R_{b;k} + R_{s;k} \tag{7.3}$$

(13) These components may be derived directly from static load test results, or estimated on the basis of ground test results or dynamic load tests.

(14)P The design resistance, $R_{c;d}$, shall be derived from either:

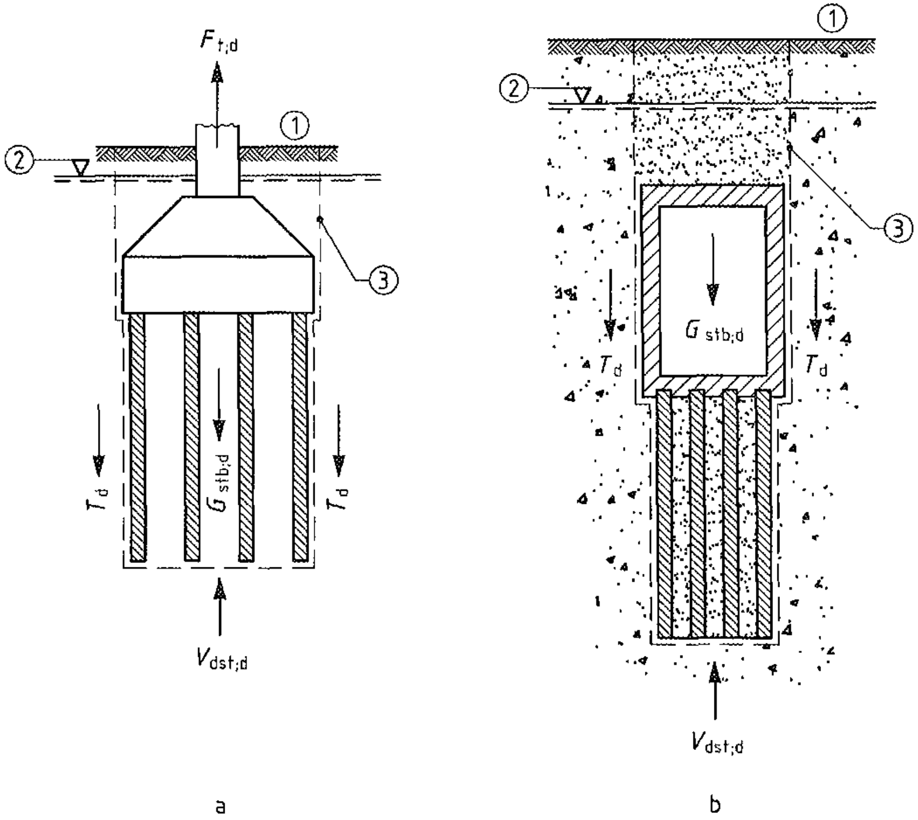
$$R_{c;d} = R_{c;k}/\gamma_t \tag{7.4}$$

or

$$R_{c;d} = R_{b;k}/\gamma_b + R_{s;k}/\gamma_s \tag{7.5}$$

NOTE The values of the partial factors may be set by the National annex. The recommended values for persistent and transient situations

are given in Tables A.6, A.7 and A.8	
[7.6.3.2 Ultimate tensile resistance from pile load tests] (1)P Pile load tests to determine the ultimate tensile resistance of an isolated pile, R_{t}, shall be carried out in accordance with 7.5.1, 7.5.2 and 7.5.4, and with regard to 7.6.2.2. (2)P The design tensile resistance, $R_{t;d}$, shall be derived from: $R_{t;d} = R_{t;k}/\gamma_{s;t}$ (7.13) NOTE The values of the partial factors may be set by the National annex. The recommended values for persistent and transient situations are given in Tables A.6, A.7 and A.8.	



1 ground surface

2 ground-water level

3 side of the ‘block’, where resistance T_d develops

Figure 7.1 — Examples of uplift (UPL) of a group of piles

~~(3) Normally when piles are to be loaded in tension, it should be specified that more than one pile should be tested. In the case of a large number of tension piles, at least 2 % should be tested.~~ *D: Comment: not precise enough*. A (3a) For micropiles with tensile loading in accordance with EN 14199 static pile load tests have to be performed – at at least 3% of the planned number of piles – not less than n=2 piles have to be tested. For very closely spaced piles, the pile group has to be proven in accordance with 8.7 A (9) A (3b) Pile load tests have to be performed in accordance with 7.5.2.1 A(5). A (3c) For grouted pile systems with tensile load (grouted micropiles in accordance with EN 14199 and grouted displacement piles in accordance with EN 12699) the design value of the pile resistance $R_{t,d\ l}$ is the characteristic tensile resistance $R_{t,k}$, derived from pile load tests, divided by the partial factor $\gamma_{s,t}$, which is multiplied by the model factor η_M. So that equation A (7.13) applies: $R_{t,d} = R_{t,k} / (\gamma_{s,t} \cdot \eta_M)$ The model factor is independent of the pile inclination ξ = 1,25. r ~~(4)P The records of the installation of the test pile(s) shall be checked and any deviation from the normal construction conditions shall be accounted for in the interpretation of the pile load test results.~~ *D: Comment: it was already mentioned that piles should be*	

representative. (5)P The characteristic value of the pile tensile resistance shall be determined by: $$R_{t;k} = \mathrm{Min}\left\{\frac{(R_{t;m})_{mean}}{\xi_1};\frac{(R_{t;m})_{min}}{\xi_2}\right\} \quad (7.14)$$ where ξ_1 and ξ_2 are correlation factors related to the number of piles tested, n, and are applied respectively to the mean $(R_{t;m})_{mean}$ and the lowest $(R_{t;m})_{min}$ value of the measured tensile resistances. NOTE The values of the correlation factors may be set by the National annex. The recommended values are given in Table A.9.	
4.2.2.2 Ultimate resistance from dynamic impact tests	
[7.5.3 Dynamic load tests] (1) Dynamic load tests[1] may be used to estimate the compressive resistance provided ~~an adequate site investigation has been carried out and~~ the method has been calibrated against static load tests on the same type of pile, of similar length and cross-section, and in comparable soil conditions, (see 7.6.2.4 to 7.6.2.6). *[FIN: Dynamic load at least with Nordic end-bearing piles is a reliable method to verify pile capacity and you don't need to have calibration static test, there's a comparable experience (many decades) in Nordic countries.* *Correlation factors are suitable compared to static load test when there*	

1 See: ASTM Designation D 4945, Standard Test Method for High-Strain Dynamic Testing of Piles.

are few load tests (first three columns, correlation factors are 7…24% bigger compared to static load test) but when there are numerous load tests (columns 4-5) the difference may be too big (25-40% bigger)]. ~~(2)P If more than one type of dynamic test is used, the results of different types of dynamic test shall always be considered in relation to each other.~~ ~~(3) Dynamic load tests may also be used as an indicator of the consistency of the piles and to detect weak piles.~~ *D: Comment: Both clauses like a textbook.*	
[7.6.2.4 Ultimate compressive resistance from dynamic impact tests] *NL: Add quasi-static test methods in this paragraph or introduce a new paragraph dedicated to quasi-static test method. If needed we are happy to draft such paragraph* ~~(1)P Where a dynamic impact (hammer blow) pile test [measurement of strain and acceleration versus time during the impact event (see 7.5.3(1))] is used to assess the resistance of individual compression piles, the validity of the result shall have been demonstrated by previous evidence of acceptable performance in static load tests on the same pile type of similar length and cross-section and in similar ground conditions.~~ *D: Comment: is already mentioned in 7.4.1.* ~~(2) When using a dynamic impact load test, the driving resistance of the pile should be measured directly on the site in question.~~	

D: Comment: Where else?

~~NOTE A load test of this type can also include a process of signal matching to measured stress wave figures. Signal matching enables an approximate evaluation of shaft and base resistance of the pile as well as a simulation of its load-settlement behaviour.~~

D: Comment: superfluous.

~~(3)P The impact energy shall be high enough to allow for an appropriate interpretation of the pile capacity at a correspondingly high enough strain level.~~

D: Comment: imprecise and also trivial.

(4)P The design value of the compressive resistance of the pile, $R_{c;d}$ shall be derived from:

$$R_{c;d} = R_{c;k}/\gamma_t \tag{7.10}$$

whereas the characteristic pile resistance $R_{c,k}$ has to be determined from test data $R_{c,m}$ of dynamic load tests, i.e. from the resistances derived from measuring signals, as follows:

$$R_{c;k} = \mathrm{Min}\left\{\frac{(R_{c;m})_{mean}}{\xi_5};\frac{(R_{c;m})_{min}}{\xi_6}\right\} \tag{7.11}$$

where ξ_5 and ξ_6 are correlation factors related to the number of piles tested, n, and are applied to the mean $(R_{c;m})_{mean}$ and the lowest $(R_{c;m})_{min}$ value of $R_{c;m}$ respectively.

NOTE The values of the partial factor and correlation factors may be set by the National annex. The recommended values are given in Table

A.11.	

A.11.

Comment: Potential for harmonization to be discussed.

German recommendation:

The values of the correlation factors for deriving the characteristic pile resistances from dynamic pile load tests in accordance with equation (7.11) are given in table A 7.2

The fundamental values of the correlation factors $\xi_{,i}$ refer to dynamic pile load tests, which are evaluated by the direct method and calibrated to static pile load tests on the same construction area. In all other cases the correlation factors ξ_i from $\xi_{0,i}$ have to be determined in accordance with a to e of table A 7.2 and picture A 7.1.

Table A 7.2: Fundamental values $\xi_{0,i}$ with corresponding increase factors and model factors for correlation factors ξ_5 and ξ_6 for the determination of characteristic values from impact tests or dynamic pile load tests.

$\xi_{0,i}$ **für** n **=**	**≥ 2**	**≥ 5**	**≥ 10**	**≥ 15**	**≥ 20**
$\xi_{0,5}$	1,60	1,50	1,45	1,42	1,40
$\xi_{0,6}$	1,50	1,35	1,30	1,25	1,25
n is the number of test loading piles					

a. Calculation of the correlation factors ξ_i as: $\xi_i = (\xi_{0,i} + \Delta\xi) \cdot \eta_D$, see also picture A 7.1.

b. For the increase factor $\Delta\xi$ is used:

– $\Delta\xi$ = 0 for the calibration of dynamic evaluation processes on static pile load test results on the same construction area;

– $\Delta\xi$ = 0,10 for the calibration of dynamic evaluation processes on static pile load test results for a comparable building measure;
– $\Delta\xi$ = 0,40 for the calibration of dynamic evaluation processes due to provable or general experiences for pile resistances. The application of direct methods, e.g. Case- or TNO-method is not permitted

c. For the model factor η_D to take account of the evaluation process it should be used:

- η_D = 1,00 for direct evaluation processes;
- η_D = 0,85 for extended process with complete modeling

d. If structures have sufficient stiffness and strength to rearrange loads from “soft” to “stiff” piles, it is allowed to divide the values of ξ_5 und ξ_6 by 1,10.

e. If different piles are present in the foundation, for the choice of the number of test piles n groups of similar piles should be considered separately. This also applies for areas with similar soil conditions within a construction area.

Figure A 7.1: Diagram for the proceeding to evaluate the correlation factors ξ_5 and ξ_6 according to table A 7.2

A (5) For piles in cohesive soils, which are not fully saturated, the results of dynamic pile load tests have to be calibrated by static pile

load tests on the same construction area. A (6) In saturated cohesive soils, pore water pressures can increase the bearing capacity measured by a dynamic pile load test. Therefore dynamic pile load tests must not to be used to determine the characteristic pile bearing capacity when the pile toe or the significant skin friction is in such soils. A (7) If both static and dynamic pile load tests are carried out on a construction area, two cases have to be distinguished: – If after the application of the correlation factors the characteristic pile resistances, solely from the analysis of the static pile load tests, is greater than those from the analysis of the dynamic pile load tests, only the results of static pile load tests may be used. – If after application of the correlation factors the bearing capacity, solely from the dynamic load tests greater, it is only allowed to increase the characteristic pile resistance compared with the results of the static pile load tests if it can to be comprehensible reasoned. Expertise and experiences in geotechnical engineering is required.	
4.2.2.3 Ultimate resistance by applying pile driving formulae	
[7.6.2.5 Ultimate compressive resistance by applying pile driving formulae] *[FIN: Pile driving formulas are still used and are suitable with end-bearing piles. Correlation factors (+10 % to dynamic load tests) are on the right magnitude. → Keep the clauses. NDP. Dynamic pile test can be a “calibration” test”]*	

~~(1)P Pile driving formulae shall only be used if the stratification of the ground has been determined.~~ ~~(2)P If pile driving formulae are used to assess the ultimate compressive resistance of individual piles in a foundation, the validity of the formulae shall have been demonstrated by previous experimental evidence of acceptable performance in static load tests on the same type of pile, of similar length and cross-section, and in similar ground conditions.~~ ~~(3)P For end-bearing piles driven into non-cohesive soil, the design value of the compressive resistance,~~ $\cancel{R_{c;d}}$, ~~shall be assessed by the same procedure as in 7.6.2.4.~~ ~~(4) When a pile driving formula is applied to verify the compression resistance of a pile, the pile driving test should have been carried out on at least 5 piles distributed at sufficient spacing in the piling area in order to check a suitable blow count for the final series of blows.~~ ~~(5) The penetration of the pile point for the final series of blows should be recorded for each pile.~~ *Comment: From German view driving formulae should not applied as they have proven to be not sufficiently reliable..*	
4.2.2.4 Ultimate resistance from wave equation analysis	
[7.6.2.6 Ultimate compressive resistance from wave equation analysis] *[FIN: Wave equations are used and are suitable in most cases. In most cases wave equations analysis results are on the “safe side” and together with +5% addition to dynamic load test correlation factors, the correlation factors are on the right magnitude. → Keep the clauses.*	

NDP. Dynamic pile test can be a “calibration test” ~~(1)P Wave equation analysis shall only be used where stratification of the ground has been determined by borings and field tests.~~ ~~(2)P Where wave equation analysis is used to assess the resistance of individual compression piles, the validity of the analysis shall have been demonstrated by previous evidence of acceptable performance in static load tests on the same pile type, of similar length and cross-section, and in similar ground conditions.~~ ~~(3)P The design value of the compressive resistance,~~ $\cancel{R_{c;d}}$~~, derived from the results of wave equation analysis of a number of representative piles, shall be assessed by the same procedure as in 7.6.2.4, using~~ $\cancel{\xi}$ ~~values based on local experience.~~ ~~NOTE Wave equation analysis is based on a mathematical model of soil, pile and driving equipment without stress wave measurements on site. The method is usually applied to study hammer performance, dynamic soil parameters and stresses in the pile during driving. It is also, on the basis of the models, possible to determine the required driving resistance (blow count) that is usually related to the expected compressive resistance of the pile.~~ *Comment: From German view wave equation analysis not enough verified for evaluation pile resistiances.*	
4.2.2.5 Re-driving	
[7.6.2.7 Re-driving] ~~(1)P In the design, the number of piles to be re-driven shall be specified.~~	

~~If re-driving gives lower results, these shall be used as the basis for ultimate compressive resistance assessment. If re-driving gives higher results, these may be considered.~~ ~~(2) Re-driving should usually be carried out in silty soils, unless local comparable experience has shown it to be unnecessary.~~ ~~NOTE Re-driving of friction piles in clayey soils normally results in reduced compressive resistance.~~ *D: Comment: This should not be part of normative text.* **A 7.6.2.8 Piled raft foundation** A(1) For the proof of a piled raft foundation at limit state GEO-2 the design value of the total resistance has to be determined with $R_{c,tot,d} = R_{c,tot,k} / \gamma_{R,v}$ *A 7.11* The partial factor $\gamma_{R,v}$ has to be applied in accordance with A 2.4.7.6.3, Table A 2.3. The used calculation method for determine the total resistance $R_{c,tot,k}$ has to take into account the interaction effects between soil, raft and piles. It is not necessary to proof the elements raft and single pile as individual elements at the limit state GEO-2.	
4.2.3 Transversely loaded piles	
[7.7.2 Transverse load resistance from pile load tests] ~~(1)P Transverse pile load tests shall be carried out in accordance with 7.5.2.~~ A (1) The characteristic lateral resistance of a single pile should be	

determined on the basis of pile load tests or experience with similar pile load tests. The pile load tests have to be performed up to the design value of the expected interaction. In the case of piles fixed at the top, the fixation need not be imitated with pile load tests. ~~(2) Contrary to the load test procedure described in 7.5, tests on transversely loaded piles need not normally be continued to a state of failure. The magnitude and line of action of the test load should simulate the design loading of the pile.~~ *D: Comment: see A(1).* (3)P An allowance shall be made for the variability of the ground, particularly over the top few metres of the pile, when choosing the number of piles for testing and when deriving the design transverse resistance from load test results. ~~(4) Records of the installation of the test pile(s) should be checked, and any deviation from the normal construction conditions should be accounted for in the interpretation of the pile load test results. For pile groups, the effects of interaction and head fixity should be accounted for when deriving the transverse resistance from the results of load tests on individual test piles.~~ *D: Comment: self-evident that pile load test should be representative*	
4.2.4 Pile resistance due to cyclic, dynamic and impact loads	
5 Ultimate limit state design	
5.1 General	

5.1.1 Limit state design

[7.6.1.1 Limit state design]

~~(1)P The design shall demonstrate that exceeding the following limit states is sufficiently improbable:~~

- ~~ultimate limit states of compressive or tensile resistance failure of a single pile;~~
- ~~ultimate limit states of compressive or tensile resistance failure of the pile foundation as a whole;~~
- ~~ultimate limit states of collapse or severe damage to a supported structure caused by excessive displacement or differential displacements of the pile foundation;~~
- ~~serviceability limit states in the supported structure caused by displacement of the piles.~~

D: Comment: That is all stated on 7.6.2 and section 2 already.

(1) As a basis for the limit-state-design the axial resistance of single piles has to be described by a resistance-settlement-curve. The resistance-settlement-(or heaving) curve has to be determined by the means of pile load tests or experiences with comparable pile load tests. Thereby creeping with constant load has to be considered.

*[*NL: I agree if the intention is to facilitate the following; to check in ULS two conditions:
1. the design load < the design resistance (this resistance is ultimate resistance, independent of the settlement
2. settlement of the pile in ULS < limiting values for the superstructure in ULS
For SLS the check is:
1. settlement of the pile in SLS < limiting values for the superstructure in SLS*]*

(2) Normally the design should consider the margin of safety with respect to compressive or tensile resistance failure, which is the state in which the pile foundation displaces significantly downwards or upwards with negligible increase or decrease of resistance (see 7.6.2 and 7.6.3). *D: Comment: Clause (2) not clear. Should be modified.* (3) For piles in compression it is often difficult to define an ultimate limit state from a load settlement plot showing a continuous curvature. In these cases, settlement of the pile top equal to 10% of the pile base diameter should be adopted as the "failure" criterion. ~~(4)P For piles that undergo significant settlements, ultimate limit states may occur in supported structures before the resistance of the piles is fully mobilised. In these cases a cautious estimate of the possible range of the settlements shall be adopted in design.~~ ~~NOTE Settlement of piles is considered in 7.6.4~~ *D: Comment: Clause (4) is relevant for all kind of foundations and should therefore be stated in Section 2.*	
5.1.2 Overall Stability	
[7.6.1.2 Overall stability] ~~(1)P Failure due to loss of overall stability of foundations involving piles in compression shall be considered in accordance with Section 11.~~ ~~(2) Where there is a possibility of instability, failure surfaces both passing~~	

~~below the piles and intersecting the piles should be considered.~~ (3)P For pile groups with tension loading the safety against F~~ailure due to~~ uplift of the soil block inside the pilegroup ~~containing piles~~ shall be checked in accordance with 7.6.3.1(4)P.	
5.2 Single standing piles	
5.2.1 Axially loaded piles	
Comment: The structure of this section should be discussed. Aim: no distinction between compression and tension, but instead of that a distinction between single pile and pile group. **[7.6.2 Compressive ground resistance]** **[7.6.2.1 General]** (1)P To demonstrate that the pile foundation will support the design load with adequate safety against compressive failure, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations: $F_{c;d} \leq R_{c;d}$ (7.1) ~~(2) In principle~~ $\cancel{F_{c;d}}$ ~~should include the weight of the pile itself and~~ $\cancel{R_{c;d}}$ ~~should include the overburden pressure of the soil at the foundation base. However these two items may be disregarded if they cancel approximately. They need not cancel if:~~ For compression piles the weight of the piles themselves can be neglected; for tension piles the weight of the piles can be considered as an additional resistance.	

~~— downdrag is significant;~~ ~~— downdrag is significant;~~ ~~— the soil is very light,~~ ~~— the pile extends above the surface of the ground.~~ *D: Comment: That´s like a textbook and self-evident.* (3)P For piles in groups the pile-group effect (pile-pile-interaction) has to be considered in determining the pile-resistance. Alternatively two failure mechanisms shall be taken into account both in ultimate and the serviceability limit state design: — ~~compressive resistance failure of~~ the piles individually; — ~~compressive resistance~~ failure of the piles in ~~groups~~ ~~and the soil contained between them acting as a block.~~ ~~The design resistance shall be taken as the lower value caused by these two mechanisms.~~ ~~(4) The compressive resistance of the pile group acting as a block may be calculated by treating the block as a single pile of large diameter.~~ ~~(5)P The stiffness and strength of the structure connecting the piles in the group shall be considered when deriving the design resistance of the foundation.~~ *D: Comment: The proof of the structure and material should be regulated in Section 2..* (6) If the piles support a stiff structure, advantage may be taken of the ability of the structure to redistribute load between the piles. A limit state will occur only if a significant number of piles fail together; therefore a failure mode involving only one pile need not be considered.	

~~(7) If the piles support a flexible structure, it should be assumed that the compressive resistance of the weakest pile governs the occurrence of a limit state.~~ ~~(8) Special attention should be given to possible failure of edge piles caused by inclined or eccentric loads from the supported structure.~~ *D: Comment: That´s like a textbook and (7) is already stated in (6).* ~~(9)P If the layer in which the piles bear overlies a layer of weak soil, the effect of the weak layer on the compressive resistance of the foundation shall be considered.~~ *D: Comment: That´s already stated in (11).* ~~(10)P The strength of a zone of ground above and below the pile base shall be taken into account when calculating the pile base resistance.~~ ~~NOTE This zone may extend several diameters above and below the pile base. Any weak ground in this zone has a relatively large influence on the base resistance.~~ *D: Comment: Provides no additional information.* (11) Punching failure should be considered if weak ground is present at a depth of less than 4 times the base diameter below the base of the pile. ~~(12)P Where the pile base diameter exceeds the shaft diameter, the possible adverse effect shall be considered.~~ *D: Comment: Regulation does not support.* ~~(13) For open-ended driven tube or box-section piles with openings of more than 500 mm in any direction, and without special devices inside the pile to induce plugging, the base resistance should be limited to the~~	

~~smaller of:~~ ~~— the shearing resistance between the soil plug and the inside face of the pile;~~ ~~— the base resistance derived using the cross-sectional area of the base.~~ *D: Comment: No rules for specific pile types in EC7-1.*	
[7.6.3 Ground tensile resistance] **[7.6.3.1 General]** (1)P The design of piles in tension shall be consistent with the design rules given in 7.6.2, where applicable. Design rules that are specific for foundations involving piles in tension are presented below. (2)P To verify that the foundation will support the design load with adequate safety against a failure in tension, the following inequality shall be satisfied for all ultimate limit state load cases and load combinations: $F_{t;d} \leq R_{t;d}$ (7.12) A (2) If for the determination of the design values of tensile stress a characteristic compressive load due to favorable permanent stresses is effective at the same time, the characteristic compressive load has to be accounted with the partial factor $\gamma_{G,inf}$ in accordance with Table A 2.1 (3)P For tension piles, two failure mechanisms shall be considered: — pull-out of the piles from the ground mass;	

— uplift of the block of ground containing the piles. A(3) The design value for the tension load $F_{t,d}$ which is needed to prove the safety against pull-out is given by equation A(7.12a) $F_{t,d} = F_{t,G,k} \cdot \gamma_G + F_{t,Q,rep} \cdot \gamma_Q - F_{c,G,k} \cdot \gamma_{G,inf}$ A 7.12a with: $F_{t,G,k}$ the characteristic value of tensile stress of a pile or a group of piles due to permanent actions; γ_G the partial factor for permanent loads in the limit state GEO-2 in accordance with Table A 2.1; $F_{t,Q,rep}$ the characteristic or representative value of tensile stress of a pile or a group of piles due to unfavorable variable interactions; γ_Q the partial factor for unfavorable variable loads in the limit state GEO-2 in accordance with Table A 2.1; $F_{c,G,k}$ the characteristic value for simultaneously acting compressive stress due to permanent interactions; $\gamma_{G,inf}$ the partial factor $\gamma_{G,inf}$ for favorable permanent compressive loads in the limit state GEO-2 in accordance with Table A 2.1. ~~(4)P Verification against uplift failure of the block of ground containing the piles (see Figure 7.1), shall be carried out in accordance with 2.4.7.4.~~ A (4a) To achieve a sufficient safety against uplift of a foundation body or a structure anchored with tensile piles, the following condition has to be proven for the limit state UPL:	

$$G_{\text{dst,k}} \cdot \gamma_{\text{G,dst}} + Q_{\text{dst,rep}} \cdot \gamma_{\text{Q,dst}} \leq G_{\text{stb,k}} \cdot \gamma_{\text{G,stb}} + G_{\text{E,k}} \cdot \gamma_{\text{G,stb}}$$

with

$G_{dst,k}$ the characteristic value of permanent vertical destabilizing loads;

$\gamma_{G,dst}$ the partial factor for destabilizing permanent load in the limit state UPL in accordance with Table A 2.1;

$Q_{dst,rep}$ the characteristic or representive value of variable, destabilizing vertical loads;

$\gamma_{Q,dst}$ the partial factor for destabilizing variable loads in the limit state UPL in accordance with Table A 2.1;

$G_{stb,k}$ the lower characteristic value of stabilizing permanent, vertical loads of the structure;

$\gamma_{G,stb}$ the partial factor of stabilizing permanent loads in the limit state UPL in accordance with Table A 2.1;

$G_{E,k}$ the characteristic weight of soil which is attached to a group of tension piles .

A(4b) The weight $G_{e,k}$ can be calculated according to the approach

$$G_{\text{E,k}} = n_{\text{Z}} \left[l_{\text{a}} \cdot l_{\text{b}} \left(L - \frac{1}{3} \cdot \sqrt{l_{\text{a}}^2 + l_{\text{b}}^2} \cdot \cot \varphi \right) \right] \cdot \eta_{\text{Z}} \cdot \gamma$$

Thereby is in addition to the already defined values:

L length of tension piles

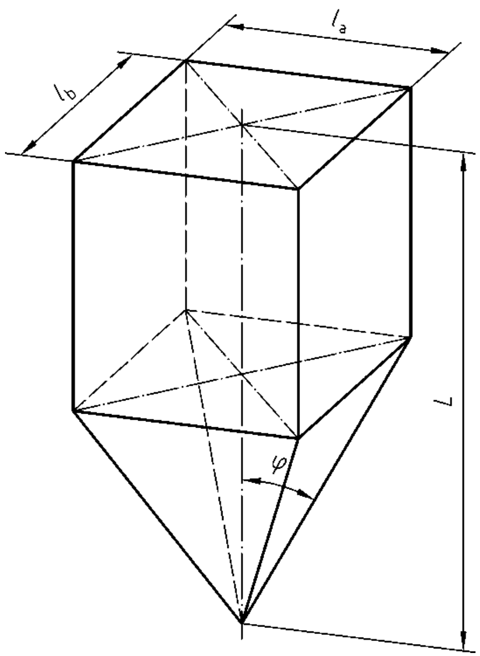
l_a the bigger grid dimension of a group of piles;

l_b the smaller grid dimension of a group of piles;

n_Z the number of tension piles;

γ the unit weight of the attached soil;

η_Z adjustment factor, η_Z = 0,80.

The associated geometric model is shown in picture A 7.2. It also applies for border piles. If necessary, the unit weight γ has to be replaced all or partly by the soil weight below the ground water level γ'. *[FIN: Ground tensile resistance A(4b): German approach need to be evaluated]* ***F:*** *The calculation method of $G_{E;k}$ does not seem very useful. Other approaches can be performed.*  **Figure A 7.2:** Geometry of the attached soil block to a single-pile in a group of piles	

~~(5) For isolated tensile piles or a group of tensile piles, the failure mechanism may be governed by the pull-out resistance of a cone of ground, especially for piles with an enlarged base or rock socket.~~ *D: Comment: This should not be part of normative text.* ~~(6) When considering the uplift of the block of ground containing the piles the shear resistance T_d along the sides of the block may be added to the resisting forces shown in figure 7.1.~~ A (6) Both for the proof of the limit state against pull-out of the piles in accordance to (1) and (2) as well as for the proof of the limit state against uplift in accordance with (4), the involvement of shear forces T_K in accordance with A 10.2.2 may be considered: a) For the proof of the limit state against pull-out of the piles (limit state GEO-2) the shear forces T_k have to be treated in line with the approach $F_{t,d} = F_{t,G,k} \cdot \gamma_G + F_{t,Q,rep} \cdot \gamma_Q - (F_{c,G,k} + T_k) \cdot \gamma_{G,inf}$ as permanent favourable compressive loads for determining the design value of tension load. b) For the proof of the limit state against uplift (limit state UPL) the shearing forces T_k have to be treated in line with the approach $G_{dst,k} \cdot \gamma_{G,dst} + Q_{dst,rep} \cdot \gamma_{Q,dst} \leq G_{stb,k} \cdot \gamma_{G,stb} + (G_{E,k} + T_k) \cdot \gamma_{G,stb}$ as permanent stabilizing loads. Thereby is in addition to the already defined values: T_k the characteristic value of the shear resistance or frictional resistance, which is developed around a block of soil in what a	

group of tension piles is acting or in a joint between soil and structure. ~~(7) Normally the block effect will govern the design tensile resistance if the distance between the piles is equal to or less than the square root of the product of the pile diameter and the pile penetration into the main resisting stratum.~~ *D: Comment: What does ´normally´ means?.* ~~(8)P The group effect, which may reduce the effective vertical stresses in the soil and hence the shaft resistances of individual piles in the group, shall be considered when assessing the tensile resistance of a group of piles.~~ *D:Comment: Already stated in A(4a)* ~~(9)P The severe adverse effect of cyclic loading and reversals of load on the tensile resistance shall be considered.~~ *D:Comment: Already stated in 7.4.1* (~~10) Comparable experience based on pile load tests should be applied to appraise this effect.~~ ***D:**Comment: not normative*	
5.2.2 Transversely loaded piles	
[7.7.1 General] (1)P The design of piles subjected to transverse loading shall be	

consistent with the design rules given in 7.4 and 7.5, where applicable. Design rules specifically for foundations involving piles subjected to transverse loading are presented below. A (1) Lateral resistance may be applied only for piles with a pile shaft diameter $D_s \geq$ 0.30 m or an edge length as $a_s \geq$ 0.30 m. The characteristic lateral resistance may be described by the characteristic values $k_{s,k}$ of the subgrade modulus, which are to be determined from pile load test results. The subgrade moduli of the involved soil layers may be recognized also simplified on the basis of soil characteristics in accordance with 7.7.3 A (3), if they only serve to determine the internal forces. ***F:*** *Micropiles can be submitted to transversal action and have a diameter Ds<0.3 m.* *Subgrade reaction modulus can be determined by correlations validated by load tests.* (2)P To demonstrate that a pile will support the design transverse load with adequate safety against failure, the following inequality shall be satisfied ~~for all ultimate limit state load cases and load combinations:~~ for all load cases and load combinations in the ultimate limit state (GEO-2): $F_{tr;d} \leq R_{tr;d}$ (7.19) As design value $R_{tr,d}$ the spatial earth resistance $E^r_{ph,d}$ can be applied for the corresponding part of the embedment depth to the pivot point (shift-zero-point). A (2) For a pile with loading transvers to the axis for the proof of the loading capacity STR, the internal forces in the pile related to the	

characteristic soil reactions (subgrade) have to be determined with characteristic or representative stresses, and only then to be converted into design values. ~~(3) One of the following failure mechanisms should be considered:~~ ~~– for short piles, rotation or translation as a rigid body;~~ ~~– for long slender piles, bending failure of the pile, accompanied by local yielding and displacement of the soil near the top of the pile.~~ A (3a) The proof of the capacity of flexible long, slender piles for the limit states STR and GEO does not need to be performed when the piles are completely embedded in the ground and the horizontal characteristic stress achieves for BS-P at most 3% and for BS-T at most 5% of the vertical stress. In all other cases, proceed as follows: – Definition of the starting values for the determination of soil reactions, e.g. in the form of subgrade modulus in accordance with 7.7.3 A (3). The necessary subgrade moduli have to be determined for the level of the characteristic loads; – Determination of the characteristic internal forces or the characteristic stresses with the characteristic values of actions and with the previously determined subgrade moduli; – Conversion of the characteristic internal forces and characteristic stresses in design values of the interactions by multiplying by the partial factors for interactions in accordance with Table A 2.1; – Simplified proof, that the characteristic normal stresses $\sigma_{h,k}$, between pile and soil does not exceed, the characteristic earth resistance stresses $e_{ph,k}$ which may be calculated simplified for the two-dimensional case;	

– Proof, that the design value of the lateral soil resistance force has been applied not larger than allowed by the design value of the spatial earth resistance force for the corresponding part of the embedment depth to cross force zero point; – Proof of safety against material failure in accordance 7.8. A NOTE for A (3a) The above method provides at first a computational simplification, which can be performed shift independently and basically refers to a simplified determination of subgrade modulus in accordance with 7.7.3 A (3) and equation A (7:20). For a closer investigation of the subgrade modulus of pile load tests, the subgrade modulus should be determined shift dependently and should be applied in accordance with the shifts from the interaction of the entire construction. (4)P The group effect shall be considered when assessing the resistance of transversely loaded piles. ~~(5) It should be considered that a transverse load applied to a group of piles may result in a combination of compression, tension and transverse forces in the individual piles.~~ *D:Comment: basic static.*	
5.3 Pile Groups	
5.3.1 Axially loaded piles	
[New, to be elaborated]	
5.3.2 Transversely loaded piles	

[New, to be elaborated]	
5.4 Piled Rafts	
[New, to be elaborated]	
6 Serviceability limit state design	
6.1 General	
[7.6.4.1 General] ~~(1)P Vertical displacements under serviceability limit state conditions shall be assessed and checked against the requirements given in 2.4.8 and 2.4.9.~~ *D:Comment: self-evident.* (2) When calculating the vertical displacements of a pile foundation, the uncertainties involved in the calculation model and in determining the relevant ground properties should be taken into account. Hence it should not be overlooked that in most cases calculations will provide only an approximate estimate of the displacements of the pile foundation. NOTE For piles bearing in medium-to-dense soils and for tension piles, ~~the safety requirements for the ultimate limit state design are normally sufficient to prevent~~ a proof of serviceability limit state might be unnecessary. ~~in the supported structure.~~ *D:Comment: should be discussed.* A (2) Basis of the determination of the characteristic vertical displacements of pile foundations should be the analysis of pile load	

tests]. Mathematical models should be used only in exceptional cases, when these have been calibrated based on pile load tests.	
[7.6.4.2 Pile foundations in compression] (1)P The occurrence of a serviceability or ultimate limit state in the supported structure due to pile settlements shall be checked, taking into account downdrag, where probable. ~~NOTE When the pile toe is placed in a medium-dense or firm layer overlying rock or very hard soil, the partial safety factors for ultimate limit state conditions are normally sufficient to satisfy serviceability limit state conditions.~~ *D: Comment: not normative.* (2)P Assessment of settlements shall include both the settlement of individual piles and the settlement due to group action. (3) The settlement analysis have to ~~should~~ include an estimate of the differential settlements that may occur. (4) When no load test results are available for an analysis of the interaction of the piled foundation with the superstructure, the load-settlement performance of individual piles ~~should~~ may be assessed on empirically established safe assumptions.	
[7.6.4.3 Pile foundations in tension] (1)P The assessment of upward displacements shall be in accordance with the principles of 7.6.4.2. ~~NOTE Particular attention should be paid to the elongation of the pile~~	

~~material.~~ *D: Comment: not normative.* ~~(2)P When very severe criteria are set for the serviceability limit state, a separate check of the upward displacements shall be carried out.~~	
6.2 Single standing piles	
6.2.1 Axially loaded piles	
[New]	
6.2.2 Transversely loaded piles	
7.7.4 Transverse displacement (1)P The assessment of the transverse displacement of a pile foundation shall take into account: — the stiffness of the ground and its variation with strain level; — the flexural stiffness of the individual piles; — the moment fixity of the piles at the connection with the structure; — the group effect; — the effect of load reversals or of cyclic loading. ~~(2) A general analysis of the displacement of a pile foundation should be based on expected degrees of kinematic freedom of movement.~~	

6.3 Pile Groups	
6.3.1 Axially loaded piles	
[New, to be elaborated]	
6.3.2 Transversely loaded piles	
[New, to be elaborated]	
6.4 Piled Rafts	
[New, to be elaborated]	
7 Testing and Instrumentation	
7.1 General	
[New, to be elaborated]	
7.2 Static load tests	
7.2.1 Loading procedure	

[7.5.2.1 Loading procedure] (1)P The pile load test procedure[2], particularly with respect to the number of loading steps, the duration of these steps and the application of load cycles, shall be such that conclusions can be drawn about the deformation behaviour, creep and rebound of a piled foundation from the measurements on the pile. For trial piles, the loading shall be such that conclusions can also be drawn about the ultimate failure load. ~~(2) Devices for the determination of loads, stresses or strains and displacements should be calibrated prior to the test.~~ ~~(3) The direction of the test load applied to compression or tensile piles should coincide with the longitudinal axis of the pile.~~ *D:Comment: That´s basic knowledge of monitoring.* **(4) Pile load tests for the purpose of designing a tensile pile foundation should be carried out to failure. Extrapolation of the load-displacement graph for tension tests must not be used.**	
7.2.2 Trial piles	
[7.5.2.2 Trial piles] ~~(1)P The number of trial piles required to verify the design shall depend on the following:~~ ~~– the ground conditions and their variability across the site;~~ ~~– the Geotechnical Category of the structure, if appropriate;~~ ~~– previous documented evidence of the performance of the same type of pile in similar ground conditions;~~	

2 See: ISSMFE Subcommittee on Field and Laboratory Testing, Axial Pile Loading Test, Suggested Method. ASTM Journal, June 1985, pp. 79-90.

~~– the total number and types of pile in the foundation design.~~ *Comment: D: That´s already stated in 7.5.1.;* *NL:* In 7.5.1. only the soil conditions are addressed not the other aspects. ~~(2)P The ground conditions at the test site shall be investigated thoroughly. The depth of borings or field tests shall be sufficient to ascertain the nature of the ground both around and beneath the pile tip. All strata likely to contribute significantly to pile behaviour shall be investigated.~~ *D: Comment: That´s subject of EC 7-2.* ~~(3)P The method used for the installation of the trial piles shall be fully documented in accordance with 7.9.~~ *D: Comment: That´s already stated in 7.9.* (1) For grouted micropiles in accordance with EN 12699 or EN 14199 static pile load tests have to be performed – at ~~at~~ least 3% of the planned number of piles. – not less than n = 2 piles have to be tested. ***PL:*** *Why it is in 'Trial piles'? Shall always trial micropiles be constructed and tested? Such statement should be in 7.5.1.* *And it is not consequent: why we always require ('have to be'!) tests on micropiles (trial or not), whereas we are much less strict regarding 'normal' piles. Are micropiles particularly danger?* *These rules are in EN 12699 and it is duplicating them here. There are no similar rules in EC7-1 for any other piles…*	

7.2.3 Working piles	
[7.5.2.3 Working piles] ~~(1)P It shall be specified that the number of working pile load tests shall be selected on the basis of the recorded findings during installation.~~ *Comment: What does it mean?* (2)P The test load applied to working piles shall be at least equal to the design load for the foundation. A(3) If test piles should be used as working piles it has to be proven that their deformation behaviour modified due to the pile load test correlates with the requirements of the construction and that they have not suffered any loss of resistance under the test load. ***PL:*** *It is not clear. A pile after being tested in compression is almost always stiffer then not loaded. But is it any danger, that 'it has to be proven…' etc? This is more applicable to piles tested in tension.* *[FIN: Agree with PL Comment]*	
7.3 Dynamic load tests	
[New, to be elaborated]	
7.4 Load test report	
[7.5.4 Load test report] (1)P ~~It shall be specified that a factual report shall be written for all load~~	

~~tests.~~ Where appropriate, this report shall include: — a description of the site; — the ground conditions with reference to ground investigations; — the pile type; — description of the pile installation and of any problems encountered during the works; — a description of the loading and measuring apparatus and the reaction system; — calibration documents for the load cells, the jacks and the gauges; — the installation records of the test piles; — photographic records of the pile and the test site; — test results in numerical form; — time-displacement plots for each applied load when a step loading procedure is used; — the measured load-displacement behaviour; — reasons for any departures from the above requirements .	
8 Structural design	
[existing Part 1, § 7.8] **Take reference to EC 3, Part 5!** *[FIN: Structural design is still in too common level. → Structural design is in general level only. More requirements to how to handle buckling*	

(Nordic approach: initial curvature, pile joint's effect and requirements, second order moment), how to use structural models (superstructure, piles, soil) in structural design of piles, structural capacity during installation. There are pile specific structural design situations. Close co-operation with EN1992, EN1993 and EN1994 where specific structural design requirements for pile should be added.] ~~(1)P Piles shall be verified against structural failure in accordance with 2.4.6.4.~~ ~~(2)P The structure of piles shall be designed to accommodate all the situations to which the piles will be subjected. These include:~~ ~~— the circumstances of their use e.g. corrosion conditions;~~ ~~— the circumstances of their installation e.g. adverse ground conditions such as boulders, steeply inclined bedrock surfaces;~~ ~~— other factors influencing driveability, including quality of joints;~~ ~~— for precast piles, the circumstances of their transportation to site and installation.~~ *Comment: should be part of section 2.* ~~(3)P During structural design, construction tolerances as specified for the type of pile, the action components and the performance of the foundation shall be taken into account.~~ A (3) The consideration of the manufacturing tolerances, which are designated for the type of pile, may be waived by the inner design of the pile, if the resulting unwanted bend loadings are excluded by the load distributing effect of pile crap, pile bent or similar constructions. (4)P Slender piles passing through water or thick deposits of very weak soil shall be checked against buckling.	

(5) Normally a check for buckling is not required when the piles are contained by soils with a representative, undrained shear strength, c_u, that exceeds 10 kPa. A NOTE For (5) For micropiles buckling can also occur in soils that are characterized by an undrained shear strength c_u > 10 kN/m^2. For more details see [EA-Pfähle]. In the case of displacement piles installation effects should be considered. In particular minimum reinforcement should be provided in the case of displacement (screw) piles [Krzystzof]	
9 Execution (supervision, monitoring and maintance)	
[NEW, incorporating existing §7.9] (1)P The construction of piles has to be supervised and monitored according to the requirements of the European execution standards. *[FIN: Execution (supervision etc) is not totally covered in section 4 and relevant execution aspects shall be highlighted in eurocode. Design code and execution standards has to have "suitable" overlapping. In design phase it's relevant for the designer to know the piling equipments (limitations and execution possibilities) etc. → Elaborate a text on issues on execution which are relevant from the design point of view. Precise matching with execution standards.]* ~~(1)P A pile installation plan shall form the basis for the piling works.~~ ~~(2) The plan should give the following design information:~~	

~~— the pile type;~~ ~~— the location and inclination of each pile, including tolerances on position;~~ ~~— pile cross-section;~~ ~~— for cast-in-situ piles, data about the reinforcement;~~ ~~— pile length;~~ ~~— pile number;~~ ~~— required pile load carrying capacity;~~ ~~— pile toe level (with respect to a fixed datum within or near the site), or the required penetration resistance;~~ ~~— installation sequence;~~ ~~— known obstructions;~~ ~~— any other constraints on piling activities.~~ ~~(3)P It shall be specified that the installation of all piles is monitored and records are made as the piles are installed.~~ ~~(4) The record for each pile should include aspects of construction covered in the relevant execution standards, EN 1536:1999, EN 12063:1999, EN 12699:2000, such as the following:~~ ~~— pile number;~~ ~~— installation equipment;~~ ~~— pile cross-section and length;~~ ~~— date and time of installation (including interruptions to the installation process);~~ ~~— concrete mix, volume of concrete used and method of placing for cast-~~	

~~in-situ piles;~~ ~~— weight density, pH, Marsh viscosity and fines content of bentonite slurry (when used);~~ ~~— for continuous flight auger piles or other injection piles, volumes and pumping pressures of the grout or concrete, internal and external diameters, pitch of screw and penetration per revolution;~~ ~~— for displacement piles, the values of driving resistance measurements such as weight and drop or power rating of hammer, blow frequency and number of blows for at least the last 0,25 m penetration;~~ ~~— the power take-off of vibrators (where used);~~ ~~— the torque applied to the drilling motor (where used);~~ ~~— for bored piles, the strata encountered in the borings and the condition of the base if the performance of the pile toe is critical;~~ ~~— obstructions encountered during piling;~~ ~~— deviations of position and direction and as-built elevations.~~ ~~NOTE EN 14199 on the execution of micro-piles is in preparation.~~ ~~(5) Records should be kept for at least a period of five years after completion of the works. As-built records should be compiled after completion of the piling and kept with the construction documents.~~ *Comment: these issues are fully considered by execution codes and section 4.* (6)P If site observations or inspection of records reveal uncertainties about the quality of installed piles, investigations shall be carried out to determine their condition and if remedial measures are necessary. These investigations shall include either performing a static pile load or integrity test, installing a new pile or, in the case of a displacement pile, re-driving	

the pile, in combination with ground tests adjoining the suspect pile. A (6) In addition, a dynamic pile load test can be performed. (7)P Tests shall be used to examine the integrity of piles for which the quality is sensitive to the installation procedures if the procedures cannot be monitored in a reliable way. (8) Dynamic low strain integrity tests may be used for a global evaluation of piles that might have severe defects or that may have caused a serious loss of strength in the soil during construction. Defects such as insufficient quality of concrete and thickness of concrete cover, both of which can affect the long term performance of a pile, often cannot be found by dynamic tests and other tests, such as sonic tests, vibration tests or coring, may be needed in supervising the execution.	