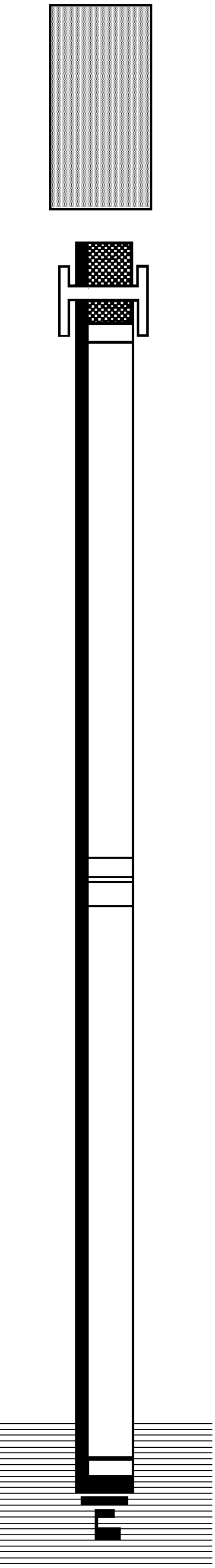


PÅLKOMMISSIONEN

Teknisk PM 1:2018

Structural pile capacity of axially loaded piles in the Nordic countries – recommendations for the revision of Eurocode 7

Delrapport 1(3)

Utförd av

Gary Axelsson och Jimmie Andersson, ELU-konsult

Förord

Pålkommissionen publicerar Gary Axelssons och Jimmie Anderssons arbetsrapport till Projekt Team 4 för nya Eurokod 7, del 3 geokonstruktioner, kapitel pålgrundläggning. Rapporten utgör diskussionsunderlag vid utarbetande av dimensioneringsregler gällande konstruktiv bärförmåga för pålar. Pålkommissionen publicerar artikeln i serien ”Tekniska PM” för att ge en insikt i det pågående arbetet för nya Eurokod 7, del 3.

För Pålkommissionen
Mats Larsson

1 GENERAL

The structural pile capacity (STR) is calculated using DA3 in Sweden, i.e. a material factor approach (MFA) on the soil modulus resisting buckling and deflection. The geotechnical bearing capacity, on the other hand, is calculated using DA2, a resistance factor approach (RFA).
Finland uses RFA (DA2*) when calculating the structural pile capacity. The use a fixed partial coefficient, similar to MFA, on the calculated limiting soil pressure. Norway, on the other hand, uses RFA (DA2) more straight forward, with a correlation factors depending on the number of ground tests.
The governing equations for the loads in ULS are 6.10a and 6.10b according to EN 1990 for both DA2 and DA3. However, a downdrag load due to negative skin friction is considered to be a geotechnical load for which equation 6.10 is used for DA3, but as a structural load for which equation 6.10a is used for DA2.

2 STRUCTURAL PILE CAPACITY (STR) IN SWEDEN

Structural pile capacity, including buckling, is calculated in Sweden, Finland and Norway according to the 2^{nd} order theory, which means that the pile deflection is accounted for during loading. In Sweden the calculations are performed with design values for the yield stress of the pile material as well as for the elastic section modulus. Moreover, design values also apply for the soil bedding modulus as well as for the soil strength.

2.1 The pile material

According to both EN 1992-1-1 and EN 1993-1-1 the calculation of the bearing capacity should be based on a characteristic value accompanied by a partial safety factor.
For steel the partial coefficient when the calculation is based on the yield stress is 1,0 ($\gamma_{m,0}$), see section 6.1 in EN 1993-1-1. No partial coefficient is defined for the elastic modulus of steel but the modulus is set to 210 GPa, i.e. the partial safety factor is 1,0.
For concrete the design elastic section modulus is calculated as:
$E_{cd} = \frac{E_{cm}}{\gamma_{cE}}$, see section 5.8.6(3) EN 1992-1-1. $\gamma_{cE} = 1{,}2$ (SS-EN 1992-1-1)
The compressive design strength for concrete is calculated as:
$f_{cd} = \frac{f_{ck}}{\gamma_C}$, see section 2.4.2.4 Table 2.1N, EN 1992-1-1. $\gamma_C = 1{,}5$ (SS-EN 1992-1-1)

2.2 Soil material, design values according to DA3 (MFA)

Clays with low strength
For clays with low strength, c_u<40 kPa, the elastic modulus is approximately proportional to the shear strength of the clay. For short term loading $E_{clay} \approx 200 \cdot c_u$ and for long term loading $E_{clay} \approx 50 \cdot c_u$ (to account for creep). There is no partial safety factor for elastic soil modulus according to EN 1997-1, but since the modulus and the shear strength are correlated, the same partial safety factor as for the undrained shear strength is used. The design shear strength is calculated as:
$c_{ud} = \frac{c_u \cdot \eta}{\gamma_M}$, where γ_M is 1,5 for undrained shear strength in clay according to SS-EN 1997-1.
The product $c_u \cdot \eta$ is the characteristic value which is a cautious estimate of the mean value (c_u) as described in EN 1990, eq, 6.6a.
The modulus of subgrade reaction, design value, k_d, is calculated as

$k_d = \frac{E_{clay,d}}{d} = \frac{50 \cdot c_{ud}}{d}$ for long term loading and $k_d = \frac{200 \cdot c_{ud}}{d}$ for short term loading.
The limiting soil pressure (q_B) against the pile is assumed to be $6c_{ud}$ for long term loading and $9c_{ud}$ for short term loading.

Non-cohesive soils, relative density
For piles in non-cohesive soil the modulus of subgrade reaction is often calculated based on the friction angle or taken from a table based on the relative density according to Reese (1974) or similar, see Table 1.

Table 1.

Soil description (relative density)	Modulus, k Saturated sand [MN/m³]	Modulus, k Unsaturated sand [MN/m³]
Loose	5	7
Medium dense	16	24
Dense	34	61

The design value of the modulus, k_d, is calculated using the partial coefficient for friction angle, i.e. γ_M is set to 1,3.

Insitu and laboratory testing for subgrade reaction modulus
The modulus, k, can also be obtained directly from pressuremeter tests. In this case the same partial coefficient value for the undrained shear strength (clay) respectively the friction angle (non-cohesive soil) is used in Sweden since there is no partial coefficient for soil modulus in EN 1997-1.
Dilatometer tests could also be used, but the method is mainly used to calculate the p-y response of horizontally loaded piles. CPT could also be used to obtain the modulus of subgrade reaction in cohesionless soils. Moreover, the long-term modulus can be obtained from oedometer test (CRS), and from triaxial testing both short term (undrained) and long-term modulus.

2.3 Calculation model

2.3.1 Buckling (instability problem, 2nd order theory)

When calculating the piles structural capacity both buckling and yielding of the pile material is considered. When the capacity for buckling is calculated, it is taken into account that the soil can behave plastically after a certain deflection.
First the buckling load is calculated based on first order theory:

$$F_{k,d}^{1st} = 2 \cdot \sqrt{k_d \cdot d \cdot E_d I}, \qquad (1)$$

where E_dI is the design flexural stiffness of the pile and d is the diameter or side length of the pile. This formula is based on 3 assumptions:

1. Completely straight pile
2. The pile is elastic

3. The medium is elastic

To account for the initial deflection the 2nd order effects are introduced via a sinusoidal shape pile deformation. The buckling load (design value) regarding 2nd order effects are calculated as:

$$F_{k,d}^{2nd}(y_0) = 2 \cdot \sqrt{k_d \cdot d \cdot E_d I} \cdot \frac{y_0}{y_0 + \delta_0}, \quad (2)$$

where y_0 is the deflection due to actual loading and δ_0 is the initial deflection.
The limiting soil pressure where the plastic state of the soil is reached is calculated as:

$$q_{b,d} = 6 \cdot c_{u,d} \quad \text{for long-term} \quad (3)$$

$$q_{b,d} = 9 \cdot c_{u,d} \quad \text{for short-term} \quad (4)$$

To account for the plastic behavior of the soil the limiting soil deflection y_B must be calculated, see Figure 1. For long term loading it is calculated as:

$$\frac{6c_{ud}}{y_B} = \frac{50\,c_{ud}}{d} \rightarrow y_B = \frac{6d}{50} = 0{,}12d \quad (5)$$

The corresponding value for short term loading $y_B = 0{,}045d$ (i.e. $9d/200$).

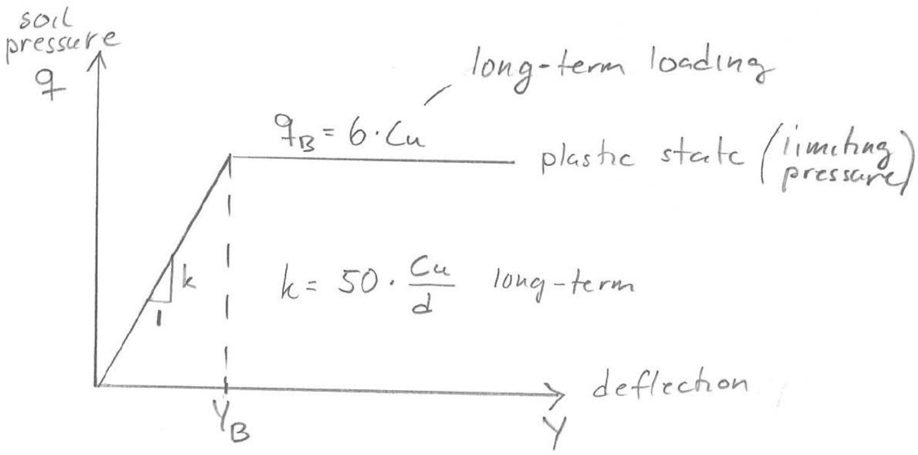


Figure 1.

The corresponding limiting pressure for a non-cohesive soil is calculated using the design values of the passive earth pressure coefficient K_p, which is a function of the friction angle of the soil:

$$q_{b,d} = 3 \cdot K_{p,d} \cdot \sigma_v' \quad (6)$$

The design value is obtained with a γ_M set to 1,3.

2.3.2 Material yield (2nd order theory)

The moment in the pile during loading due to 2nd order theory can be calculated as:

$$M_d(y_0) = \frac{F_{kd}^{2nd} \cdot (\delta_0 + y_0)}{2} \quad (7)$$

To account for yielding of the pile material (steel in this example) the following interaction is used:

$$\frac{F_{kd}^{2nd}(y_0)}{N_{c,Rd}} + \frac{M_d(y_0)}{M_{c,Rd}} \leq 1 \quad (8)$$

where $M_d(y_0)$ is presented above, $N_{c,Rd}$ is the bearing capacity in compression of the steel cross-section according to EN 1993-1-1 and $M_{c,Rd}$ is the bending moment capacity of the steel cross-section according to EN 1993-1-1.
This interaction creates the linear line shown in the figure below.

2.3.3 Pile capacity with regard to buckling and material yield

To determine the structural pile capacity, one can plot the force and moment envelopes (F/M-envelopes) based on the material yield and the buckling-curve. Two modes can appear, where material yield in the pile material determines the capacity or when buckling determines the capacity. When material yield determines the capacity the F/M-envelopes look like in Figure 2 (principal sketch for a steel pile).

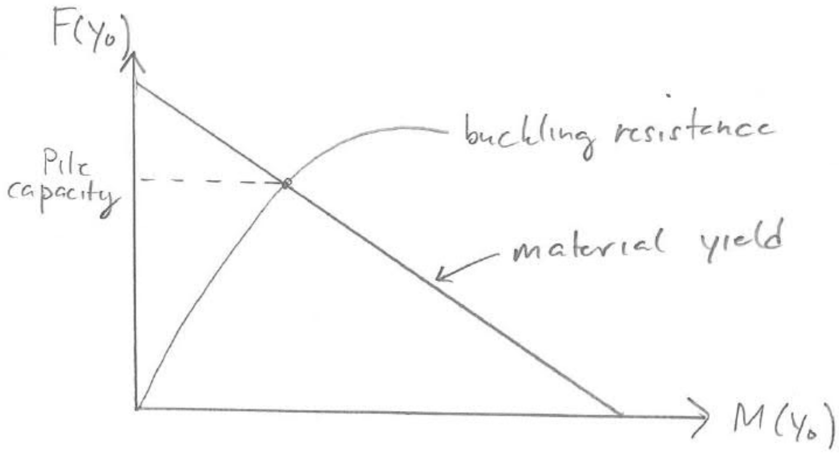


Figure 2.

When buckling of the pile determines the capacity the F/M-envelopes have the appearance shown in Figure 3 (for steel piles).

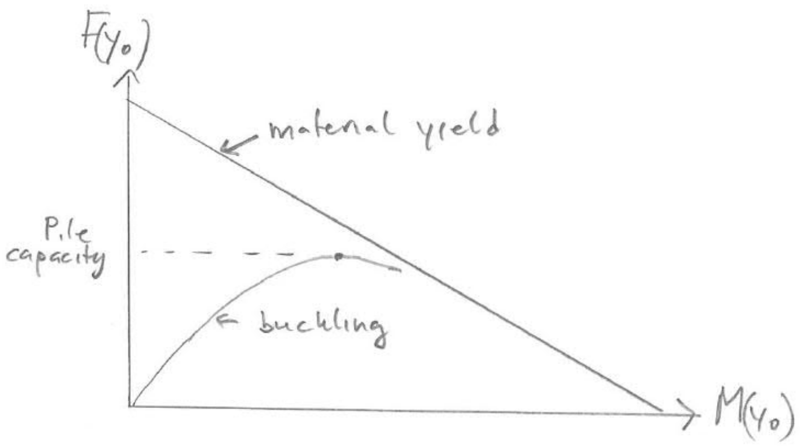


Figure 3.

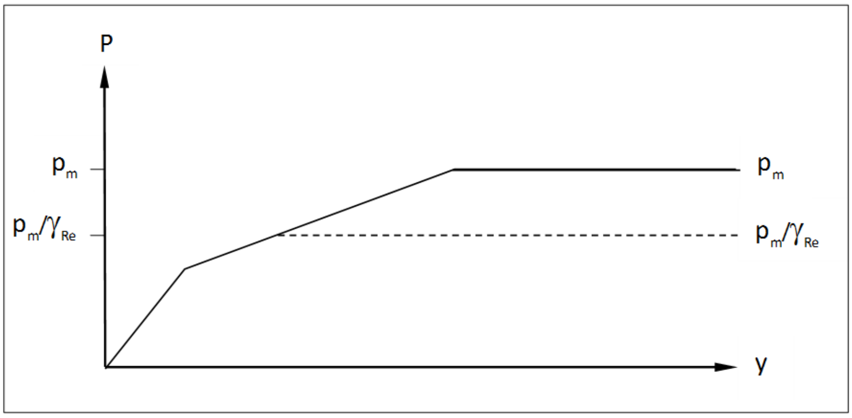
The same procedure is applied for concrete piles, but the material yield curve is a more complicated due to the properties of concrete.
These examples are illustrated with a cohesive material with undrained shear strength. The same procedure can be applied to non-cohesive soils. In this case γ_M=1,3.

3 LIMITATIONS IN SLS

As shown above, the stiffness of the soil has an important influence on the stresses in the pile due to second order effects. Since there are stress limitations for the pile material in SLS for both concrete (max stress 0,6xf_{cc} with regard to creep of concrete) and for steel (no plastic deformation is allowed) according to EN 1992 and EN 1993 respectively. This means that the stresses in the pile should be calculated in SLS also. Furthermore, the stiffness of the soil also has an influence on the crack width for concrete piles. However, the partial coefficients are unity for both the pile and the soil material according to the Eurocodes.

4 STR IN FINLAND, IN BRIEF

Finland uses the same basic equation (2) for buckling as Sweden and Norway. Structural design of piles is described in their national guidelines PO-2016. Finland uses a RFA (DA2*) with γ_{Re} = 1,5 for clay and γ_{Re} = 1,25 for cohesionless soil when calculating the limiting pressure $q_{b,d}$ (p_m in Figure 4), and uses a characteristic value for the soil modulus, as is specified in EN 1997-1.



Soil stiffness, ultimate lateral resistance and design resistance (γ_{re} = 1,5 in Finland)
Figure 4.

This means there is no safety factor on the soil parameters when calculating the structural capacity of steel piles when q_b is not reached and when the pile yields before it buckles. This happens in stiff soils, at small 2nd order deflections. In this case the safety will only be applied on the actions. For concrete piles, there are partial coefficients on strength and modulus.
It should, however, be noted that Finland uses another model than Sweden and Norway for the soil modulus, with a significantly lower modulus when $y > y_B/5$ for long-term loading ($y_m = y_B$ in Figure 5). A similar model is used for short-term loading.

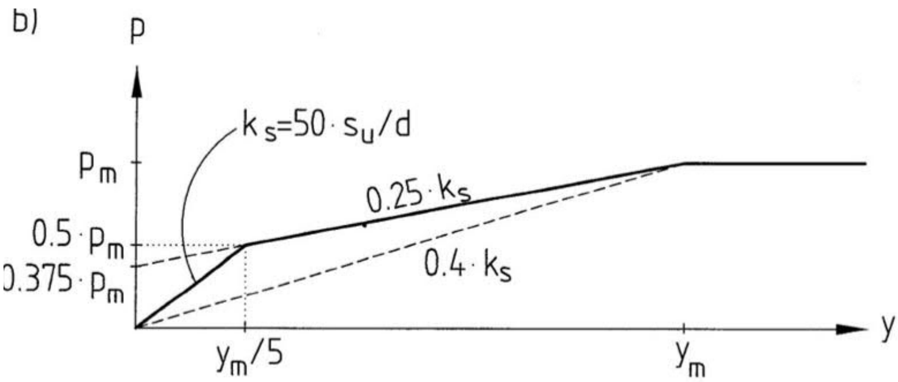


Figure 5.

Finland has stated officially that they use DA2/DA2* for piles in all the ultimate limit states, such as GEO, STR and UPL. However, they adopt a fixed partial coefficient γ_{Re} on the limiting pressure and do not utilize the correlation factors ξ for DA2 according to EN 1997-1.

5 STR IN DENMARK, IN BRIEF

In Denmark, there are no national guidelines how to calculate the structural capacity of piles embedded in soil, taking buckling and pile deflection into account. However, there is a national limitation that is used, from the former Danish standard, stating that slenderness of the pile can be disregarded if the compressive concrete stress does not exceed 10 MPa. Lately, Denmark has recognized that more detailed structural calculations are necessary and that second order analysis is needed even for piles in clay with a c_u greater than 10 kPa.

6 STR IN NORWAY, IN BRIEF

In Norway, pile design with regard to buckling is based on second order theory and equation (2) is used as described in their national guidelines Peleveiledningen 2012. Furthermore, Norway uses DA2 (RFA) by factoring the calculated buckling resistance (force) as follows:

$$F_{k,d}^{2nd} = \frac{F_k^{2nd}}{\gamma_r \cdot \xi} \qquad (9)$$

Characteristic values are used for the soil parameters and the Young's modulus of the pile, E. Furthermore, when the pile material yield is checked, the stresses shall not exceed the design strength of the pile material. The design strength is calculated according to the structural Eurocodes, i.e using partial factors (MFA) on the pile material only. Consequently, in soils where buckling is not a problem, the moment and stress in the pile are calculated with a characteristic soil modulus. For steel piles this means there is no safety on the structural pile capacity since a characteristic soil modulus is used for calculating the 2nd order deflection and the partial factors for steel is γ_M = 1,0. The safety is in this case only applied on the actions. The correlation factor ξ depends on the number and quality of the ground test results and are pre-fixed tabled values listed in their national annex NS-EN 1997-1, table NA.A.10. Note that these correlation factors are foremost used for pile design in GEO, and was not necessarily intended for pile design in STR using 2nd order theory.

Some differences also occur between the Swedish and the Norwegian calculation models. For example, Norway assumes that the limiting soil deflection is y_B = 0,2d for long term loading, compared to equation (5), and y_B = 0,05d for short term. Furthermore, Norway uses a different approach when calculating q_b, rendering values between $5c_u$-$10c_u$, compared to equation (3) and (4).

Table 2 is an excerpt from EN 1997-1 describing the correlation factors for verification of both STR and GEO.

Table 2

Design approach 2, piles, correlation factors:

A.3.3.3 Correlation factors for pile foundations

(1) P For verifications of structural (STR) and geotechnical (GEO) limit states, the following correlation factors ξ shall be applied to derive the characteristic resistance of axially loaded piles:

— ξ_1 on the mean values of the measured resistances in static load tests;

— ξ_2 on the minimum value of the measured resistances in static load tests;

— ξ_3 on the mean values of the calculated resistances from ground test results;

— ξ_4 on the minimum value of the calculated resistances from ground test results;

— ξ_5 on the mean values of the measured resistances in dynamic load tests;

— ξ_6 on the minimum value of the measured resistances in dynamic load tests.

NOTE The values to be ascribed to ξ_1, ξ_2, ξ_3, ξ_4, ξ_5 and ξ_6 for use in a country may be found in its National annex to this standard. The recommended values are given in Table A.9, in Table A.10 and in Table A.11.

Table A.10 - Correlation factors ξ to derive characteristic values from ground test results (n - the number of profiles of tests)

ξ for n =	1	2	3	4	5	7	10
ξ_3	1,40	1,35	1,33	1,31	1,29	1,27	1,25
ξ_4	1,40	1,27	1,23	1,20	1,15	1,12	1,08

7 IMPLICATION ON DESIGN ACTIONS FOR STR

It may seem more logical to use a MFA (i.e. DA3) with the current Eurocodes when calculating the structural capacity since this approach is also used for the pile material parameters. However, there is a minor drawback with MFA since the downdrag load is a geotechnical action and is calculated using eq. 6.10 for DA3 but with eq. 6.10a for DA2 according to EN 1990. Consequently, this gives different design actions when calculating in STR using DA3 and DA2 respectively, which is not a desirable outcome.

8 CONCLUDING REMARKS

According to the structural Eurocodes slender piles shall be calculated using second order theory. The pile structural capacity is not only limited by possible buckling in clay when $c_u <$ 10 kPa as stated in 7.8 (5) in EN 1997-1. The stiffness of the soil may also limit the pile load with regard to pile material yield, as well as stress limitations and cracking in SLS.
DA2 is a resistance factor approach (RFA) where the safety is applied on the calculated capacity (resistance). A general problem in using DA2 for STR is that the structural capacity should be calculated using a material factor approach (MFA) according to the structural codes. To be able to use DA2 also for STR it is important that appropriate changes and clarifications are made in the structural steel and concrete codes. Alternatively, DA2* could be used in which the effect of actions (i.e. stress, moment) is calculated with characteristic values and then factored. However, buckling is an instability problem only depending on the stiff-

ness of the pile and the surrounding soil for which this approach addresses the uncertainty in the soil modulus.
DA3 is a material factor approach (MFA) where the ground strength parameters and structural parameters are factored to produce design values. However, there is no partial coefficient for soil modulus in Eurocode 7 as there is, for instance, on the Young's modulus of concrete.
Using a RFA is more conservative than a MFA since the buckling equation is made up of both the soil stiffness and the pile structural stiffness as shown in equation (2). The correlation factor ξ is applied on the total buckling capacity when using RFA. This gives a higher total factor of safety than if the partial coefficients are applied on E using MFA (compared with today's values of ξ according to table A.10).

9 RECOMMENDATIONS

The clause in 7.8 (5) in EN 1997-1 stating that buckling does not need to be considered when c_u < 10 kPa is misleading. It should be rewritten and cross-references to appropriate paragraphs in the structural Eurocodes are needed for better understanding and ease of use. Furthermore, it should be clarified that second order theory also applies for slender piles (a pile is normally slender). Changes in the structural Eurocodes may also be needed.
The structural pile design can be undertaken using either MFA (DA3) or RFA (DA2/DA2*). However, the following main revision of EN-1997-1 would be necessary depending on what approach is finally chosen.

<u>The following revision is recommended for MFA:</u>

Since the soil modulus has a major influence on the structural capacity and the value often has a large degree of uncertainty, a partial coefficient on the soil modulus should be introduced. Alternatively, upper and/or lower bound values could be used for the soil modulus, like the newly-proposed clauses in prEN 1997-1:2017 for groundwater levels and for soil parameters used in numerical analysis. If upper/lower bound is used, a model factor could be introduced if the overall factor of safety is too low. With MFA there is, however, a partial coefficient on the soil strength when calculating the limiting pressure.

<u>The following revision is recommended for RFA:</u>
A new set of correlation factors ξ is needed in order to consider the combined uncertainty of both soil and pile material parameters <u>together</u>. Today's values of ξ using table A.10 (EN1997-1) renders a significantly higher factor of safety than using MFA. The calculations should be performed with characteristic values for both soil and pile material parameters before applying the correlation factor. Consequently, changes in the structural Eurocodes (steel and concrete) may also be needed.