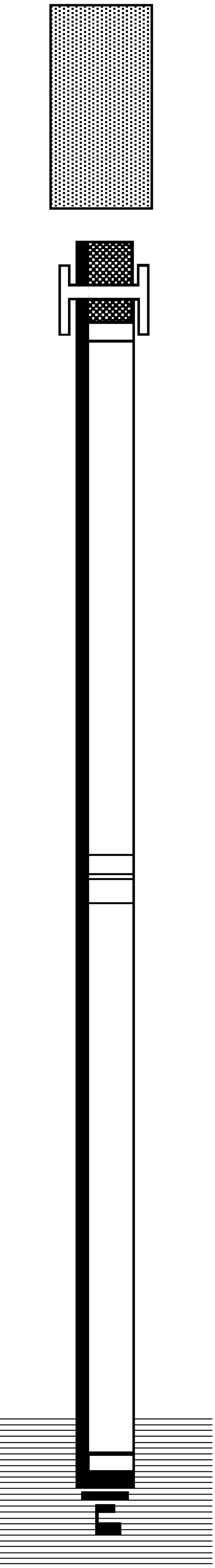

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Design of piles- Swedish practice

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Förord

Pålkommissionen publicerar Gary Axelssons artikel till ”ISSMGE –ETC 3 international symposium on Design of Piles in Europé” Leuven, Belgien 28&29 april 2016 som Tekniskt PM 1 2017. Artikeln genomlyser förutsättningar och metoder för pålning och påldimensionering i Sverige. Artikeln är en utmärkt sammanfattning och ger en bra översikt för den som vill bli bekant med området. Pålkommissionen publicerar artikeln i serien ”Tekniska PM” för att ge allmän åtkomst och referens till övrig litteratur inom ämnet pålning.

För Pålkommissionen
Mats Larsson

Design of piles – Swedish practice

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ABSTRACT

More than 95% of the piles installed in Sweden are made up of driven displacement piles or drilled end-bearing piles to hard till or into hard crystalline rock. The piles are either concrete pre-cast piles or small diameter steel pipe piles. The most common in-situ investigation methods with regard to piling are super-heavy dynamic probing and percussion drilling to into rock. All the end-bearing piles are driven to a termination criterion. The geotechnical design capacity (GEO) is often determined by dynamic testing and Design approach 2 is used. Because of the very high end-bearing capacities, the structural capacity of the piles can often be critical for design as it is not uncommon with very soft normal consolidated clay overlaying the till or rock. The structural capacity (STR) is checked for buckling and material yield using 2nd order theory and Design approach 3 is used.

1. REGIONAL GEOLOGY

1.1. The main geology of Sweden

Sweden is part of the geological Fennoscandia Shield which includes Sweden, Norway, Finland and part of north-western Russia. Approximately 75 % of the bedrock in Sweden is covered with very dense moraine (till) and for 10 % of the area there is no soil (or less than 0,5 m) overlaying the bedrock. In the remaining areas the bedrock is directly covered with various glacial or post-glacial sediments such as clay, silt and sand, or by peat. The last ice age ended around ten thousand years ago, so the Swedish geological soil history is very young. On the other hand, our bedrock is one of the oldest in Europe and the most common rocks are crystalline, such as granite and gneiss.

Nearly all of Sweden, except the southern part (Skåne and the large islands Öland and Gotland) consist of very hard rock, with uniaxial compression strength in the region of 100-300 MPa. It is common that the bedrock surface varies (undulates) significantly within short distances. In the south, where sedimentary rock dominates (limestone and sandstone), hard clay-till is often encountered. The geology here is similar to what is found across the Öresund in Denmark.

During the melting period of the ice-mass, moraine of variable thickness was deposited on top of the bedrock. When the moraine was subjected to extremely high pressure from the large ice-mass it became very dense, so called till. Overlaying the till in some places are deposits of loose moraine, which were not subjected to high pressure. The moraine is often very well-graded and the dominant fractions vary between silt and gravel, except in the south where clayey till predominates. It is not uncommon to encounter larger boulders in moraine.

Loose deposits of alluvial sediments of clay silt or sand generally cover the moraine. The clay in Sweden is predominately post-glacial. Normally under a thin layer of dry crust the undrained shear strength of the clay is very low (10-20 kPa) or even extremely low (< 10 kPa). Furthermore, the clay is normally consolidated or only very lightly over-consolidated. This means large settlements can occur even for a slight ground-water lowering or placement of a thin layer of fill on top of the clay. The thickness of the clay can exceed more than 100 m as it does in Gothenburg, Sweden’s second largest city.

A special geological feature commonly found in Sweden is the glacial esker, see figure 1. This geological formation was formed by a river running beneath the great ice-mass, often situated over a fault in the bedrock where the water could easily flow. The soil material in a typical esker consists of relatively rounded washed particles of mainly loose coarse soils, such as sand, gravel and stones. Moreover, boulders are often found in this type of formation. Normally, the esker is partly covered with glacial and post-glacial clay. The city centres of large cities like Stockholm, Uppsala and Södertälje are founded on top of an esker formation.

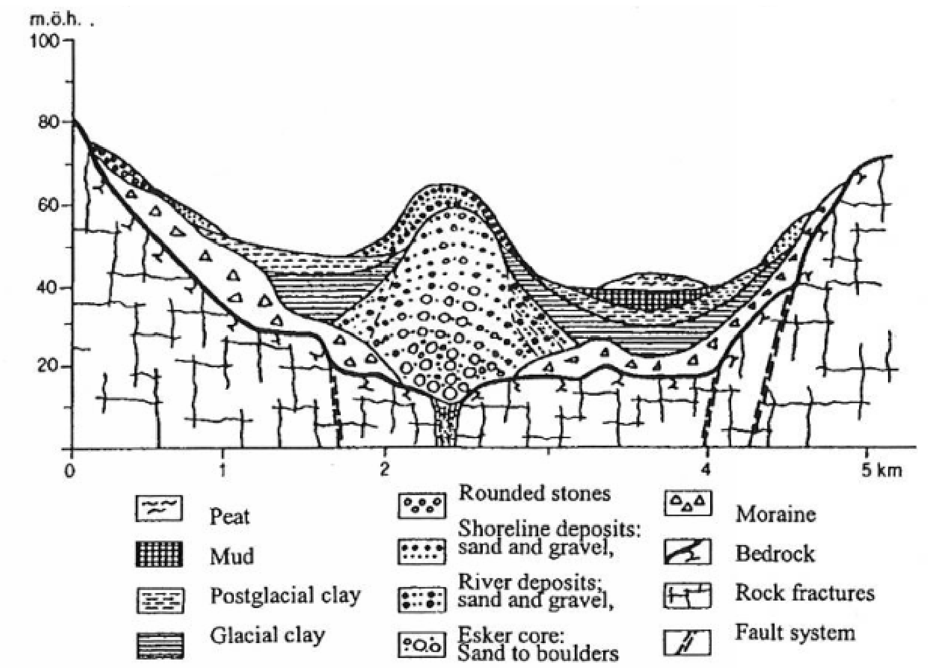

Figure 1: A schematic section through a typical glacial esker (Knutsson & Morfeldt, 1995)

1.2. Implications on piling

In Sweden, the geological conditions are mostly very favorable for driven end-bearing piles. Soft clay overlaying very dense moraine or hard rock is ideal for driven displacement piles. The geotechnical end-bearing capacity is normally very high on either till or crystalline rock. However, the very low shear strength of the clay means that the structural capacity with regard to buckling has to be checked.

2. SOIL INVESTIGATION

2.1. General description

Generally, the soil investigation with regard to piling is mainly focused on finding the hard till or the bedrock and to determine the undrained shear strength of the soft clay. The most common in-situ investigation methods used in Sweden are (not in priority order):

- Swedish weight sounding, WST (penetration resistance in clay and silt)
- Cone penetration, CPTU (cone resistance in clay and non-cohesive soil)
- Super-heavy dynamic probing, DPSH-A (ramming resistance in non-cohesive soil)
- Percussion drilling with continuous measuring, MWD (location of boulders in soil, the bedrock surface and a rough estimate of the rock quality)
- Field vane test, FVT (undrained shear strength)
- Soil tube sampling (undisturbed, for laboratory testing)
- Screw sampling (disturbed, for soil classification)

The most common laboratory testing methods with regard to piling are:

- Fall cone test, FCT (undrained shear strength)
- Oedometer test with constant rate of strain, CRS (compressibility characteristics)

2.2. Investigation methods with regard to pile function

2.2.1.End-bearing piles

Design of end-bearing piles is based on either dynamic pile load test or on a pile termination criteria (stop set) based on experience (i.e. a max allowed set per blow).

End-bearing piles are either driven to hard till or bedrock or drilled piles into the rock (rock-socket). The super-heavy dynamic probing (DPHS-A) with a 63 kg ram and 50 cm fall height is often used as an indication of pile termination into the till. Percussion drilling (MWD) is used as a complement to find large stones or boulders, to locate the bedrock surface and to obtain a rough estimate of how much the rock is fissured. Furthermore, if there is soft clay in the profile, the undrained shear strength is determined for use in structural design (buckling).

2.2.2. Friction piles (cohesionless soil)

Friction piles are mostly designed by pile load testing; driven piles by dynamic testing and in-situ manufactured piles by static testing. Calculation methods provide an estimate of the pile length and capacity.

Driven friction piles are often pre-cast concrete piles (figure 2). The same methods used for end-bearing piles i.e., DPSH-A and MWD, are also used for friction piles. Here, the DPSH-A is mainly used to get an indication of the pile drivability and MWD is to locate possible boulders in the profile. The cone resistance, $q_{c,}$ obtained from CPT is often used as an input for calculating the estimated bearing capacity that can be expected from dynamic pile load tests. Unfortunately, the CPT-rigs in Sweden are generally lightweight. They do not provide adequate tip resistance and the sounding often terminates at $q_c \sim 20\ Mpa$. Heavier CPT-rigs are required in Sweden to get satisfactory input results for design of friction piles.

2.2.3. Cohesion piles

Cohesion piles in soft clay are the only pile types where the design is based on calculation; it is very rare that static load testing is undertaken.

Cohesion piles are predominately driven pre-cast concrete piles or combi-piles (i.e. wooden and concrete combined). The design of cohesion piles in soft clay is based on calculation by the α-method where the undrained shear strength is the main parameter. The soil investigation mostly consists of in-situ FVT and/or soil sampling and laboratory FCT. Furthermore, piezocones sometimes are installed at different levels in the clay for obtaining pore pressure measurements. From undisturbed soil samples the compressibility parameters of the clay is determined. The laboratory results provide, together with the pore pressure measurement, the input for settlement and negative skin friction calculations, and also an evaluation if ongoing settlements are occurring.

3. PILING TECHNOLOGY & CLASSIFICATION

3.1. Common pile types

The following pile types are most commonly used in Sweden by falling order of importance (source: Pile Commission statistics 2014):

1. **Driven pre-cast concrete piles (60%)** (figure 2); the two main dimensions are 235 mm square and 270 mm square, but up to 400 mm square can be made. The pile can be used as end-bearing, friction or as a cohesion pile.
2. **Driven steel pipe piles (23%)** (figure 3); dimensions vary between 76 mm up to 219 mm and further from 270 mm up to 1220 mm (large diameter piles). Used as an end-bearing pile. The piles will preferably be filled with concrete for increased stiffness, corrosion strength and for internal corrosion protection.
3. **Drilled steel pipe piles (13%)** with the same dimensions as above. However the maximum practical diameter is 813 mm. Used as end-bearing piles in rock (rock socket). Filled with concrete.
4. **Timber piles (4%).** The piles are used as cohesion piles. Permanent piles are always combined with a concrete pile section (combi-pile) above the groundwater table.
5. **Steel core piles (<1%)** (figure 4). A steel casing is drilled down and left to rest on the bedrock. A drilling continues down into the rock to create a socket. A massive steel bar (core), normally between 70-150 mm in diameter is placed in the casing and down into the socket. The space between the steel bar and casing is filled with concrete. The pile can be either end-bearing on rock or, in case of tension piles or bad rock quality, shaft-bearing in rock (common length in rock 3-6 m). Execution standard EN 14199 is used for this pile.
6. **Other pile types occasionally used are:**
    - Shaft grouted drilled piles (e.g. TITAN Ishcebeck).
    - Cast in-situ piles, where normally the soil is excavated inside a casing tube, which is at the same time pushed down.
    - CFA-piles.

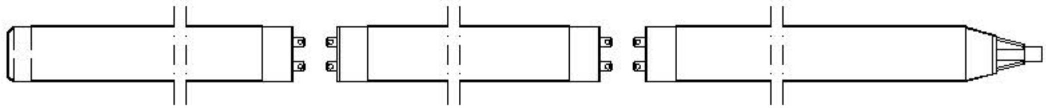

Figure 2: Standard pre-cast concrete pile, with mechanical joint and a rock tip (shoe).

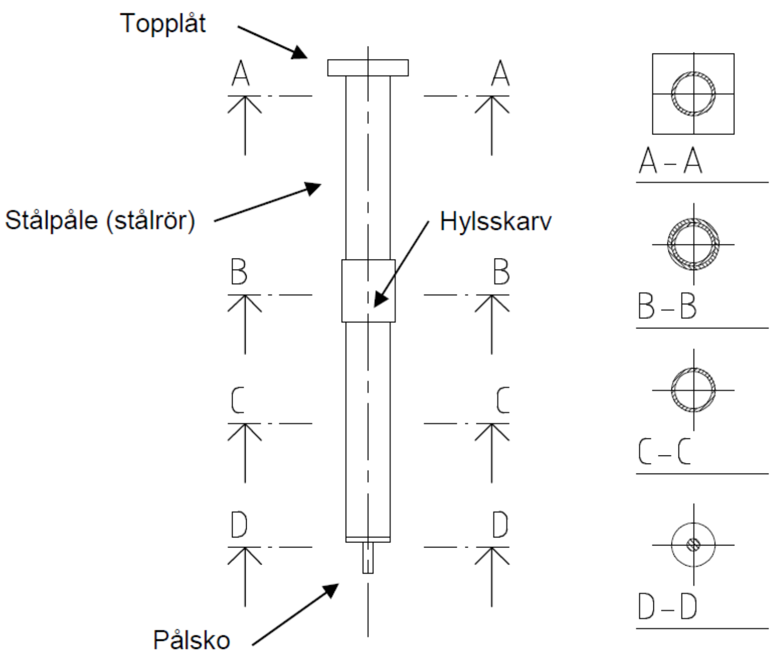

Figure 3: Driven steel pipe pile.

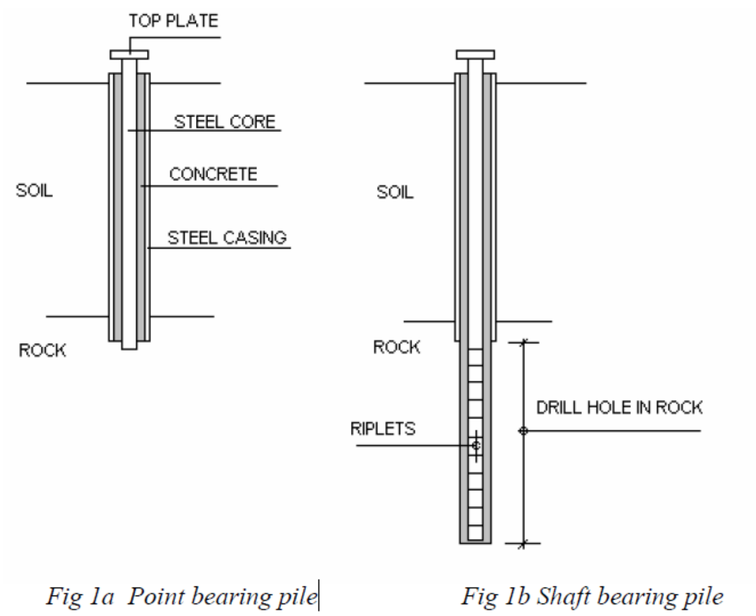

Figure 4: Steel core pile (Bredenberg).

4. NATIONAL DOCUMENTS

4.1. Pile design and installation

In Sweden two national annexes are used, one for infrastructure and one for buildings:

- VVFS 2004:43 (with changes in TRVFS 2011:12) provided by Trafikverket (The Swedish Transport Administration)
- BFS 2015:6 EKS 10, provided by Boverket (The Swedish National Board of Housing, Building and Planning).

Today (January 2016) pile design in Sweden is the same using either of the two annexes.
In the previous annexes by Boverket, EKS1 through EKS 9, all the partial coefficients γ_R were a factor 0,1 higher than the annex provided by Trafikverket. Furthermore, no reduction of the correlation coefficients for dynamic testing, ξ_5 and ξ_6, for stiff foundations was allowed. This gave a 19 % higher total factor of safety (TFS) for dynamic testing and 8 % higher TFS for static testing and design based on soil investigation.

With regard to pile installation the following execution standards are normative with regard to pile installation:

- SS-EN 12699, Displacement piles
- SS-EN 1536, Bored piles
- SS-EN 14199, Micro piles

4.2. Technical specifications and complementary documents

For the purpose of The Swedish Transport Administration procurements, use is normally made of a tender enquiry document model that concurs with AMA Anläggning 13 (a general description of material and works). Furthermore, Trafikverket has a document for specification of technical design; TK Geo 13 regarding GEO and TRVK Bro 11 regarding STR.

Complementary documents that are normally referred to with regard to pile design are:

- IEG Tillämpningsdokument Pålgrundläggning, Rapport 8:2008. A design Guide to SS-EN 1997-1.
- Pile Commission reports, e.g. for concrete piles, small slender pipe piles, friction piles, cohesion piles, steel core piles, drilled steel pipe piles etc. (all in Swedish).

5. DESIGN METHOD ACCORDING TO THE PRINCIPLES OF EUROCODE 7

5.1.1.Actions

When comparing different nations with regard to the total safety on the resistance side, it is important to bear in mind changes made on the safety on actions in EN 1990. Sweden has introduced three “safety classes” on actions, SK1, SK2 and SK3. The classes only reflect the possibility of human injury/fatality and how substantial the effect of a collapse will be. The lowest class with respect to safety, SK1, may be used when large deformations of the foundation cannot bring about a sudden collapse of the overlaying structure, or if people are very seldom in the vicinity of the structure. On the other hand, SK3 requires that large deformations will lead to a sudden collapse of the structure and that many people are frequently in the proximity of the structure. SK2 is applicable for all other cases and is the safety class that is mostly used in Sweden. A partial coefficient γ_d on actions is introduced for the different classes:

- SK1: $\gamma_d = 0{,}83$
- SK2: $\gamma_d = 0{,}91$
- SK3: $\gamma_d = 1$ (no reduction)

The result of the safety classes is a reduction in the safety on actions with regard to EN 1990, except for SK3. Moreover, the reduction factor ξ in equations 6.10b (EN 1990) is set to 0,89. Overall this means that the Swedish safety factor on actions is generally smaller than for most of the other countries using the Eurocode.

Since DA3 is used for STR, geotechnical actions are calculated according to equation 6.10 instead of 6.10a/6.10b as for DA2 and GEO. The main implication of this is how the action from negative skin friction is calculated. For GEO only the derived mean value of c_u is used. For STR, however, the characteristic value c_{uk} is used (high value).

5.1.2. Geotechnical category

In Sweden geotechnical category GK 1 and 2 should be used for well-established piling methods with regard to the geotechnical conditions. For GK2, the bearing capacity should be verified by calculation and/or load testing. In GK3 an independent reviewer should be appointed by the builder and have third party interests as the main focus. GK2 is considered to be the normal category.

Example of pile foundations in GK1:

- Footings or cast in-situ piles on low fissured rock above the groundwater table and low relative compression stress.
- Conventional driven pre-cast concrete (figure 2) piles or steel pipe piles (figure 3) with small relative l**o**ads with regard to the end-bearing capacity and the structural capacity.
- Furthermore, the risk of stability problems or damaging soil movement should be negligible.

Example of pile foundations in GK3:

- Large possibility of stability problems or damaging soil movement.
- Very complex ground conditions which require monitoring during installation
- In-situ manufactured piles, e.g. CFA-piles, in untested conditions.
- Exceptional loading conditions, e.g. piles subjected to large tension forces or a large degree of dynamic loading.

5.1.3. Design approach

DA2 is used for GEO. DA 2 with the “model pile” procedure is the primary design approach. When using the the “alternative” procedure an extra model factor, $\gamma_{Rd,e,}$ of 1,4 is applied since no correlation factors are used.

DA3 is used for STR when calculating the structural capacity for the pile, i e. partial factors is used on the soil´s strength and stiffness parameters. Structural capacity should be calculated taking into account deformations/deflections caused by the loading (2nd order theory).

5.1.4. Partial coefficients

Partial resistance factors for pile foundations for GEO according to Tables A.6, A.7 and A.8 in EN 1997-1. The following changes are made for R2 (DA2) according to the national annexes:

- Table A6 (driven piles): $\gamma_{b,}$ $\gamma_{s,}$ γ_t = 1,2 and $\gamma_{s;t}$ = 1,3 (0,1 higher value than EN 1997-1)
- Table A7 and A8 (bored/CFA piles): $\gamma_{b,}$ $\gamma_{s,}$ γ_t = 1,3 and $\gamma_{s;t}$ = 1,4 (0,1 higher value)

Partial resistance factors for pile foundations for STR according to Tables A.6, A.7 and A.8 in EN 1997-1. No changes are made for R3 (DA3), $\gamma_{b,}$ $\gamma_{s,}$ γ_t = 1,0 and $\gamma_{s;t}$ = 1,1 are left unchanged. However, changes are made in table A.4 in EN 1997-1 as follows:

- $\gamma_{\phi'} = 1{,}3$ (0,05 higher value)
- $\gamma_{c'} = 1{,}3$ (0,05 higher value)
- $\gamma_{cu} = 1{,}5$ (0,1 higher value)
- $\gamma_{qu} = 1{,}5$ (0,1 higher value)

5.2. Definitions and symbols

η = Correction factor taking into account the uncertainties related to both the soil and the structure.
$\bar{X}$ = Mean value
ξ = Correlation factor
δ_0 = Initial pile defection
γ_M= partial factor of safety for material parameters
γ_{ce}=partial factor of safety concrete elasticity modulus
γ_c=partial factor of safety concrete elasticity modulus
c_{ud} = Undrained shear strength of clay
d = Pile diameter
f_{ck} = Characteristic yield strength of concrete
f_{cd} = Design value of yield strength of concrete
k_d = Design value of bedding modulus
q_u = Uniaxial compression strength
y_0 = Pile deflection due to loading
y_B = Limiting soil deflection at yield
A = Cross sectional area
E_{cm} = Average value of elasticity modulus for concrete
E_{cd} = Design value of elasticity modulus for concrete
E_d =Design value of elasticity modulus for the pile
F_{cd}= Buckling load, design value
I = Moment of inertia
R_k =Characteristic pile resistance
R_{ck} =Characteristic pile resistance in compression
R_{cal} = Calculated pile resistance
R_m = Estimation of measured pile resistance
$R_{m,max}$ = Maximum pile resistance from dynamic testing due to pile material strength
R_{mean} = Measured pile resistance (mean value)
R_{min} = Measured pile resistance (minimum value)
R_d = Design value of pile resistance

5.3. ULS Design based on soil investigation test results

5.3.1.Design values GEO

The design value for the "model pile" procedure (DA2):

$$R_d = \frac{1}{\gamma_{R_d}} \cdot \frac{R_k}{\gamma_R} \tag{1}$$

where

$$R_k = \frac{R_{cal}}{\xi} \tag{2}$$

The design value for the "alternative" procedure (DA2):

$$R_d = \frac{1}{\gamma_{Rd,e}} \frac{1}{\gamma_{R_d}} \cdot \frac{R_k}{\gamma_R} \tag{3}$$

where

$$R_k = R_{cal} \tag{4}$$

For the "alternative" procedure an extra modelfaktor $\gamma_{Rd,e}$ =1,4 is used to provide for the lack of correlation factor when calculating R_k.

5.3.2.Axial compression of a single pile (GEO)

In Sweden, design based on calculation from soil investigation results is predominately only undertaken for cohesion piles. For friction piles, design by calculation is very seldom undertaken. However, calculation is used for estimating the pile length for friction piles as part of the planning for dynamic pile testing. Model factors for calculation are shown in table 1 and 2.

In contrast to EN 1997-1, wave-equation analysis (WEAP) for determining the termination criterion of end-bearing piles is considered a calculation and design is performed according to the "alternative" procedure. Model factors are shown in table 3. Pile driving formula is not allowed in Sweden for determining the termination criterion (stop set).

Table 1 : Model factors for friction piles

Calculation method type	**Accepted methods (example)**	γ_{Rd}
Geostatical method based on friction angle (alternative procedure)	- API-RP-2A - Toolan (1990)	1,6
Method directly correlated to SPT or DPSH-A	- Decourt (1982)	1,5
Method directly correlated to CPT	- Bustamente & Giansellis (1982) - ICP-method	1,4

Table 2 : Model factors for cohesion piles

Calculation method type	**Accepted methods (example)**	γ_{Rd}
Alpha-method, undrained parameters(c_u)	- Pile Commission report no 100	1,1
Beta-method, drained parameters (alternative procedure)	- Flaate & Selnes (1977) - ICP-method	1,2

Table 3 : Model factors for end-bearing piles.

Pile type	**Accepted method**	γ_{Rd}
Driven end-bearing piles	Wave equation analysis	1,3
Drilled end-bearing rock-socket piles	Wave equation analysis	1,1

Axial tension of a single pile (GEO)

According to Pile Commission report 103, the reduction factor for friction piles in tension (compared to compression) can be calculated according to the expression presented by Nicola & Randolph (1993). The reduction factor normally varies between 0,7-0,9. In the national annexes a reduction factor of 0,7 should be used if it is not determined by calculation.

For cohesion piles there is no reduction for tension required.

5.3.3.Specific issues

As mentioned previously, the design load from negative skin friction is calculated slightly differently for GEO and STR respectively. Hence, the design loads are also different for the same load combination.

Seismic design is not needed to be accounted for because of the very low seismic activity in Sweden.

5.3.4.Problems not covered by National Annexes and future developments

The EN 1997-1 does not provide any guidance how to calculate the design load from negative skin friction based on soil-structure interaction and with regard to the neutral plane. The Swedish Pile Commission has started a project group which will provide guidelines for this purpose. Furthermore, another project group is working to see if DA 2 can also be used for STR, i.e. partial coefficients on the structural material parameters combined with a safety factor on the overall resistance.

5.4. SLS design

Normally the structural engineer provides the allowable total and differential settlement for the structure.
SLS design with regard to settlement is predominately undertaken for cohesion piles. For end-bearing piles the settlement is very small and mainly consists of elastic deformation of the piles..

A check should be made that neither the pile material nor the surrounding soil reaches the plastic state when loaded in SLS due to pile deflection.

The observational method for piling is mainly used with regard to monitoring soil movement for stability of clay slopes or for monitoring ground settlement. The measures undertaken are often changing the pile installation order and/or by excavating the clay before installing the piles.

5.5. Design based on load tests

Dynamic load testing is the main design method in Sweden since an overwhelming majority of our piles are either driven end-bearing piles or drilled end-bearing piles. The bearing capacity is evaluated using the Case-method. For the driven friction piles, the dynamic testing together with signal matching (CAPWAP) is always performed. The characteristic value is calculated as follows:

$$R_{ck} = \frac{R_{mean}}{\xi_5} \tag{5}$$

$$R_{ck} = \frac{R_{\min}}{\xi_6} \tag{6}$$

No changes have been made to the correlation factor values presented in table A.11 in EN 1997-1. However, three more columns have been added; one for 4 piles, one for more than 40 piles and one if all the piles are measured, see table 4. Furthermore, at least 3 piles instead of 2 should be tested within an area of max 25x25 m. In table 5, all the model factors for dynamic testing are presented; most are found in the Swedish annexes as minimum allowed values. The last two values in the table can only be found in the Swedish Pile Commission report no. 106.

Table 4 : Correlation factors for dynamic testing according to the Swedish annexes.

n	**3**	**4**	**≥5**	**≥10**	**≥15**	**≥20**	**≥40**	**all**
ξ_5	1,6	1,55	1,5	1,45	1,42	1,4	1,35	1,3
ξ_6	1,5	1,45	1,35	1,3	1,25	1,25	1,25	1,25

Table 5 : Model factors for dynamic testing.

Pile function	**Method of analysis**	γ_{Rd}
Friction piles	Case-method.	1,2
Friction piles	Case-method and CAPWAP-analysis.	0,85
Friction piles loaded in tension	CAPWAP-analysis. A reduction factor of 0,7 should also be used.	1,3
Cohesion piles	CAPWAP-analysis. Correlation should be made with static load tests.	1,0
End-bearing piles	Case-method.	1,0
End-bearing piles	Case-method and CAPWAP-analysis.	0,85
End-bearing piles on hard till or rock	Case-method. Set per blow less than 2 mm and a quake less than 60/d	0,85
End-bearing rock-socket piles	Case-method or set per blow less than 1 mm.	0,80
Shaft bearing rock-socket piles (grouted)	Wave-up[1] method.	0,80

5.6. Design based on experience

The use of prescriptive measures for piles design is limited to the following cases:

- Driven concrete end-bearing piles of standard dimension 235 and 270 mm square respectively. The piles are terminated at maximum 10 mm per 10 blows on till or at maximum 3 mm per

[1] The grouted bearing capacity is evaluated from the returning stress wave, i.e. the wave-up.

10 blows on rock using a free-fall hammer. In table 6, as an example, design bearing capacities for 270 mm square piles with regard to fall height and hammer weight are presented.

- Driven steel pipe piles of dimensions from 76-219 mm in diameter. The piles are terminated at 5 mm per 10 blows by a free-fall hammer or at 5 mm per minute by hydraulic or pneumatic hammer.
- Footings or cast in-situ piles on rock. The maximum design compression stress is limited to 10 MPa for crystalline rock.

Table 6: Design values of bearing capacity for driven end-bearing concrete piles (270x270 mm).

Termination	Fall height	4 tonnes	5 tonnes
Till: 10 mm/ 10 blows	0,3 m	620 kN	670 kN
	0,4 m	730 kN	800 kN
	0,5 m	825 kN	850 kN
Rock: 3 mm/ 10 blows	0,3 m	680 kN	740 kN
	0,4 m	800 kN	830 kN
	0,5 m	855 kN	NA

5.7. Structural design

5.7.1. Design values STR

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, see section 6.1 in EN 1993-1-1. No partial coefficient is defined for the elastic modulus of steel but the modulus itself is defined to be 210 GPa, which would make the partial safety coefficient 1,0.

For concrete the design elastic section modulus is calculated according to EN 1992-1-1:

$$E_{cd} = \frac{E_{cm}}{\gamma_{CE}} \tag{7}$$

and $\gamma_{CE} = 1{,}2$

The design compressive strength of concrete is calculated according to EN 1992-1-1:

$$f_{cd} = \frac{f_{ck}}{\gamma_C} \tag{8}$$

and $\gamma_C = 1{,}5$

Soil material

The design value for geotechnical parameters when calculating the pile structural capacity (DA3, STR):

$$X_d = \frac{1}{\gamma_M} \cdot \eta \cdot \overline{X} \tag{9}$$

where:

η is a correction factor taking into account the uncertainties related to both the soil and the structure (normal values range between 0,8-0,95). The product $\eta \cdot \bar{X}$ is considered the characteristic value.

η can be divided into 8 parts, $\eta = \eta_1 \cdot \eta_2 \cdot \ldots \cdot \eta_8$ where:

- η_1 and η_2 have to do with the number of ground tests and the coefficient of variation, V, see figure 5.
- η_3-η_5 have to do with the quality of ground tests and geotechnical conditions

- η_6 and η_7 have to do with the ground structure, e.g. if the failure will be ductile and if load transfer can take place between piles.
- η_8 is used as a statistical correction when necessary.

In contrast, when calculating the negative skin friction (a load), the mean value of c_u is divided (instead of multiplied) by η to get the characteristic value, i.e. c_u/η.

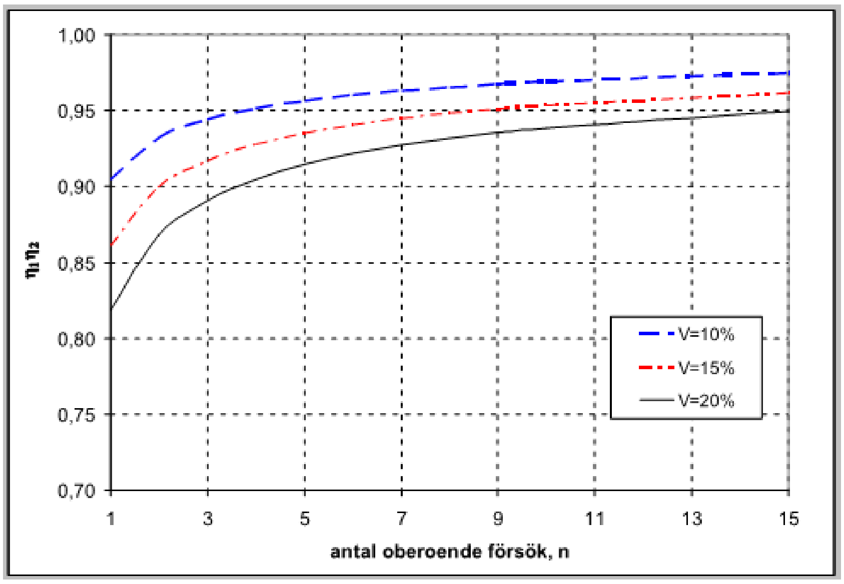

Figure 5 : The product of the correction factors, $\eta_1 \cdot \eta_2$, as a function of the number of tests, n (*IEG Rapport 8:2008, rev 2)*.

5.7.2. Buckling and compression resistance

Pile buckling and material yield are calculated according to 2nd order theory which means that the pile deflection during loading is accounted for. The calculations are performed with design values for the yield stress of the pile material as well as the elastic section modulus. Design values also apply to the soil bedding modulus.

When calculating the buckling load of piles the bedding modulus, k, of the soil is required. It is assumed that the bedding modulus for clay is directly proportional to the undrained shear strength with $k \cdot d = 200 \cdot c_u$ for short term loading and $k \cdot d = 50 \cdot c_u$ for long term loading. Buckling will not occur in cohesionless soils because of the large bedding modulus. However, material yield due to an additional bending moment because of pile imperfection and second order effects have to be checked. The bedding modulus for cohesionless soil according to empirical values by Reese (1974) is normally used.

Note that Eurocode does not have a partial coefficient on soil modulus for ULS, which raises the question, what safety factor to use if the bedding modulus is determined directly from pressuremeter tests.

<u>Buckling (instability problem, 2^{nd} order theory)</u>

When calculating the piles structural capacity, both buckling and yielding of the pile material is taken into account. When the capacity for buckling is calculated, it is assumed that the soil can behave ideally plastic when the critical deflection is reached. First the buckling load is calculated based on first order theory:

$$F_{cd} = 2 \cdot \sqrt{k_d \cdot d \cdot E_d I} \qquad (10)$$

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 soil medium is elastic.

To account for the initial deflection the 2nd order effects are introduced via a sinusoidal shape. The buckling load (design value) with regard to 2nd order effects are calculated as (Pile Commission report 84a):

$$F_{cd}^{2nd}(y_0) = 2 \cdot \sqrt{k_d \cdot d \cdot E_d I} \cdot \frac{y_0}{y_0+\delta_0} \tag{11}$$

where y_0 is the deflection due to actual loading and δ_0 is the initial deflection. δ_0 is normally $L_c/300$ for concrete piles without a joint and $L_c/150$ with a joint. For steel pipe piles the initial deflection can be measured using an inclinometer.

The buckling length along the pile is calculated from:

$$L_c = \pi \sqrt[4]{\frac{E_d \cdot I}{k_d \cdot d}} \tag{12}$$

To account for the plastic behavior of the soil, the limiting soil deflection y_B has to be calculated. For long term loading it is calculated as:

$$\frac{6c_{ud}}{y_B} = \frac{50c_{ud}}{d} \rightarrow y_B = \frac{6d}{50} = 0{,}12d \tag{13}$$

For short term loading the 6 is replaced by a 9 and 50 is replaced by 200.

<u>Material yield (2nd order theory)</u>

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

$$M_d = \frac{F_{cd}^{2nd} \cdot (\delta_0 + y_0)}{2} \tag{14}$$

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

$$\frac{F_{cd}^{2nd}}{N_{c,Rd}} + \frac{M_d}{M_{c,Rd}} \leq 1 \tag{15}$$

where:
M_d is presented above,
$N_{c,Rd}$ is the bearing capacity in compression of the steel cross-section according to EN 1993-1-1,
$M_{c,Rd}$ is the bending moment capacity of the steel cross-section according to EN 1993-1-1.

5.7.3. Transverse loading of piles

In Sweden, the pile foundations are very seldom entirely lateral loaded, normally the vertical load is the main load. Pile foundations subjected to high lateral loads will be designed using inclined piles as shown in figure 6b and where the main pile load will be axial. If the lateral load is relatively small the piles may be installed vertically as shown in figure 6a. The bending moment due to the lateral load is normally deducted from the moment capacity when calculating the axial capacity of the pile. A common problem in Sweden is settling ground on inclined piles, often giving rise to large additional moments as depicted in figure 6c.

Calculation models for transverse loading are presented in the Pile Commission report no. 101. Design values for the bedding modulus and material properties are the same as outlined for buckling and material yield compression.

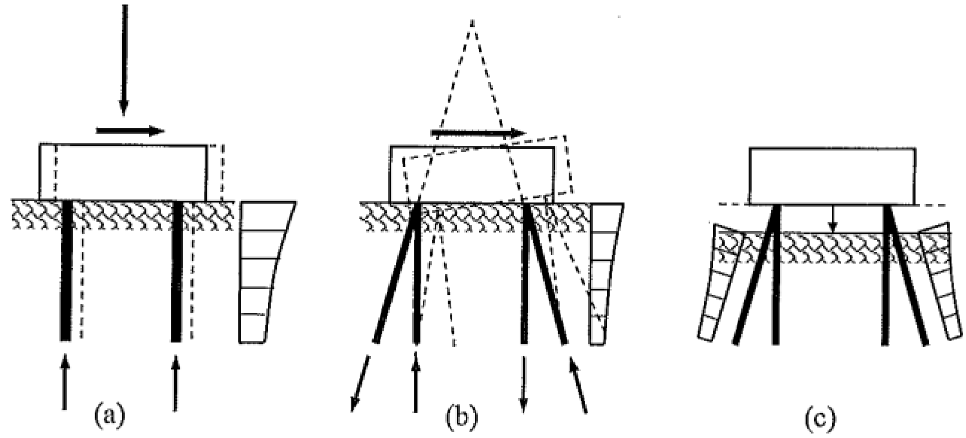

Figure 6: Three principle cases where the piles are transversally loaded (Pile Commission report no.101).

5.7.4.Allowable stresses during installation and piles testing

Recommendations for maximum design load with regard to allowable stresses in the pile during installation and dynamic testing are presented in the Swedish Pile Commission report no.106.

6. QUALITY CONTROL, MONITORING AND TESTING PRACTICE

In Sweden all end-bearing piles, both driven and drilled (bored), should be verified for end-bearing resistance by a termination criterion. This criterion is determined or verified either via dynamic testing, WEAP-analysis or using a fixed termination criterion according to prescriptive measures (as outlined above).

7. PARTICULAR NATIONAL EXPERIENCES AND DATABASES

7.1. National experience regarding end-bearing piles

As mentioned earlier, the design value of pile resistance in Sweden is determined predominately by dynamic testing. However, for preliminary design of the pile group and for planning the load testing, values for the end-bearing resistance based on experience are needed. Recommended values for end-bearing resistance on rock based on Coates & Gyenge (1973) and on hard till based on Axelsson et al. (2004) are presented in the Pile Commission report no. 106, see table 7. The uniaxial compression strength q_u of non-fissured crystalline rock of granite or gneiss is normally in the region of 150-250 MPa. The mean end-bearing resistance that is possible to measure by dynamic load testing can be estimated using (see also table 7):

$$R_m = \sigma_b \cdot A \tag{16}$$

Furthermore, the following empirical equation can be used to estimate the maximum measured bearing capacity without exceeding the strength of the concrete piles during dynamic testing:

$$R_{m,max} = k_1 \cdot k_2 \cdot f_{ck} \cdot A \tag{17}$$

where:

k_1=0,75 on rock (experience value from dynamic testing).
k_1=0,70 on hard till (experience value from dynamic testing).

and:

k_2=0,8 according to SS-EN 12699.
k_2=0,8·1,1=0,88 if dynamic testing is performed on working piles, i.e the stresses are monitored during driving.

In the Pile Commission report no. 106 there is also k_1 values for steel pipe piles. The stress in the pile can also be calculated using wave-equation analysis (WEAP).

Table 7 : End-bearing resistance on rock and hard till (Pile Commission report no. 106)..

Soil/rock	σ_b (MPa)
Pile drilled into rock[1]	$5 \cdot q_u$
Rock shoe dowel D<150 mm, on rock[1]	$4 \cdot q_u$
Pile on rock[1]	$3 \cdot q_u$
Pile on hard coarse till	25-30
Pile on hard silty/fine sandy till	20

[1] Intact, non-fissured

7.2. Pile database

In Sweden, dynamic testing is generally accepted without performing any static load tests at the same site for the following cases:

- Driven end-bearing piles. Bearing capacity is determined by Case-method (or CAPWAP)
- Driven friction piles. CAPWAP-analysis is normally a requirement.
- Drilled steel pipe piles or steel core piles on rock, end-bearing. Bearing capacity is determined by Case-method (or CAPWAP).
- Drilled steel core piles, shaft bearing rock-socket piles (grouted). Bearing capacity is determined by the Wave-up method.

The database outlined in Likins (1996) is an accepted general reference for the correlation between static and dynamic load tests for the first two cases listed above. The latter two cases are piles in hard crystalline rock where a low set (< 1 mm/blow) is a requirement. In that respect, mobilization of the whole resistance has not occurred and therefore the measured capacity is considered to be conservative.

In a national database presented by Axelsson et al. (2004) the correlation between the Case-method and CAPWAP-analysis for driven end-bearing concrete piles is shown in figure 7. The correlation is very strong and explains the Swedish choice of using the same model factor for both CAPWAP–analysis and the Case-method with regard to end-bearing piles on hard till or rock, as shown in table 5 above.

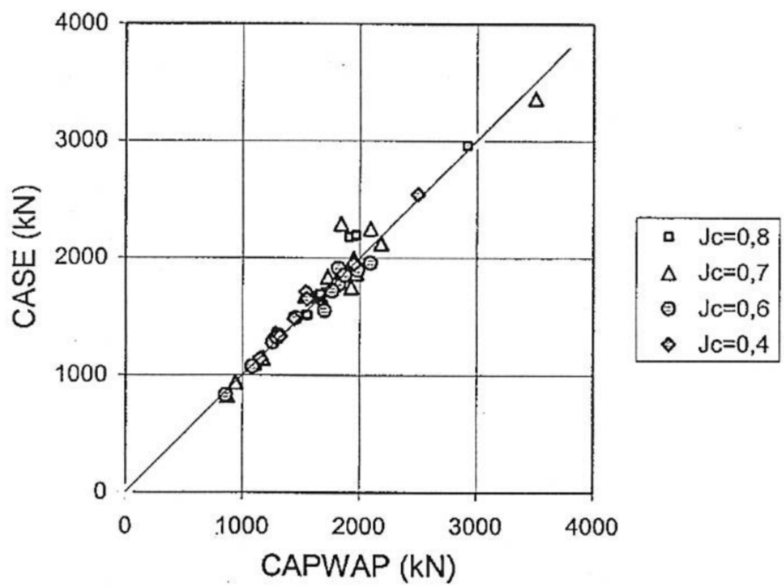

Figure 7: Correlation between CAPWAP and the Case-method for driven end-bearing piles according to Axelsson et al. (2004).

7.3. Cohesionless soil

In Sweden there are accepted correlations for CPT, dynamic probing DPSH-A (hejarsondering) and weight sounding WST (viktsondering) with regard to friction angle as shown in figure 8.

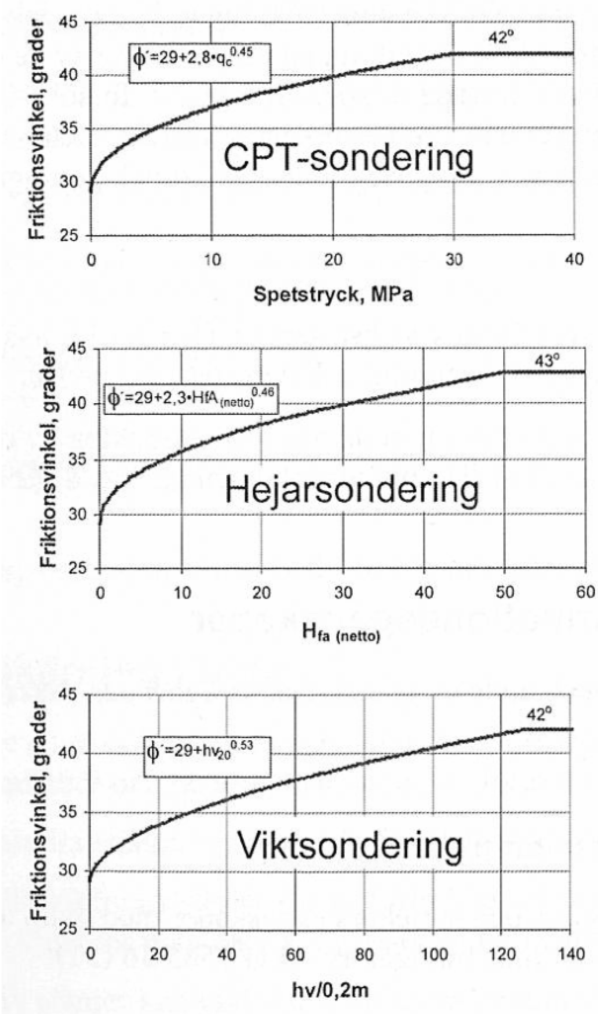

Figure 8: Correlation between penetration resistances obtained from CPT, DPSH-A and WST respectively and the friction angle of the soil (Trafikverket, TK GEO 13).

8. DESIGN EXAMPLE

8.1. Design of driven concrete end-bearing piles for limit state, GEO and STR

Design GEO

Four 270 mm square concrete piles (see fig. 2) were dynamic load tested for a bridge foundation consisting of a total of 16 driven end-bearing piles. The axial design load (design action) is E_d = 1200 kN according to the structural engineer. In Sweden pile foundations for bridges are normally treated as separate sites (piles within an area corresponding to 25x25 m). All the piles were driven by a Junttan pile driving crane with a 5 tonnes free-fall hammer. The piles were driven using a fall-height of 0,4 m until termination (refusal) and the last settlement (set) per 10 blows was measured. The piles were terminated in hard till.

The four test piles were dynamic load tested at restrike with a fall-height of 0,8 m at 18 hours after installation. The bearing capacity was determined by the Case-method (using RMX with an assumed Case-damping factor of 0,7). No signal matching (CAPWAP) was performed since the set per blow was ≤ 2 mm and the quake was evaluated to be less than d/60= 5 mm. Table 8 provides the results from the dynamic testing. The concrete according to the specifications should have strength of at least grade C40/50 at the time of installation. This means that the maximum compression stress during driving and restrike should not exceed 0,8x40 = 32 MPa according to execution standard SS-EN12699 (displacement piles).

Table 8 : Results from dynamic load testing.

Pile no	Pile length (m)	Termination, set per 10 blows	Test blow, set	Max compression stress	Bearing capacity Case-method
4	14,1	7 mm	1 mm	26,6 MPa	2037 kN
6	17,5	9 mm	1,5 mm	25,5 MPa	2010 kN
9	13,8	10 mm	2 mm	24,2 MPa	1920 kN
12	12,9	7 mm	1,5 mm	26,3 MPa	1995 kN

The mean value from the dynamic load tests is R_{mean} = 1990 kN, with a minimum value of R_{min} = 1920 kN. Using the correlation factors and model factor from table 4 and table 5 respectively, the characteristic bearing capacity of the piles is calculated as:

$$R_k = min\left\{\frac{R_{mean}}{\xi_5 \cdot \gamma_{Rd}}; \frac{R_{min}}{\xi_6 \cdot \gamma_{Rd}}\right\} = min\left\{\frac{1990}{1{,}55 \cdot 0{,}85}; \frac{1920}{1{,}45 \cdot 0{,}85}\right\} = 1510\ kN$$

The design value of pile bearing capacity is then calculated as:

$$R_{d(GEO)} = R_k/\gamma_t = 1510/1{,}2 = 1258\ kN$$

The termination criterion for the other piles in the group (not load tested) should be max 7 mm/10 blows as this was the smallest observed set for the tested piles.

Design STR

The average undrained shear strength (from two vane shear test profiles) over the buckling length L_c=5,7 m according to eq. (12) is c_u=9,0 kPa. The coefficient of variation (COV) is normally assumed to be 15 % for homogeneous normal consolidated clays. The initial deflection for a standard concrete pile with a joint is assumed to be $\delta_0 = L_c/150 = 0{,}033\ m$.

The design value of the undrained shear strength is calculated using equation (9), with $\eta_1 \cdot \eta_2 = 0{,}9$ according to figure 5 and $\eta_6 = 1{,}05$ since the foundation structure can, to a degree, redistribute load between weak and strong piles (all other η-factors are chosen as 1,0):

$$c_{ud} = \frac{\eta \cdot c_u}{\gamma_{cu}} = \frac{0{,}9 \cdot 1{,}05 \cdot 9}{1{,}5} = 5{,}7\ kPa$$

Approximately 30 % of the load is variable (short time) and 70 % is permanent load. A weighted average bedding modulus is calculated as:

$$k_d \cdot d = (0{,}7 \cdot 50 + 0{,}3 \cdot 200) \cdot 5{,}7 = 542\ kN/m^2$$

Using the equations (11) through (15) the structural capacity can then be calculated by an iterative process (not shown here):

$$F_{cd}^{2nd} = 1353\ kN$$

Overall pile capacity

The overall pile capacity is the smallest value obtained from GEO and STR:

$R_d = min\{R_{d(GEO)}, F_{cd}^{2nd}\} = \{1258, 1353\} = \mathbf{1258\ kN}$ (> E_d = 1200 kN, OK)

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