NORDIC GUIDELINE FOR REINFORCED SOILS AND FILLS

Y. Rogbeck, G. Franzén
Scandiaconsult/Ramboll, Sweden

C. Alén, K. Odén
Swedish Geotechnical Institute, Sweden

A. Kjeld
Byggros, Denmark

E. Øiseth
SINTEF, Norway

ABSTRACT: This paper presents a Nordic Guideline for reinforced soils and fills, published by the Geotechnical Societies in the Nordic countries and Nordic Industrial Fund in December 2003. The guideline includes chapters on materials and testing, design, execution, quality control and procurement. There are requirements on materials and testing but they are only shortly described in this paper. The guideline gives recommendations on design and constructions for the following applications: vertical walls and slopes, embankment on soft soil, embankments on improved soil and soil nailing. The guideline is applying a limit state format approach using partial factors of safety in the design. The design is based on the existing Eurocodes and National Application Documents. The paper gives the principles of design according to partial factors of safety for actions and combinations of actions, conversion factors for material properties and interaction coefficients. Brief introductions to the design procedures are given in the paper for some of the applications. Embankment on improved soil is presented in detail. Execution, quality control and procurement are shortly described.

1 INTRODUCTION

The Nordic Geosynthetic Group (NGG) has initiated the project with the guideline. NGG is organised by the Nordic Geosynthetic Societies. It has been financed by 29 organisations including Nordic Industrial Fund, the national Road- and Railroad Administrations in Sweden and Finland, the Road Administration in Norway, Swedish Geotechnical Institute, consulting- and construction companies and also producers. The authors to this paper have prepared the guideline together with A. Watn, SINTEF in Norway and H. Rathmayer, VTT in Finland.

The use of soil reinforcement technique has been increasing during the past decade in the Nordic countries. In Norway reinforced walls started earlier and are more common than in the other Nordic countries. In Sweden piled embankment is a common soil improved method and during the last years it has been commonly combined with reinforcement in the fill.

The purpose of the guidelines is to increase the knowledge of reinforced soil and to make it easier to use these type of constructions in the Nordic countries. The applications included are:

- vertical walls and slopes
- embankment on soft soil
- embankments on improved soil
- soil nailing

2 MATERIALS AND TESTING

The chapter about materials and testing in the guideline is based on the European Standardisation work. It's recommended that a test duration of 10000 hours is used for the EN-ISO 13431 creep test. It's always preferable if long time performance of the material is available from tests but conversion factors are given if some data is missing. Performance data is especially important for creep and damage during installation.

3 PRINCIPLES OF DESIGN

For the design of building and civil engineering works a set of harmonised technical rules are established by the Eurocodes. For the guideline at hand, it is Eurocode 1, version ENV 1991-1, Basis of design and actions on structures and Eurocode 7, version ENV 1997-1, Geotechnical design that give the guidelines for the design. From a practical point of view it would be preferable to refer to the EN-versions of the Eurocodes, which are due to replace the ENV:s. However, for the guideline this has been impossible since they only exist as provisional versions, prENs, which have no legal validity.

3.1 *Limit state design/Partial factors*

In recent decades it has become mandatory in structural design to verify structures in two different limit states, ultimate limit state (ULS) and serviceability limit state (SLS). As this design concept combined with partial factoring is incorporated in the Eurocodes, the outlined description of reinforced soil is based upon these two principles. In an ultimate limit state design a low probability of failure is demanded. This must be reflected in the choice of partial factors. In a serviceability limit state design deformations are normally concerned. Partial factors are normally set to unity, i.e. the calculations of the deformations are based on the characteristic values. However, there are no restrains in using partial factors larger than unity in serviceability limit state design to achieve a higher quality of the structure.

Of special interest when using the technique of reinforced soil is that materials of different deformation behaviour have to work together. Especially, mobilisation of geosynthetic reinforcement requires a certain amount of deformation. The basic principle should be to combine strength values of the materials at compatible deformation levels, see Figure 1.

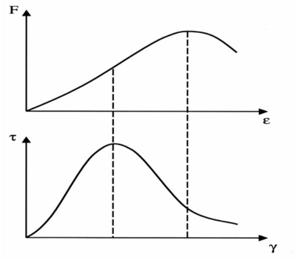

Figure 1 Example of compatible deformation levels. Geosynthetic reinforcement (top figure) compared to surrounding quick clay (bottom figure)

3.2 *Design values of actions*

ULS-values of partial factors in Eurocodes, 1.35 for permanent actions and 1.5 for variable actions, include model uncertainty. At least the first value applied to soil density and density of water, give raise to tricky situations in geotechnical engineering, e.g. the unit weight of water becomes ≥ 1350 kN/m^3. Hence the guidelines are restricted to design case C in ENV 1997-1, which states a partial factor 1.0 for permanent actions and 1.3 for variable actions.

3.3 *Design values of geotechnical parameters and reinforcement properties*

Characteristic values of material properties for both the soil and the reinforcement shall be based upon the results of laboratory or field testing. Design values are then derived from characteristic values by the equation, in ENV 1991-1:

$$X_d = \eta X_k / \gamma_M \qquad (1)$$

where η is a conversion factor for test results to site conditions and γ_M a partial factor for remaining material uncertainty. The conversion factor is normally not applied to geotechnical properties but here used to incorporate geosynthetic design practice in a general Eurocode format. Hence the factor is build up of several factors considering

- creep behaviour
- installation damage and
- chemical degradation and biological degradation

The procedure for the evaluation of the design strength of the reinforcement is outlined in Figure 2. If sufficient test data of long term performance is not available, the evaluation can be based upon the reduction factors given in "Guide to durability" (CEN CR ISO 13434:1998), i.e. $\eta_1=1/F_{cr}$ etc. For design of soil-nails the value of η take into account the number of field tests.

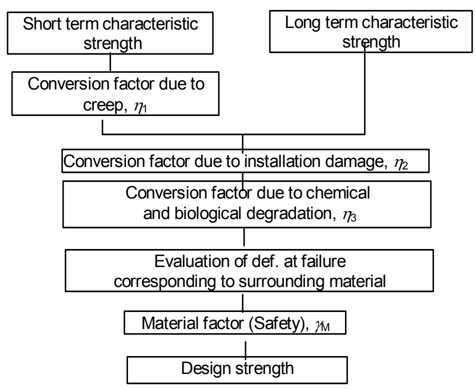

Figure 2 Scheme for determining of the design strength of geosynthetic reinforcement, c.f Eq 1

4 VERTICAL WALLS AND SLOPES

All design chapters in the guideline are discussed from the following view:

- Specific information needed for design
- Ultimate limit state design
- Serviceability limit state design
- Drainage
- Facing
- Durability

By defining principals in relation to design, execution and control it has become possible to put special attention to the problems related to the vertical wall and slopes.

Learned by experience it has succeeded to define general guidelines, which gives new participant in the geosynthetic world a tool to understand and relate the understanding to traditional geotechnical knowledge.

Due to the fact that to many designs still are being worked out by the supplier of the geosynthetic in not only Scandinavia, special attention has been given to improve the understanding of simple design methods although more sophisticated methods normally will be chosen in practise.

Basic hand calculation methods are therefore implemented and the aim of the guideline seems to fullfill in regard of giving maximum flexibility in the design stage.

As the definition of the geotechnical assumption often are directly involved in failures seen in this type of structures, not only in Scandinavian, it has been important to define a number of parameters which must be incorporated in the design.

By connecting the geotechnical investigation, design, execution and control into one unit, a more complete and better understanding are expected throughout the existence of this guideline.

Examples showing the use of safety principals, based on partial factors of safety have been incorporated in order to educate and to promote the use of ENV-7 and related National Application Documents in the Scandinavian countries.

5 EMBANKMENT ON SOFT SOIL

5.1 *Reinforcement effects*

Soil reinforcement may be used to increase the bearing capacity of embankments on soft subsoil. The purpose of the reinforcement is to resist the shear stresses from the embankment (lateral sliding of embankment) and possibly also shear stresses from the subsoil (extrusion/squeezing).

The use of reinforcement may commonly increase the bearing capacity in the range of 30-50 % depending on the type of subsoil (maximum theoretical limitation is 83 % when the total horizontal force component is taken as tensile force in the reinforcement).

5.2 *Calculation method*

The calculation method described is similar to the method in British Standard BS8006 (1995). This method calculates the required tensile strength for the reinforcement as the sum of the force calculated from lateral sliding and foundation extrusion stability. Also the force required to achieve sufficient stability for critical shear circles are calculated. The required tensile strength is the larger of either

- The sum of the forces required for laterals sliding and foundation extrusion stability.

or

- The force required for shear circle stability

The anchoring of the reinforcement at the edges (the bond length) should be checked for the lateral sliding force, the extrusion force and all critical shear circles close to the edge.

5.3 *Time dependency*

The design lifetime and consolidation should be considered when calculating the reinforcement design strength. The most critical situation is generally at the completion or shortly after completion of the construction works. Consolidation will over time increase the subsoil strength and therefore result in less required reinforcement strength as illustrated in Figure 3. Foundation settlement can increase the tensile strain and hence the load, in the reinforcement. The long term settlements therefore may offset a reduction in reinforcement load due to consolidation and an increase in embankment stability.

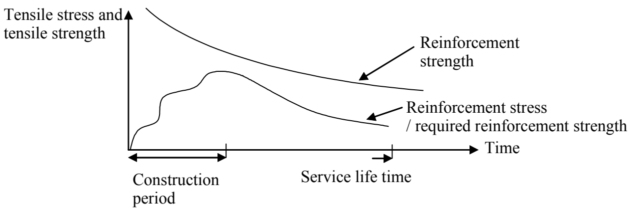

Figure 3 Required and actual reinforcement strength

6 EMBANKMENT ON IMPROVED SOIL

Reinforcement above lime cement columns may have two functions. For soft columns, which are most commonly used, the function is to prevent sliding. Calculations could then be done almost according to Embankment on soft soil with a complement for the lime cement columns resistance to sliding. The load is carried both by the columns and the soil between the columns. The difference in displacement of the columns and settlements in the soil will be small, leading to a small strain in the reinforcement and the effect will be very low for the function vertical load shedding. In the design of lime cement columns both settlements and sliding have to be considered. If the settlements are dimensioning to the space of the columns and this gives a safe construction from stability point of view, then a reinforcement is not necessary.

For stiff columns and reinforced piled embankments the function can be both to prevent settlements of the embankment and to prevent sliding. In this case the function is the same as for piled embankments and calculations could be done according to this chapter.

6.1 *Failure modes*

The ultimate limit states is to be considered for pile group capacity, pile group extent, overall stability of the piled embankment, vertical load shedding onto the pile caps and lateral sliding stability of the embankment fill. Pile group capacity, pile group extent and the overall stability considerations should be dealt with according to national regulations. Lateral sliding is only relevant if vertical piles are used beneath the embankment slope. In this chapter the lateral sliding and vertical load shedding are calculated.

6.2 *Restrictions of the model*

The calculation assumes arch formation between the pile caps and that the reinforcement is deformed during loading. The model is based on reinforcement placed in one layer, but an approximation is given for reinforcement in two layers. The function of the reinforcement is ideally if it is placed closest to the pile caps, but it should for practical reasons be about 0.1 m above the pile caps. In order to ensure that the displacements in the road surface won't be too large, the embankment height should be at least as large as 1,2 · the distance between the pile caps. The degree of cap coverage should be at least 10 %. In Sweden the cap coverage has been around 20-25 % where this method has been used.

The model has a wedge top angle of 30º and the strength in the reinforcement has shown to be comparable with results from finite element calculations when the friction angle of the fill is 35º. For higher friction angles the needed strength in reinforcement is lower than calculated in this model.

It is recommended that calculations should be carried out for an initial strain of maximum 6 % and with a remaining creep strain after the construction period and during the lifetime of the construction of an additional 2 % at the most. The strain has to be checked for the specific product and compared with the design strength at chosen strainlevel. The total strain should not during the design lifetime, exceed more than 70 % of the strain at failure for the reinforcement used.

If more than one layer of reinforcement is considered or lower embankment heights than the restriction, it is recommended finite element calculations to analyse the design with.

The analytical calculation model proposed is judged reasonable if there is a risk of cavities arising under the reinforcement, *e.g.* a future change of the load situation by ground water lowering. In design with the proposed analytical model, the foundation support of the soil between the pile caps is not taken into account, but the effect can be considerable. If more complex situations are considered more economical solutions can be done if finite element calculations are used to model the complex interaction behavior.

6.3 *Design of horizontal force*

If vertical piles are used beneath embankment slope instead of inclined piles, the tensile force in the reinforcement can be calculated as the active soil pressure:

$$T_{ds} = P_a = 0.5K_a\left(\gamma_d H + 2(q_{Qd} + q_{Gd})\right)H \qquad (2)$$

$$K_a = \tan^2(45 - \frac{\phi_d}{2}) \qquad (3)$$

where

H embankment height including tolerances (m)
γ_d design value for unit weight of fill/soil (kN/m^3)
ϕ_d design value of friction angle (°)
q_{Gd} , q_{Qd} design value of surcharge load (kPa)

6.4 *Design of vertical load transfer*

The method is based on the formation of an arch, which spreads the soil load onto the pile caps. The cross-sectional area of the soil under the arch, which is the load carried by the reinforcement, is approximated with a soil wedge described in Figure 4. This applies even if the embankment height is lower than (c-a)/2 tan 15°, which is the height of the soil wedge. The initial strain in the reinforcement of a maximum of 6 % should be used, or maximal allowed strain level to ensure creep strain < 2 %.

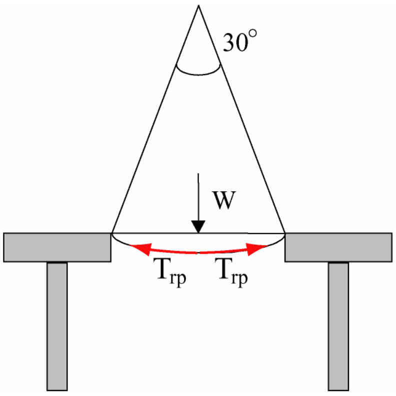

Figure 4 The soil wedge, which is carried by the reinforcement

The weight of the soil wedge, W, according to Figure 4 (in kN per metre in length):

$$W_{2D} = \frac{(c-b)^2}{4 \cdot \tan 15°} \cdot \gamma_d = 0{,}93(c-b)^2 \cdot \gamma_d \tag{4}$$

where

c centre distance between piles including tolerances (m)

b pile cap width including tolerances (m)

The three-dimensional effects are estimated through load distribution according to Figure 5 where the load is distributed over the surface according to the figure and is taken up by the reinforcement along the edge of the pile cap. The reinforcement transfers the load to the pile caps. The weight of the soil in three dimensions, W_{3D}, is calculated as follows:

$$W_{3D} = \frac{1+\frac{\varepsilon}{b}}{2} \cdot W_{2D} \tag{5}$$

The arc length of the reinforcement when it is displaced by the load of the soil wedge, can be calculated as follows:

$$s = (1+\varepsilon)(c-b) \approx c - b + \frac{8}{3}\frac{d^2}{c-b} \tag{6}$$

where the displacement, d, is dependent on the chosen strain in the reinforcement, ε, according to:

$$d = (c-b)\sqrt{\frac{3}{8}\varepsilon} \tag{7}$$

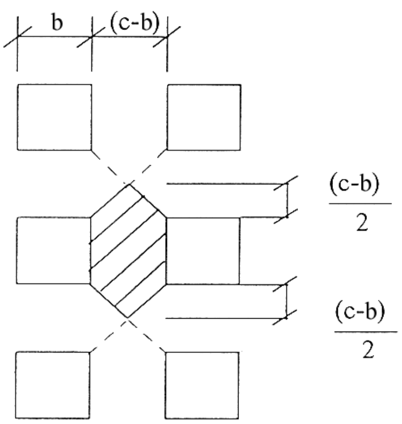

Figure 5 Load distribution to estimate the forces in the three-dimensional case

The designer should decide if the calculated displacement is acceptable. Normally an accepted strain gives an acceptable displacement. For the cases in Sweden where reinforcement has been used in piled embankments, the displacement has been calculated to be in the order of 0.1-0.2 m. If reinforcement is combined with stiff columns the displacement might be larger than 0.1-0.2 m for acceptable strains. There are no practical experience in the Nordic countries to judge if larger displacement can be tolerated.

The force in the reinforcement due to the vertical load in three dimensions, $T_{rp\ 3D}$ according to Figure 4 and Figure 5, is calculated using the equation:

$$T_{rp3D} = \frac{W_{3D}}{2} \cdot \sqrt{1 + \frac{1}{6\varepsilon}} \tag{8}$$

6.5 *Design of total force*

The force due to lateral sliding is assumed a plane case and the three dimensional behaviour is not calculated. The total force, T_{tot}, in the reinforcement is:

$$T_{tot} = T_{ds} + T_{rp3D} \tag{9}$$

If a force from stability calculation showed to be greater than T_{rp3D} the T_{tot} = a force from stability calculation added to T_{ds}.

In the calculations the strength of the seam has to be considered.

6.6 *Design of reinforcement*

The design of the reinforcement is done almost as in the British Standard 8006 (1995). The differences are the partial factors of safety. Two principles apply when designing the reinforcement:

- during the design life of the structure the reinforcement should not fail in tension
- at the end of the design life of the structure strains in the reinforcement should not exceed a prescribed value

The design strength of the reinforcement, T_d, should be the lowest of the following:

$$T_d = T_{cr} \cdot \eta_1 \cdot \eta_2 \cdot \eta_3 \tag{10}$$

or

$$T_d = T_{cs} \cdot \eta_1 \cdot \eta_2 \cdot \eta_3 \tag{11}$$

where

T_{cr} the peak tensile creep rupture strength at the appropriate temperature

T_{cs} the average tensile strength based on creep strain considerations at the appropriate temperature

η conversion factors

The design strength of the reinforcement should be greater than the total needed strength according to the calculations, $T_d > T_{tot}$. The calculation model is based on one layer of reinforcement. If two layers of reinforcement are used, it's recommended to place them close to each other, but not on top of each other due to loss of friction. The distance could for example be 0.1 m. The desired design strength can approximately be chosen as 40 % more than calculated for one layer. If a more economical solution should be achieved with the two layers finite element calculations are recommended.

Depending on the embankment height the reinforcement's frost durability has to be considered. Design of transverse sliding across the bank and pull-out of the reinforcement have to be done.

7 SOIL NAILING

The main aspects of design are similar for an excavated wall with nails and a natural slope reinforced with soil nails. However, there are some differences and therefore the two cases have been treated separately in the guideline.

For the two cases the following aspects are discussed in the guideline:

- Specific information needed for design
- Ultimate limit state design
- Serviceability limit state design
- Drainage
- Facing
- Durability

7.1 *Specific information needed for design*

The design of the soil nailed structure is based on information about the soil, ground water conditions, loads, wall geometry and soil nail system.

Information about the soil layering and properties of each layer is important for the design. In the guideline the necessary information for each design step is summarised.

7.2 *Ultimate limit state design (ULS)*

The failure in a limit state analyses may occur due to failure in the soil (stability of the slope, pullout of the nail or bearing capacity failure below the nail) or failure of the nail (tension, shearing and bending failure).

The final soil nailed wall will act as a gravity wall and consequently the same failure modes that is relevant for a reinforced wall may also be applicable for a soil nailed wall (Bearing capacity below the wall, tilting, sliding and overall stability).

Limit stage design includes the following;

- Preliminary layout of the soil nails and choice of soil nailing system
- Stability analyses
- Verification of the chosen soil nailing system
- External and overall stability
- Stability analyses of each excavation phase.

In the guideline the different aspects of the design is discussed thoroughly. In this paper only the stability analyses based on a partial factor approach is further discussed.

7.2.1 *Stability analyses*

Instead of applying a traditional global factor of safety format, the Nordic Guideline recommend an approach based on a partial factor format. The limit equilibrium equation that should be fulfilled may be expressed as follows.

$$\frac{1}{\gamma_{Rd}}\cdot\left[\frac{c_k'}{\gamma_{c'}}\text{"+"}\frac{\tan\phi_k'}{\gamma_{\phi'}}\text{"+"}\eta\cdot\frac{R_{N_k}}{\gamma_{R_N}}\right]-\gamma_{S_d}\cdot\left[\gamma_\gamma\cdot\gamma_k\text{"+"}\gamma_G\cdot q_{Gk}\text{"+"}\gamma_Q\cdot q_{Qk}\right]\geq 0$$

In the specific case of soil nailing traditional slope stability analyses is used. Partial factors are applied to all parameters and the calculation is performed aiming for F = 1.0. Symbolical the equation can be written as follows. (The actual equation depend on the analyse method used; *e.g.* Bishop, Morgenstern and Price, Janbu)

$$F=\sum_i\frac{\dfrac{c_{k_i}'}{\gamma_{c'}}+\left(\sigma_{N_i}'+\Delta\sigma_{N_i}'\right)\dfrac{\tan\phi_{k_i}'}{\gamma_{\phi'}}+\eta\cdot\dfrac{T_{k_i}}{\gamma_T}}{\gamma_{S_d}\cdot\left[\gamma_\gamma\cdot\gamma_{k_i}\text{"+"}\gamma_G\cdot q_{Gk_i}\text{"+"}\gamma_Q\cdot q_{Qk_i}\right]} \quad (13)$$

c'	cohesion of the soil,
σ'_N	effective normal stress
$\Delta\sigma'_N$	increase in effective normal stress perpendicular to the failure surface due to the nail force normal component P_N
T_k	increase in shear resistance du to the component P_p of the nail force parallel to the shear surface.
$\gamma_{c'}$	partial factor for cohesion intercept
$\gamma_{\phi'}$	partial factor for soil friction
γ_T	partial factor related to the natural variation in pullout capacity of a soil nail depending on the soil characteristics and nail characteristics. c.f. Chapter 2
$\gamma_\gamma, \gamma_Q, \gamma_G$	partial factor for action,
γ_{Sd}	model factor

The above equation is solved using a classic method of slices incorporating the forces of the nail in those slices where the nails intersect the failure surface. The maximum nail force, which could be mobilised, is determined considering pullout failure due to lack of friction between the nail and soil (both in active and resisting zone) and failure of the nail.

7.3 *Serviceability limit state design (SLS)*

The movement of the crust of the soil nailed wall depends on a number of factors. A low global factor of safety tends to give greater movement. If the ratio between nail length and wall height (*H*/*L*) is great the wall tilts more outwards. Other factors that influence are the rate of construction, height of excavation phases and spacing between nails, extensibility of nails, inclination of nails and bearing capacity of the soil below the wall.

For structures where movement of the wall is acceptable it may be sufficient to estimate the deformation based on empirical correlation. For more sensitive structures a more thorough study of the deformation might be necessary which could be accomplished by application of Finite Element.

7.4 *Facing and Drainage*

These chapter of the guideline is based on the recommendation that can be found in the prEN 14490 Execution of special geotechnical work – Soil nailing.

7.5 *Durability*

The requirements on a corrosion protection system depend on the environment, type of nail and consequences of failure. In the guideline a methodology for how to determine the necessary level of protection is suggested.

The environment is classified into three different environmental classes depending on the soil nails potential for corrosion in the certain environment.

First in step 1 a preliminary classification of the environment is made based on known facts from the site. If this preliminary classification indicates that the environment has low corrosion potential, a not too rigorous corrosion protection system can be chosen. On the other hand if the preliminary classification shows a normal to major corrosion potential, additional investigations should be made in step 2. Finally in step 3 additional factors effecting the environment is evaluated and the final environmental class determined.

In step 4 factors depending on the chosen soil nail system and consequences of failure is combined with the known environmental class to determine the necessary corrosion protection system. The proposed system is based on similar systems in Clouterre (1991) and in an article presented by Bergdahl (1986).

8 EXECUTION

This chapter in the guideline is based on the information in the drafts of the European execution standards for Reinforced fill (prEN 14475) and Soil Nailing (prEN 14490). Additional information from other standards and handbooks has been incorporated. (*e.g.* British standard, Clouterre, FHWA).

For Reinforced Fill the following aspects are discussed:

- Selection of material
- Reinforcing element material
- Materials for facing and connections
- Site conditions and site investigation
- Foundation
- Drainage
- Facing
- Selection, placement and construction of fill material
- Installation of reinforcing elements and connections.

For Soil nailing the following aspects are discussed:

- Preliminary work
- Excavation and face preparation
- Nail installation
- Drainage installation
- Facing installation

9 QUALITY CONTROL

This chapter in the guideline includes three subsections; supervision, testing, and monitoring. As for the chapter about execution the Nordic guideline is based on information from the draft of the forthcoming European execution standards.

GENERAL INFORMATION	TYPE OF CONTRACT C[27] Employer	Contractor	D AND C[28] Employer	Contractor
REQUIREMENTS:				
On the final function of the structure	(x)[29]		x[30]	
Level of safety	(x)		x	
On the geometry of the structure	x		x	
Esthetical aspects of the construction	x		x	
Permissible deformations of the construction during service life	x		x	
Loading (permanent, temporary and dynamic)	(x)		x	
Service life (temporary, permanent)	(x)		x	
Requirement on measurement equipment, type and quantity	x		x	
Regulations and standards applied	x		x	
Type of construction if there is any requirements or wishes from the employer	x		x	

Comments:
27) Construction contract
28) design and construct contract
29) (x) the employer have this information and may provide the contractor with it
30) x party marked with x should provide the opposite party with information

Figure 6 Example of table in Chapter 9

10 PROCUREMENT

This chapter in the guideline discuss the effect of the chosen type of contract on procurement, the contents of the tender document and the responsibility for different activities. In this guideline only the two main types of contracts in their original form is considered "design and construct" and "construction" contract.

The main objective of the chapter is to raise the questions that need to be considered for execution of the work. The responsibility for different activities may vary between different projects and the same information requires to be considered for the execution but the responsibility of providing or obtaining the information will fall upon either party depending on which contract type be chosen.

The information that needs to be considered and the distribution between the parties with respect to obtaining or providing the information, for respective type of contract, is presented in a number of tables. Additional tables with suggestion of who should be responsible for which activities are also included, *c.f.* Figure 6.

11 CONCLUSION

The paper gives an introduction to the Nordic Guideline for reinforced soils and fills. It's based on Eurocode 1 and 7. The structures are verified in ultimate limit state (ULS) and serviceability state (SLS). ULS-values of partial factors of safety in Eurocodes, 1.35 for permanent actions and 1.5 for varable actions, include model uncertainty. At least the first value applied to soil density and density of water, give raise to tricky situations in geotechnical engineering, e.g. the unit weight of water becomes ≥ 1350 kN/m^3. Hence the guidelines are restricted to design case C in ENV 1997-1, which states a partial factor 1.0 for permanent actions and 1.3 for variable actions.

Testing is based on the European standardisation work. Execution and quality control are based on the European execution standards.

The complete guideline in English can be downloaded from the Swedish Geotehnical Societies homepage www.sgf.net.

12 REFERENCE

Bergdahl, U., 1986: Corrosion on hot galvanised power line pylons – a study of different investigation and evaluation methods, Proceedings 10th Scandinavian Corrosion Congress, Stockholm, pp 317- 322.

British Standard (1995), "Code of Practice for Strengthened/reinforced soils and other fills", BS 8006:1995

Draft prEN14475 Execution of special geotechnical work reinforced fill, Draft prEN 14490 Execution of special geotechnical work Soil Nailing

ENV 1991-1: Basis of design and actions on structures - Part 1: Basis of design

ENV 1997-1: Geotechnical design - Part 1: General rules

Franzén G., 1998: Soil Nailing – A Laboratory and Field study of Pull-Out Capacity, Doctoral Thesis, Department of Geotechnical Engineering, Chalmers University of Technology, Gothenburg, Sweden

Nordic Guideline for reinforced soils and fills (2003). Download from www.sgf.net

Schlosser, F., Plumelle, C., Unterreiner, P., Salencon, J., Magnan, J.P., 1991: Recommendations Clouterre 1991, French national research project Clouterre, FHWA-AB-93-026