NORDIC GUIDELINES
FOR
REINFORCED SOILS AND FILLS

Nordic Geosynthetic Group
A section of Nordic Geotechnical Societies
Nordic Industrial Fund
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Photo on the front page	Reinforced wall — Anders Kjeld Reinforced slope — Anders Kjeld Soil Nailing wall — Gunilla Franzén Embankment on improved soil — Hartmut Hangen

PREFACE

These Nordic guidelines for strenghtened/reinforced soils and fills is published by the Geotechnical Societies in the Nordic countries and the Nordic Industrial Fund.

The Nordic Geosynthetic Group (NGG) has initiated these guidelines. The group is organised by the Nordic Geotechnical Societies. NGG has been the project management group. The guidelines have been prepared by a project group. It has been financed by 29 organisations including the companies for the project group. The other organisations have been engaged in the reference group. The large reference group has been valuable to gain approval to the content in the guidelines.

The book is only gives guidelines and the designers have full responsibility. . Engineering judgement should be applied to determine when the recommendations are relevant to every specific object. The guideline is intended for engineers with experience of geotechnical works.

The purpose of these guidelines is to increase the knowledge of reinforced soil and to make it easier to use these types of structures. The reinforced soil structures are often more economical than conventional structures. The applications included in these guidelines are:
- Vertical walls and slopes
- Embankment on soft soil
- Embankment on improved soil
- Soil-nailing (excavated walls and natural slopes)

The guidelines include chapters on:
- Materials and testing
- Design
- Execution
- Quality control
- Procurement

The use of soil reinforcement technique has been increasing during the past decade in the Nordic countries. In Norway the use of reinforced walls started earlier and is more common than in the other Nordic countries. In Sweden piled embankment is a common soil improvement method and during the last years it has been commonly combined with reinforcement in the fill. Some guidelines already exist in some of the Nordic countries but the purpose with these guidelines is to use partial factors in the design according to Eurocode.

The guidelines concerning design of reinforced fill are mainly for the use of polymeric reinforcement but some guidelines are also given for stiffer reinforcements.

The work with Eurocodes is still in progress. ENV 1991-1, Basis of Design and Actions of Structures is going to be a norm soon. The difference between existing ENV 1997-1, Geotechnical Design and prEN 1997-1 is considerable. For these guidelines existing ENVs have been chosen as the base. When the Eurocodes have been approved as ENs these guidelines might need a revision depending on the outcome of the ENs. National

Application Documents (NAD) are made as a complement to the ENVs and are different in the Nordic countries. Concerning these guidelines there are only NAD ruling in Sweden and Norway and figures from these are refereed in Annex B.

The intention with the guidelines is that all design work should be able to calculate by hand. For more complex objects or a faster procedure there are computer programs on the market, but in these guidelines there are no recommendations made to existing programs. They have to be evaluated by the designers.

To get a better understanding of the consequences of using partial factors it would be useful to make consequence analyses. However, this is not included in this project.

These guidelines is written in English to be able to get feed back also for persons outside the Nordic countries. The guidelines are translated to Norwegian and Swedish during 2003 and published by the Geotechnical societies in Denmark, Norway, Sweden and Finland in corporation with the Nordic Industrial Fund.

The project organisation is presented below.

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ANNEX

1 INTRODUCTION

1.1 SCOPE

The Nordic Guidelines for strengthened/reinforced soils and other fills are guidelines with recommendations for different applications. The guidelines are based on a limit state approach using partial factors of safety in the design. The base of the design is existing Eurocodes, ENV 1991-1 "Basis of Design and Actions of Structures" and ENV 1997-1 "Geotechnical Design". As a complement to these pre-standards National Application Documents (NAD) are made in countries where other opinions are ruling than what is given in the ENVs. As to these guidelines there are NAD for Sweden and Norway with other instructions concerning the different designs, see Annex B.

The applications in the guidelines, illustrated in Figure 1.1 are:
1. Vertical walls and slopes
2. Embankment on soft soil
3. Embankment on improved soil
4. Soil-nailing (excavated vertical wall and natural slope)

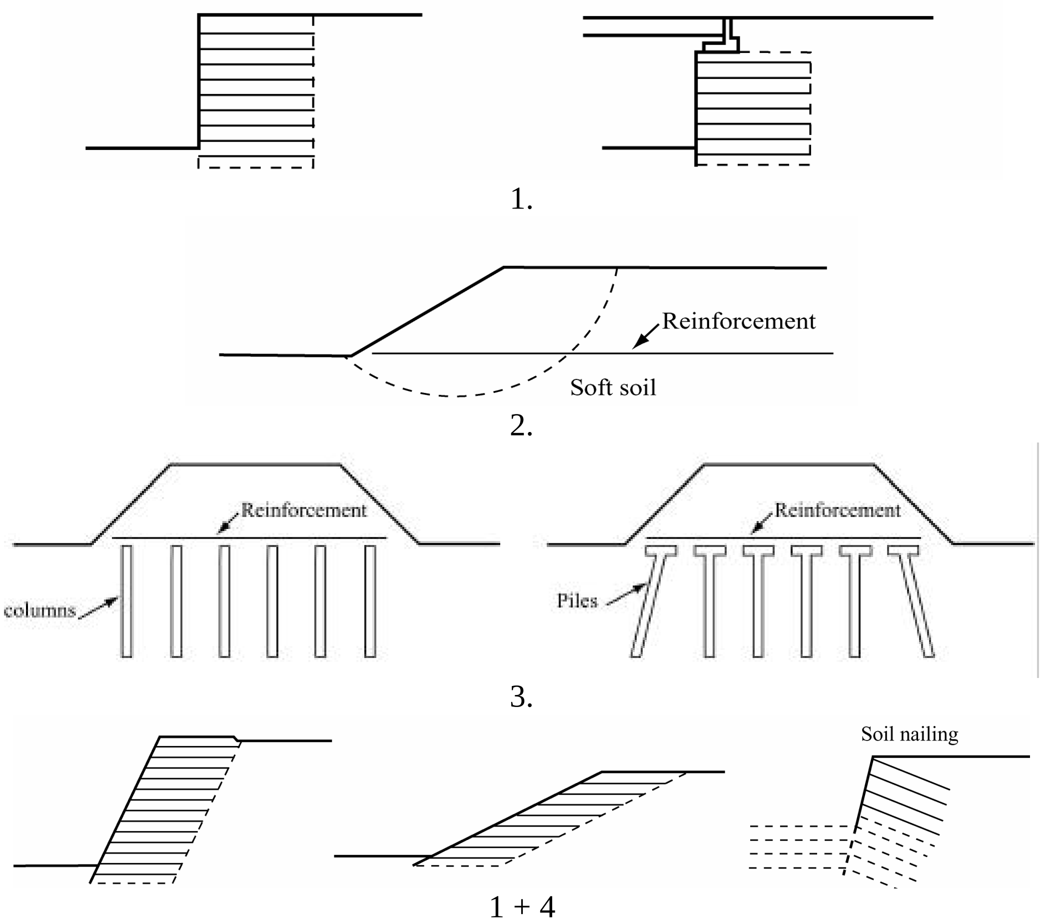


Figure 1.1 Applications described in the guidelines.

The guidelines deal with materials and testing, design, execution, quality control and procurement.

Chapter 1 contains introduction with the scope, notations and symbols.

Chapter 2 gives an introduction to the materials used in reinforced soil applications: natural soil, fill and reinforcement. Test methods for different materials are shortly described and references are made to different standards. Values from testing procedures according to standards are preferable, but there are tables presented giving general values depending on type of polymer and fill.

Design procedures are given in **Chapter 3-6**. For all applications the following procedure is used: introduction, function of reinforcement, specific information needed for design, limit state design with failure modes, restrictions of the model and design step by step.

Principle of design is described in **Chapter 3** and it is recommended that the designer read this chapter before starting the design. **Chapter 3** is the base of the design of the different applications. This chapter contains partial factors of safety for actions and combinations of actions as well as for different material properties according to ENV 1991-1, ENV 1997-1 and different NAD, conversion factors for material properties and interaction coefficients. No back analyses have been made to decide the size of the partial factors of safety, this has to be made by the authorities in the different Nordic countries.

Chapter 4 deals with horizontal reinforcement in walls and slopes. It includes retaining walls, bridge abutments and sound barriers.

In **Chapter 5** the design method for reinforced embankments on soft soil is given. For reinforced embankments on improved soil **Chapter 6** describes a method for piled embankments and some recommendations are also given for deep stabilisation.

Design methods for soil-nailing are divided into excavated vertical slope and natural slope in **Chapter 7**.

Execution is divided into reinforced fill and soil-nailing in **Chapter 8**. This chapter as well as quality control in **Chapter 9** are based on pre standards for the CEN work done by TC 288, named execution of work.

A recommendation on requirements to be included in the tender documents and advise on procurements are given in **Chapter 10**.

References from all chapters are listed in **Chapter 11**.

For the different applications design examples are given in respective Annex.

1.2 INTRODUCTION OF THE METHODS

1.2.1 Reinforced steep slopes and walls

1.2.1.1 Principal

By introducing horizontal layers of reinforcement into a structure it is possible to stabilise and reinforce the fill.

Since the early eighties a number of projects have been constructed using reinforced fill.

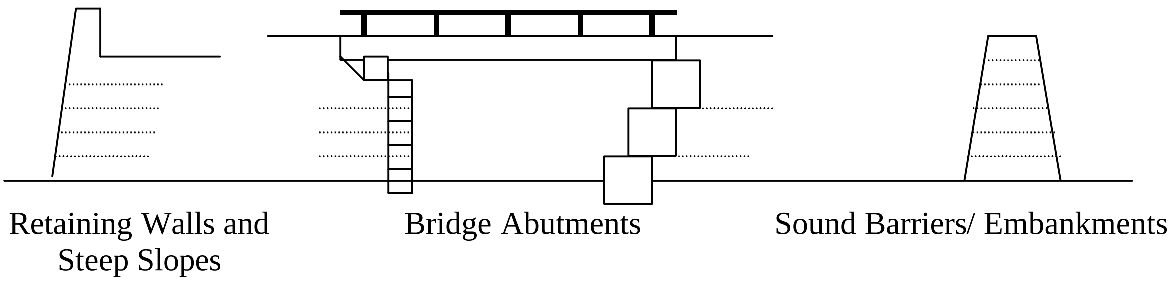


Figure 1.2 Typical applications of reinforced fill

Walls and Abutments normally cover applications of Reinforced Soil with a slope angle between 70 - 90 degrees, while Steep slopes cover slopes less than 70 degrees.

1.2.1.2 Main application

Reinforced fill very often appears to be an economically attractive way of constructing walls, abutments, embankments, sound barriers, steep slopes *etc.*

1.2.2 Embankment on soft subsoil

1.2.2.1 Principal

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).

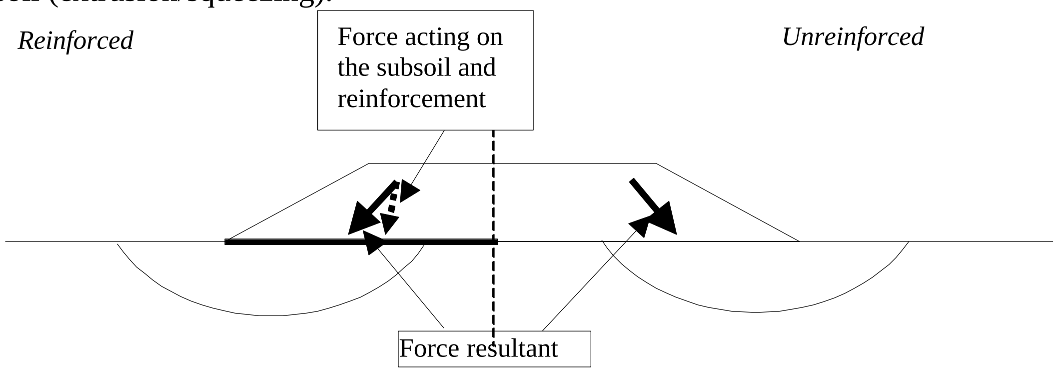


Figure 1.3 The effect of the reinforcement in an embankment on soft soil

1.2.2.2 Main application

The main application is construction of road and railway embankments on soft subsoil.

1.2.3 Embankment on improved soil

1.2.3.1 Principal

For embankments on improved soil reinforcement may be used in the fill in the lower part of the embankment.

Reinforcement above lime cement columns may have two functions. For soft columns the function is to prevent sliding. For stiff columns the function can be both to prevent settlements of the embankment and to prevent sliding, the same was as it works for reinforced piles.

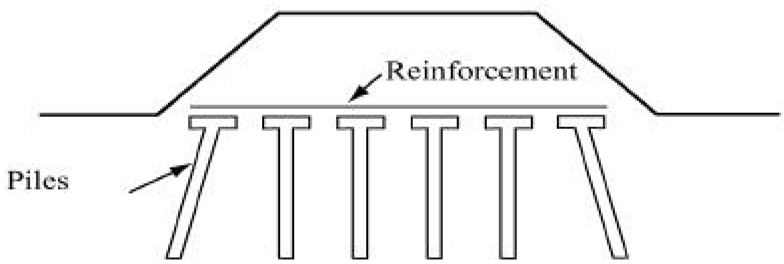


Figure 1.4 Embankment on improved soil

1.2.3.2 Main application

The main application is in this case reinforcement of piled embankment.

1.2.4 Soil-nailing

1.2.4.1 Principal

Soil-nailing is a technique to increase the stability of existing or newly excavated soil structures, by installation of relatively slender passive reinforcing bars in the soil. At a small movement of the active zone of the slope the reinforcement will experience both axial and lateral displacement with respect to the soil. This displacement will generate forces in the nail

- Tensile forces will be generated due to the axial displacement. Either the maximum tensile capacity of the reinforcement or the maximum soil friction that may be mobilised between nail and soil limits the maximum tensile force.
- The lateral displacement will result in lateral stresses against the reinforcement and is limited by the soil bearing capacity. This lateral displacement may result in shear forces and a bending moment in the nail, the magnitude depending on the nail stiffness and the inclination of the nail.

The soil-nailed slope can be divided into two zones;

- The active zone where the friction forces along the nail are directed towards the facing and therefore have a tendency of pulling out the reinforcement.
- The resisting zone, where the frictional forces are directed inwards into the slope, hence preventing outward movement of the nail and, as a consequence, also of the active zone.

At a small movement of the active zone the reinforcement in the resisting zone will be activated, hence preventing the movement of the active zone. Due to the interaction between the soil and the nail, a ”reinforced body” is created, which essentially works as a gravity wall stabilising the unreinforced soil behind. Figure 1.5.

To retain the soil between the nails at the very front, some sort of facing is necessary. Often a shotcrete facing or a geotextile is applied.

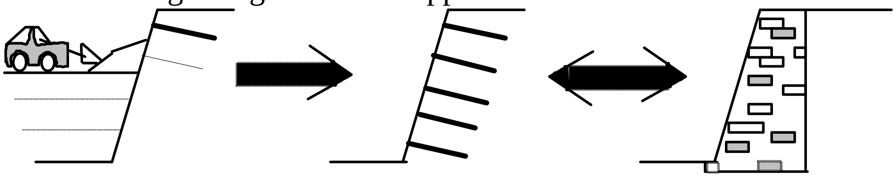


Figure 1.5 Principle of Soil-nailng

To visualise how the soil-nails affect the slope, the method is sometimes compared to a tree on a slope. The roots of a tree will retain the soil, creating a reinforced body consisting of roots and soil. Hence the interaction between the soil and roots results in a stable slope.

1.2.4.2 Definition

There is not one single definition of soil-nailing in the literature; instead a number of different proposals exist. However, to limit the scope of these guidelines the following definition of soil-nailing, based on suggestions from several authors, is used.

A soil-nail is a small diameter reinforcing element, passively installed in a slope, with a typical installation angle of 10° - 45° to the normal of the potential failure surface, and therefore working mainly in tension, with shearing/bending forces as a possible but negligible secondary effect.

Other definitions include both dowels and rigid nails installed more or less perpendicular to the failure surface and hence consider the shearing resistance as a significant contribution to the reinforcing effect of the soil-nails.

1.2.4.3 Main application

The method has two main applications:

- Increasing the safety of failure of natural slopes
- Construction of steep slopes by stepwise excavation and installation of nails

1.3 NOTATIONS AND SYMBOLS

Definitions and symbols used in these guidelines are taken from the ruling Eurocode ENV 1991-1 and 1997-1 as well as drafts of prEN 14475 for reinforced fill and prEN 14490 for soil-nailing.

1.3.1 Definitions

1.3.1.1 General

Action:

a) Force (load) applied to the structure (direct action).
b) An imposed or constrained deformation or an imposed acceleration caused for example, by temperature changes, uneven settlement.

Permanent Action (*g*): Action which is likely to act throughout a given design situation and for the variation in magnitude with time is negligible in relation to the mean value.

Variable Action (*q*): Action which is unlikely to act throughout a given design situation or for the variation in magnitude with time is neither negligible in relation to the mean value nor monotonic.

Representative value of an Action: Value used for the verification of a limit state.

Characteristic value of an Action: The principal representative value of an action.

Design value of an Action F_d: The value obtained by multiplying the representative value by the partial factor of safety γ_F.

Action effect: The effect of actions on stuctural members, *e.g.* internal force , moment, stress, strain.

Construction works: Everything that is constructed or results from construction operations.

Type of Construction: Indication of principal structural material, *e.g.* reinforced concrete construction, steel construction.

Construction material: Material used in construction work, *e.g.* concrete, steel, geosynthetics.

Execution: The activity of creating a building or civil engineering works.

Design criteria: The quantitative formulations which describe for each limit state the conditions to be fulfilled.

Design situations: Those sets of physical conditions representing a certain time interval for which the design will demonstrate that relevant limit states are not exceeded.

Transient Design situations: Design situation which is relevant during a period much shorter than the design working life of the structure and which has a high probability of occurrence.

Persistent Design situations: Design situation which is relevant during a period of the same order as the design working life of the structure.

Accidental Design situation: Design situation involving exceptional conditions of the structure or its exposure, *e.g.* fire, explosion, impact or local failure.

Design working life: The assumed period for which a structure is to be used for its intended purpose with anticipated maintenance but without substantial repair being necessary.

Load case: Compatible load arrangements, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular verification.

Limit states: States beyond which the structure no longer satisfies the design performance requirements.

Ultimate Limit states (uls): States associated with collapse, or with other similar forms of structural failure.

Serviceability Limit states (sls): States which correspond to conditions beyond which specified service requirements for a structure or structural element are no longer met.

Maintenance: The total set of activities performed during the working life of the structure to preserve its function.

Characteristic value of a Material property X_k: The principal representative value of a material property.

Reliability: Reliability covers safety, serviceability and durability of a structure.

Resistance: Mechanical property of a component, a cross-section, or a member of a structure, *e.g.* bending resistance.

Strength: Mechanical property of a material, usually given in units of stress.

1.3.1.2 Reinforced soils and fills (definitions specific to these guidelines)

Bearing plate: a plate connected to the head of the soil-nail to transfer a component of load from the facing or directly from the ground surface to the soil-nail.

Drainage system: a series of drains, to control surface and ground water.

Facing: a covering to the exposed face of reinforced fill. The facings are divided into three groups depending on their characteristics.

Hard Facing: a rigid covering, generally in the form of precast concrete sections, which may be structurally connected to the reinforcement. A facing with no capacity to accommodate differential settlement between fill and facing.

Flexible Facing: a flexible covering which prevents breaking out and sliding off of soil and rock material from between the nails or reinforcement layers, which has to fulfil a static function and depends substantially on the ground conditions and the arrangement of the reinforcement/nails. A facing with capacity to accommodate differential settlement.

Soft Facing: a very flexible facing that may be formed by extending a full width reinforcement sufficiently to encapsulate the face of the fill. This is often called a wrapped facing. Such facings are often used seeded to establish vegetative cover. A facing with capacity to accommodate differential settlement.

Fill: A natural or man made particulate medium, including certain rocks, used to construct engineered fill.

Reinforced Fill: Engineered fill incorporating discrete layers of soil reinforcement, generally placed horizontally, which are arranged between successive layers of fill during construction.

Randomly Reinforced Fill: Engineered fill incorporating randomly oriented soil reinforcement in the form of continuous polymeric filaments or discontinuous elements which are placed simultaneously with the fill during construction.

Ground: Soil, rock and fill existing in place prior to the execution of the construction works.

Reinforcement or reinforcing element: Generic term for reinforcing inclusions inserted into ground or incorporated into fill.

Fill Reinforcement: A reinforcement, typically in the form of a strip, sheet, rod, grid, mesh or filament, usually placed in discrete layers, which enhances stability of the reinforced fill mass by mobilising the axial tensile strenght of the fill reinforcement.

Soil-nail: A reinforcing element, installed into the ground, usually at a sub-horizontal angle, that mobilises friction along its entire length with the soil.

Sacrificial nail: a soil-nail installed using the same procedures as production nails solely to establish pull-out capacity and is not to be used in final design

Test nail: a nail installed by the identical method as the production nails, for the purpose of testing to establish/verify pull-out capacity.

Production nail: a soil-nail which forms part of the completed soil-nail structure.

Soil-nail system: consists of a reinforcing element and may include the following main parts, joints and couplings, centralizers, spacers, grouts and corrosion protection.

Soil-nail installation procedure: the process of inserting soil-nails into the ground typically by one of following methods; driven, ballistic, percussive or vibratory, drilled, grouted and placed or simultaneously drilled and grouted.

1.3.2 Units recommended for geotechnical calculations

forces	kN, MN
moments	kNm
mass density	kg/m^3, Mg/m^3, (t/m^3)
unit weight	kN/m^3
stresses, pressures and strengths	kN/m^2, kPa

1.3.3 Symbols

LATIN UPPER CASE LETTERS

A	Accidental action
A	Characteristic cross-sectional area of a soil-nail
B	Width
C	Characteristic circumference of the hole for the Soil-Nail
C_u	Coefficient of uniformity
D	Diameter

E	Action effect, general term
F	Action, Axial or transverse loads on pile/soil-nail
G	Permanent action
H	Horizontal action or force
H	Effective retained height or height of embankment
K	Earth pressure coefficient
L	Length
M	Safety margin
N	Bearing resistance factor
P	Resulting earth pressure
Q	Variable action
R	Resistance
R_N	Shear strength of reinforcement/soil-nail
R_u	Coefficient of pore water pressure
S	Stiffness of reinforcement
S	Spacing of the nail
T	Action effect when reinforced soil
T	Tensile strength
T	Pull-out capacity of a soil-nail
V	Vertical action or force
V	Coefficient of variation
W	Weight
X	Material property

Latin lower case letters

a	Adhesion
b	Pile cap width
c	Shear strength
c	Centre distance between pile caps
c_u	Undrained shear strength
c’	Cohesion intercept in terms of effective stress
d	Deformation
d	Displacement
k	Permeability
n	1/n slope inclination
p	Vertical pressure
q	Overburden or surcharge pressure
q_s	Pull-out resistance of a soil-nail
r_u	Coefficient of pore water pressure
s	Settlement
u	Pore water pressure (lateral stresses)

Greek lower case letters

α_i	Coefficient of interaction
α_i	Sensitivity factor
β	Reliability index
δ	Angle of shearing resistance between ground and structure
f	Angle of shearing resistance = friction angle
ϕ’	Angle of shearing resistance in terms of effective stress
g	Unit weight

g	Partial factor
g	shearing
γ_A	Partial factor for accidental actions
γ_f	Partial factor for actions
γ_F	Partial factor for actions, also accounting for model uncertainties and dimension variations
γ_G	Partial factor for permanent actions, also accounting for model uncertainties and dimension variations
γ_m	Partial factor for a material property
γ_M	Partial factor for a material property, also accounting for model uncertainties and dimension variations
γ_Q	Partial factor for variable actions
γ_{rd}	Partial factor associated with the uncertainty of the resistance model and the dimensional variations
γ_R	Partial factor for the resistance, including uncertainty of the resistance model and the dimensional variations
γ_{Rd}	Partial factor associated with the uncertainty of the resistance model
γ_{Sd}	Partial factor associated with the uncertainty of the action and/or action effect model
e	Strain
h	Conversion factor
m	Coefficient of friction
θ	Perimeter
σ	Total normal stress
σ'	Effective normal stress
t	Shear stress
x	Reduction factor
x	Increase in shear strength per meter depth
$\psi 0$	Coefficient for combination value of a variable action
$\psi 1$	Coefficient for frequent value of a variable action
$\psi 2$	Coefficient for a quasi-permanent value of a variable action

SUBSCRIPTS

G	Permanent action
Q	Variable action
a	Active earth pressure
ax	Axial
cr	Creep rupture
cs	Creep strain
d	Design value
e	Effective
ext	Extrusion
h	Horizontal
k	Characteristic value
o	At rest
o	Initial condition
p	Passive earth pressure
p	Pull-out

s	Side slope
s	Sliding
t	Tensile
t	Total
v	Vertical
w	Water

ABBREVIATIONS

GWT	Ground water level
CWT	Capillarity

1.4 TRANSLATION OF TEXT IN FIGURES

Figure 2.2

Töjning	Strain
Sekunder	Seconds
Dag	Day (24 hours)
År	Year

2 MATERIALS AND TESTING OF MATERIAL

This chapter 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). A paragraph that is indented marks text that is a quotation.

2.1 FILL REINFORCEMENT

Fill reinforcement in the broad sense encompasses a wide range of construction principles, which have the following major components:

- Fill reinforcement
- Fill material
- If required a facing system

The reinforcing, man-made elements are incorporated in the fill to improve its behaviour and thus to control the stability of the reinforced fill structure.

All material components shall be specified in the design, their material parameters determined according to the relevant European Standards and they shall meet the requirements of the E.U. Construction Products Directive 89/106/EEC. Several reinforced fill construction systems are commercially available and marketed in packages that contain design, specifications and all man-made materials necessary for the execution of the complete structure.

2.1.1 Reinforcement products

2.1.1.1 General

Reinforcing elements shall provide tensile strength to the fill material and are generally made of the following materials or combinations of these:

- Steel
- Polymeric materials and
- Fibre glass.

Other reinforcement materials may also be used. The reinforcing elements control the long-term stability of the structure, therefore their suitability and durability has to be assessed based on trials, experience or test data. It has to be proved that the specified properties of the reinforcing elements are valid for the whole design life of the reinforced structure.

2.1.1.2 Steel reinforcement

For steel reinforcement the life of the structure will depend on the corrosion resistance of the reinforcing elements, which in turn depend on their geometrical layout, the type of steel and mode of corrosion protection. Widely used types of steel reinforcement are linear elements like rods, strips, corrugated bars and ladders, or planar sheets as grids, woven wire mesh and welded steel mesh.

Nearly all steel reinforcing elements are made of durable grades of steel, which ensure a relatively uniform mode of corrosion at a predictable rate in moderately aggressive environment. Steel reinforcing elements may be provided with a protective coating (*e.g.* hot dip galvanising according to EN ISO 1461, with a local coating thickness of 70µm minimum) to mitigate the effects of electrochemical corrosion. Zinc-aluminium thermal spray coating may be applied to steel reinforcing strips for use in specific aggressive environments (type: Zn85Al15/70, 70µm local coating thickness according to requirements of ISO 22063). Polymeric coatings provide some corrosion protection but are susceptible to construction damage, which may reduce their effectiveness.

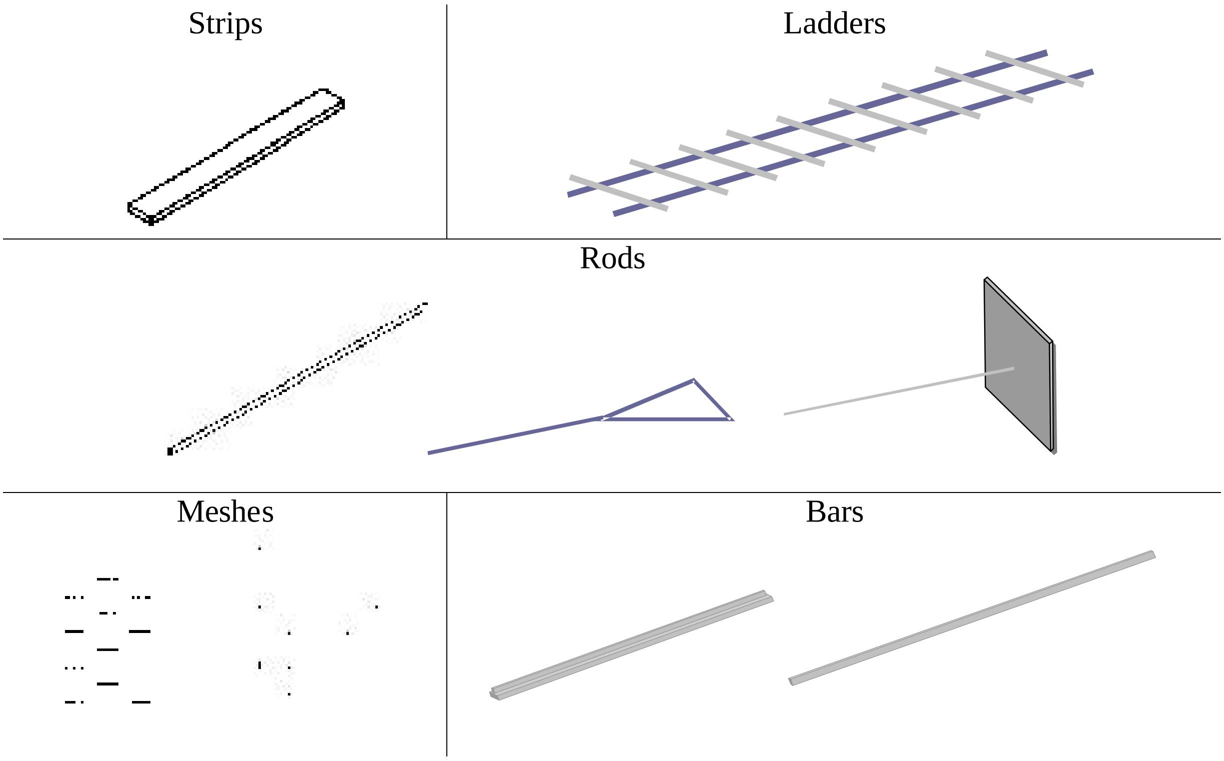


Figure 2.1 Steel Reinforcements (from prEN 14475)

Other metals, like stainless steel or aluminium alloys should not be used for permanent reinforced fill structures, unless their long-term corrosion resistance can be warranted.

Steel grades according to EN 10025 (Hot-Rolled Products of Non-Alloy Structural Steel), or EN 10013 (Hot Rolled Products in Weldable Fine Grained Structural Steels - Technical delivery conditions (part 1 - 3)) and suitable for galvanising (*e.g.* S235, S275, S355, S420 or S460) are recommended for steel strip reinforcement.

Cold drawn steel wire conforming to EN 10080 or hot rolled steel conforming to EN 10025 and EN 10113, welded into the finished reinforcing product in accordance with EN 10080 is recommended for welded steel wire mesh, grids or ladders. Rods and bars made of cold drawn steel wire shall conform to EN 10138, rods and bars made of hot rolled steel to EN 10025 and EN 10113. Nuts and bolts used to join steel reinforcements should comply with ISO 898-1.

For woven steel wire meshes made of cold drawn steel EN 10218 and accordingly EN 10223/ 3 apply. Hot dip galvanised coatings on wires for woven meshes should comply with EN 10244 and EN 10245 for extruded organic coating.

2.1.1.3 Geosynthetic reinforcement

The most commonly used polymers are polyester and polyolefins. Geosynthetic reinforcement is manufactured in the form of strips (one-dimensional), grids, meshes or sheets (two-dimensional), cell / honeycomb structures (three-dimensional) or fibres and filaments, see Figure 2.2. Polymeric strips are installed at predetermined vertical and horizontal spacing, grids or sheets are usually installed as full width reinforcement in which case only a vertical spacing is specified.

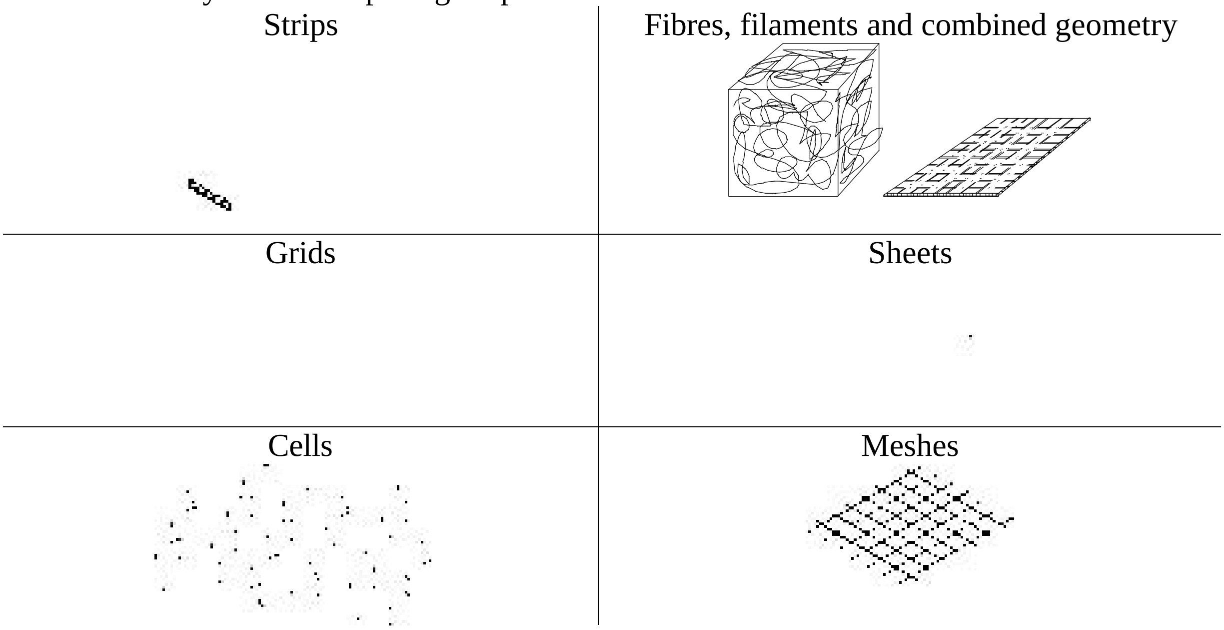


Figure 2.2 Polymeric Reinforcements (from prEN14475)

The force/strainrelation of the material in a geosynthetic reinforcement is a very important issue to take into account, see Figure 2.3.

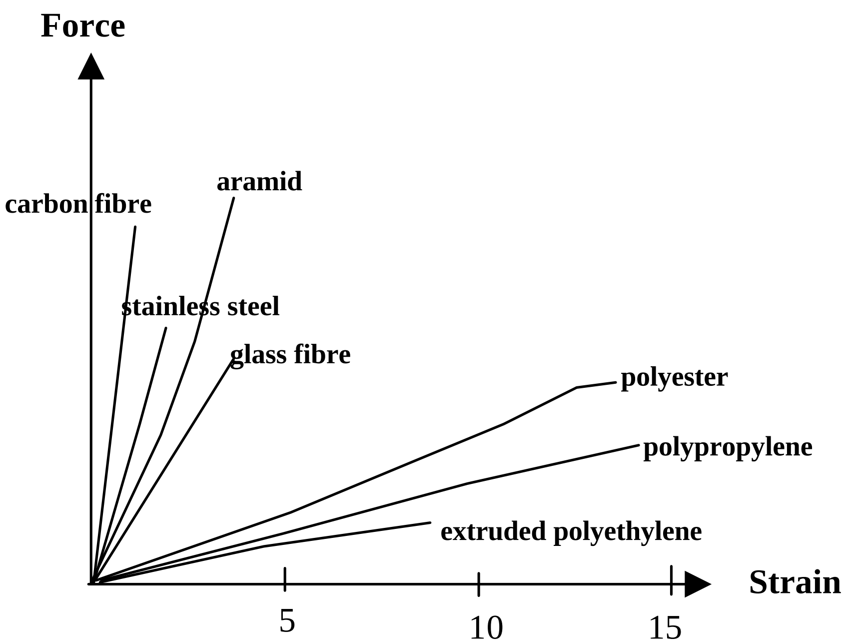
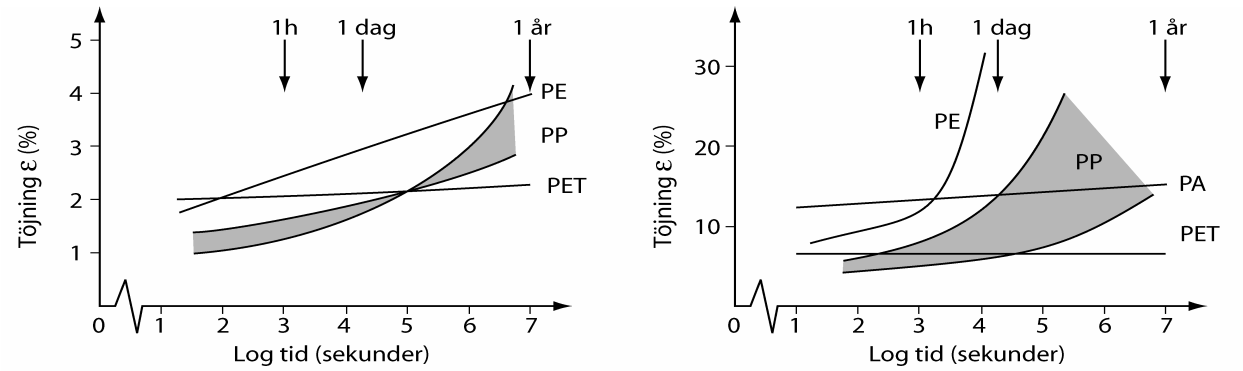


Figure 2.3 Typical strain-force behaviour of reinforcement (Carlsson, 1987)

The creep effect of the material is important as well as the great differences in creep behaviour between the different polymers, Figure 2.4. The figure shows that either polyethylene, polypropylene or polyester could be used at a 20 percent loading if the strains in the reinforcement are acceptable to the specific structure. The characteristics change dramatically to polyethylene and polypropylene at a 60 percent of the failure load and at those levels they are not suitable to use as they would brake. These polymeres could be used at smaller loads or if creep tests for the specific product are performed to show the maximum loadlevel at which the strain still is acceptable. The important issues when choosing the reinforcing material is the creep behaviour over time and load capacity.



a) creep at 20 % loading   b) creep at 60 % loading

Figure 2.4 Creep behaviour for different types of reinforcement (den Hoedt, 1986) PE-Polyethylene, PP-Polypropylene, PET-Polyester, PA-Polyamide (translation of text in figure in chapter 1.4 Translation of text in figures)

The required parameters to be determined for geosynthetic reinforcing elements are assessed in EN 13251 for the purpose of CE-marking. The CE-mark and its accompanying documents provide certified values (95 percent confidence limit) of *e.g.* tensile strength and load-strain characteristics. According to this standard certified values pertaining to the specified design life and environmental conditions of the reinforced fill structure shall be based on tensile creep (and creep rupture) as EN ISO 13431, construction induced damage as ISO 10722-1 and fill-reinforcement interaction as EN ISO 12957-1.

Aspects of durability (biological and chemical attack) shall be determined according to EN 12224, EN 12225, EN 14030, EN 12447 and EN 13438, resistance to weathering according to ENV 12224, see further below.

DURABILITY CHARACTERISTIC OF GEOSYNTHETIC

Geosynthetics may serve for temporary structures or may be needed temporarily until consolidation of foundation soils or fill material. Long-term application is the majority of applications, therefore durability is an important requirement and certified test values have to be given for the properties listed below:

- **Resistance to weathering (EN 12224: 2000)**
  Products exposed uncovered to light and products placed without cover-soil for some time are tested by artificial weathering. Exposure to UV-light of defined emission spectrum and rain at elevated temperature accelerates the test. After exposure the loss in tensile strength (in percent) when compared to reference specimen is determined.

- **Resistance to microbiological degradation (EN 12225: 2000)**
  Fungi and bacteria living in soils may attack the polymeric materials used as geosynthetics. In the test the product is buried in biologically active soil and after the “soil burial” test residual strength is measured.

- **Resistance to acid and alkaline liquids (screening test) (EN 14030: 2001)**
  From all chemical attacks two were selected for an accelerated screening test to evaluate resistance to hydrolysis for polyester and resistance to thermal oxidation for polyolefines. The results of this test give an indication of behaviour in acid and alkaline environments, but are not suitable to evaluate long term performance of the products.

- **Resistance to hydrolysis (EN 12447: 2001)**
  Hydrolysis of polyester is the reverse action of the crystallisation process and means connecting water molecules or parts of it to the polyester molecules. External hydrolysis by alcaline attack occurs also at low temperatures, internal hydrolysis in neutral environments is relevant at elevated temperatures. In the test products are immersed in liquids for times up to 90 days and residual strength and strain are tested.

- **Resistance to thermal oxidation (ENV ISO 13438: 2002)**
  To the molecules of polyethylene or polypropylene oxygen may be connected creating increased brittleness of the polymers. Stabilising additives and the reduced availability of oxygen in soil delay this oxidation. For the test the products are subjected to accelerated thermal oxidation and after the test the residual strength is determined. The retained strength should exceed 50 percent of the tensile strength of the reference samples.

2.1.1.4 Tensile creep tests

Tensile creep tests give information on time-dependent strain at a constant load. Loads for creep testing are most often dead weights.

Creep rupture tests give time until failure at a constant load. A strain measurement is not necessary for creep rupture curves. The EN-ISO creep tests require 1000 hours testing, for creep rupture extrapolation to long-term (30, 60, 120 years) a test duration greater than 10000 hours is necessary. Results are plotted for creep as linear deformation versus log-time, for creep rupture linear or log-stress level versus log-time. From creep curves at different stress levels isochronous stress strain curves may be derived for calculation of the structure’s deformation at a given time. Typical curves for a polyester product are shown in Figure 2.5.

The creep behaviour of geosynthetics depends mainly on the polymer used and how the base materials (yarns, tapes) are treated thermomechanically.

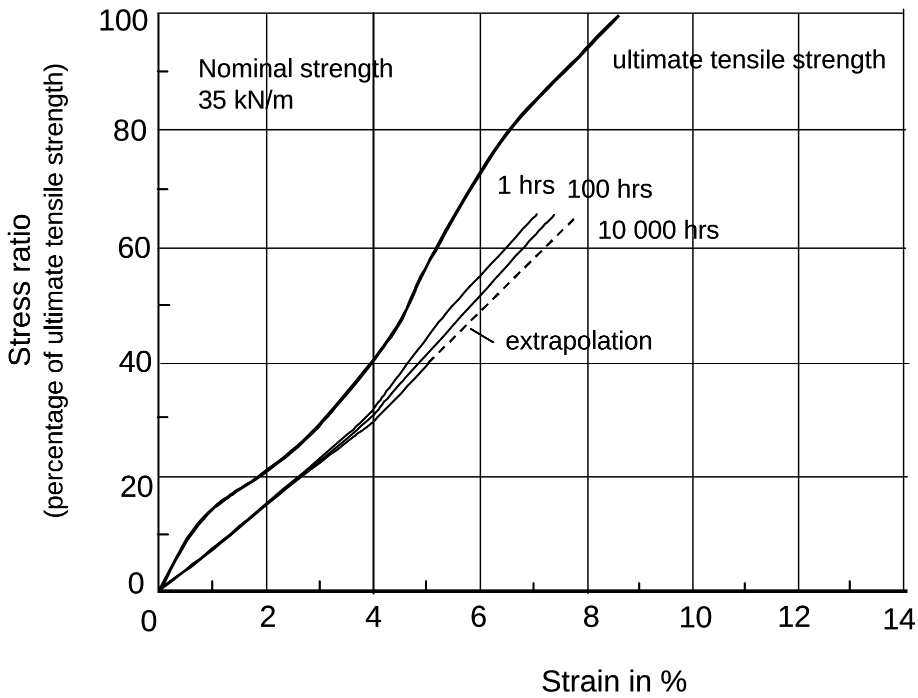


Figure 2.5 Isochronous curves of tensile creep tests. Examle for a polyester type.

2.1.1.5 Conversion factors applied to short term mechanical parameters

The allowable tensile force per unit width of the reinforcement usually depends upon the type and safety requirements of the reinforced structure, the stresses the geosynthetic reinforcement is exposed to, the work execution and on environmental factors. Because of these factors ultimate strength parameters determined from short term index tests are divided by several conversion factors to account for potential creep, installation damage and ageing.

According to discussions in Chapter 3 the following conversion factors, Table 2.1-Table 2.3 should be used. However, it should be noted that it is always preferable if long time performance of the material is available from tests, which means that higher values can be used.

Table 2.1 Conversion factors of geosynthetic reinforcements

Conversion parameter – material aspect	Conversion factor
Factor of creep (depending on lifetime),	η_1
Installation damage	η_2
Biological and chemical degradation	η_3

Table 2.2 Example on conversion factors, h_1[1], which account for long term properties based on short term test results[2].

Raw material	Conversion factor, η_1
Steel	0.8
Polyester (PETP)	0.4
Polypropylene (PP)	0.2
Polyamide (PA)	0.35
Polyethylene	0.2

Table 2.3 Example on material factors and following conversion factors, h_2[1], for damage during installation depending on the fill material in contact with the reinforcement[2].

	Clay /silt[3]	Sand	Gravel (Natural)	Gravel (Broken)	Crushed Rockfill
Material factor: F	1.1	1.2	1.3	1.4	1.5
Conversion factor: η_2=1/F	0.91	0.83	0.77	0.72	0.67

Experience shows that higher values of the material factor sometimes needs to be chosen than given in Table 2.3. According to FHWA publication NHI-00-043 the value of the material factor varies between 1.2-3.0 for fill with maximum grain size 100 mm and d_{50} = 30 mm. For fills with maximum grain size 20 mm and d_{50} less than 0.7 mm the value is 1.1-2.0. For critical structures it is advisable to perform tests.

The material factor for biological and chemical degradation, F_{env}, may according to the Swedish Road administration publication 1992:10 be assumed to 1.1 as long as the pH-value ranges between 4 and 9, which gives a conversion factor of η_3 = 0.91.

2.1.1.6 Interaction factors for soil/reinforcement friction

For the interaction factor for soil/reinforcement friction the following values may be used according to the publication by the Swedish road administration, 1992:10 and the publication from the Norwegian road administration, publication 016. In the Norwegian publication the values for clay/silt is not mentioned.

Table 2.4 Interaction factors for soil/reinforcement friction values, a, depending on soil conditions

Type of reinforcement	Soil type[4]				
	Clay, silt	Sand	Gravel (Natural)	Gravel (Broken)	Crushed Rockfill
Mesh, grid	0.8	0.9	0.95	1.0	1.0
Sheet	0.7	0.7	0.7	0.8	0.8

2.1.1.7 Values of the partial factors for pull-out and sliding

The partial factor for pull-out resistance, γ_p, is according to British standard 1.3. However British standard is based on a total safety approach and consequently a lower value might

[1] η_i is $1/F_i$ when comparing with Vägverket rapport 1992:10, Statens Vegvesen rapport 016
[2] If possible it is preferable to do tests instead of using the table
[3] Values for clay/silt is taken from Vägverket rapport 1992:10
[4] Fill with d_{50} < 1.5 times the width of geo grid should be used

be used when in this case partial factors are applied not only to the pull-out but also to the friction angle.

The partial factor for sliding resistance, γ_s, is also 1.3 according to British standard. With the same reasoning as for the partial factor for pull-out resistance the value might be taken lower than 1.3.

2.1.2 Fill materials

Selected fill material consists usually of naturally occurring or processed material, which is compactible. Reinforced fill structures allow the possibility of using also substandard soils or recycling products. This may lead to significant savings in hauling and construction costs. Purely cohesive soils are not generally accepted in the construction of reinforced soil structures for permanent works, due to low stress transfer between the soil and the reinforcement and due to their frost susceptibility.

Demands on the quality of the fill material are guided by the requirements set to the reinforced fill structure, such as bearing capacity, type and intensity of loads, allowable deformations, frost susceptibility and drainage efficiency. If seepage or infiltrating water cannot be drained by other means, the fill material has to be free draining, resist suffusion and degradation.

2.1.2.1 Fill internal friction and cohesion

The mechanical properties of the selected fill material are usually described in terms of internal friction and cohesion. These soil parameters shall be representative under the conditions in which the fill is used (*e.g.* density, moisture content, stress level) and determined from the weakest materials. For free draining or granular fill materials the relevant parameters may be derived from previous experience or determined on the basis of the fill gradation.

2.1.2.2 Fill reinforcement interaction

The reinforcing materials interact with the fill according to two different principles, friction and interlocking. Linear elements like rods, strips and corrugated bars, and also planar sheets function on the basis of intermaterial friction. Grids, meshes and honeycomb structures act also by interlocking with the frictional fill.

Parameters describing fill reinforcement interaction can be based on relevant testing procedures, *e.g.* shear box or pull-out test, but can also be assessed on the basis of previous experience.

2.1.2.3 Suitability (factors of influence for the selection of fill materials)

The suitability of a fill material for a reinforced fill structure depends on factors like constructability, environmental conditions, fill layer thickness, facing technology, vegetation cover, drainage arrangements, aggressivity, fill – reinforcement stress transfer, internal friction and cohesion and frost susceptibility.

2.1.2.4 Constructability

Selected fill material shall be suitable for the climatic conditions under which the fill will be placed, as well as for the compaction equipment and procedure, the local practice and experience. Fill materials are usually described with parameters obtained from index testing, as moisture content, modified (or normal) Proctor density, compressive strength and grain size distribution.

The fill constructability shall be such that it can be placed and compacted to produce the properties required by the design, usually 95 percent of modified Proctor density at optimum water content. The fill material shall be free from snow and ice. Frost susceptible materials shall not be placed during construction at winter conditions.

If the material is processed any additives used to improve its workability, *e.g.* lime, cement, shall be considered with regard to their compatibility with reinforcement layers within the fill, chemical durability and environmental limitations.

2.1.2.5 Facing technology

Facing systems for reinforced fill structures have specific tolerances in respect to compaction induced post construction settlement and wall deformation. Proper selection of the fill material and its adequate compaction is essential for keeping the movements within tolerance values.

Durability of both the facing and the reinforcing elements is important in the context of the typical 50 to 100 years service life expected of reinforced fill structures. The required durability in respect to freeze-thaw and wet-dry behaviour of *e.g.* block facings is obtained by using the proper cement and additives in the manufacturing process.

2.1.2.6 Vegetative cover

Vegetation covering the facing requires fill material suitable for plants near the front of the construction.

2.1.2.7 Environmental conditions and aesthetics

Reinforced fill structures can tolerate deformation and settlements to a certain extent. Excess deformation or settlements will not be tolerated at e.g. bridge abutments, walls supporting infrastructure and buildings, which have hard, flexible or block facings. If post construction settlement is critical from environmental or aesthetic aspects, easily compactable fill material with low compressibility should be selected.

Fine grained soils and degradable fill materials should not be used without assessing their strength and long-term properties from laboratory tests or trials to validate their use.
In zones of the reinforced fill structure prone to frost penetration fine grained, frost susceptible fill material shall not be used unless frost insulation is installed.

Free draining fill material should be used in such cases, where possible floods, fluctuating groundwater level and occasional access of run-off water to the structure may occur. The effective particle size (D_{10}) can be used to estimate the permeability of cohesionless fill material.

2.1.2.8 Layer thickness and maximum particle size

A uniform layer thickness of the loose fill material should not exceed 300 mm and the maximum particle size should be less than 2/3 of the compacted layer thickness. Factors restrictive to the maximum particle size are possible construction damage of the reinforcing elements and the demand for light-weight compaction equipment and thinner compacted layers close to the facing units. Both layer thickness and maximum particle size are also depending on the spacing of the layers of the reinforcement, and on type and size of the facing units.

2.1.2.9 Aggressivity of the fill

Relevant properties, which indicate a potential aggressiveness of the fill material or of the natural soil adjacent to the reinforced fill zone are: pH-value, Redox potential, electrical resistivity, salt content including sulphate, sulfides and chlorides. Data on the electrochemical, chemical or biological suitability of the selected fill material will help in the selection of durable reinforcing elements and necessary protective measures. Long-term properties may be assessed based on previous experience, which establishes correlation between relevant soil characteristics and the long-term mechanical parameters of the reinforcement.

Crushed, angular shaped fill material can be considered mechanically aggressive with regard to the reinforcement or facing. The risk for mechanical damage of the reinforcements, or of their protective coatings, caused by the selected fill material during construction may be assessed based on previous experience or on specific site testing. For steel reinforcement with polymeric coating against corrosion a reduced maximum particle size of the fill (*e.g.* < 20 mm, round shaped particles) is recommended.

2.1.2.10 Frost susceptibility

Frost susceptible material shall not be used in sections, where frost penetration and consequent frost heave might cause damage to the reinforced structure, e.g. behind the facing and at the foundation level. The fill material may be protected with frost insulation layers.

2.1.3 Drainage

2.1.3.1 Drainage properties

When using geosynthetics as drainage material the drainage and filtration properties of the geosynthetic should be compatible with the selected fill. If fines (silt, clay) are allowed in the reinforced zone or as backfill soil, any possible water in front, behind or beneath the reinforced zone must be carefully drained off. Proper filtration and drainage control is critical and any infiltration of surface water into the reinforced fill has to be prevented.

2.1.4 Facing

2.1.4.1 General

Facings are the visible part of completed reinforced fill structures and thus control their aesthetics. They protect the structure against loss of fill material and erosion and may provide a lining or drainage pathway and enable connection of reinforcing elements. A wide range of materials, configurations with a variety of reinforcement connections, joint fillers and bearing devices is available on the market. The type of facing strongly influences the deformation characteristics of the completed structure.

Facings systems, which include connections between facings and reinforcement and possible jointing materials, have to be designed such, that they can be constructed within specified tolerances of vertical and horizontal alignment and perform within specified deformation tolerances and without structural damage over the design life. The serviceability of the system should be verified by comparable experience.

Performance and service life of a vegetated facing system depend in addition to technical aspects on climatic and biological conditions of the reinforced fill site. Green plants have special demands to the backfill material at the facing, like moisture conditions and content of organic matter. Roots of plants may have detrimental effects on the reinforced soil structure.

2.1.4.2 Prefabricated concrete units

Prefabricated facing panels of concrete have usually a minimum thickness of 140 mm, tensile reinforcement and system for connecting the reinforcing elements to the panel and shear pins to adjacent panels. To achieve the required construction tolerances and durability the concrete panels should comply with ENV 206 and ENV 1992 and be free of cracks or defects at any stage of the construction. Both the materials used and the manufacturing tolerances are of great importance to the achievable construction tolerances and thus durability of the facing system.

Segmental block wall units of concrete are smaller in size and especially manufactured for reinforced fill applications. The concrete used for manufacturing such segmental blocks should comply with ENV 206. Like the facing panels the blocks are equipped with a system for connecting to the reinforcing elements and to adjacent blocks. Usually no mortar or other filler is used between the block wall units.

2.1.4.3 Steel facings

Materials for metallic facings and connections must be such that accelerated corrosion will not occur due to electrolytic action from contact between dissimilar metals.

Facing units manufactured of carbon steel meshes or grids may be hot-dip galvanized with a minimum average zinc coating of 70µm, unless stated by the producer, that less thickness of zinc coating combined with polymer coating will provide equivalent long term resistance. Facing units, if without coating, should be designed for a sacrificial thickness. Full sacrificial steel thickness shall be applied and access of soil moisture to the surface of the steel components shall be prevented at the connections. The facing and connections should be so designed that the stability of the reinforced fill structure will considered for the event of fire.

2.1.4.4 Geosynthetics facing units

Various types of geosynthetic reinforcement are wrapped around as a facing of reinforced fill layers or used as baskets to form a reinforced fill structure. Geosynthetic materials are susceptible to damage during installation, to degradation by ultraviolet radiation, to vandalism and to damage due to fire unless they are protected by shotcreting, concrete or wooden facing panels or vegetation. All geosynthetics materials used for the construction of wrapped facing units or gabion baskets, shall comply with pr EN 13251. In applications where the fire risk is high geosynthetic facing units shall only be used protected.

2.1.4.5 Normative references – summary

Facing systems shall comply with requirements of relevant European Standards, Table 2.5.

Table 2.5 Relevant Standards for Requirements on Facing Systems (from prEN 14475)

REQUIREMENTS	Facing systemsS					
	Concrete Panel	Segmental Block Wall	Welded Steel Mesh	Woven Steel Mesh and Gabions	Semi Elliptical Steel	Wrap Around
Concrete quality	ENV 206	ENV 206				
Steel reinforcement (in panel)	ENV1008/ ENV1992-1-1			EN 10223 part 3		
Dimension tolerances	X[5]	X[5]				
Compressive strength at installation	X[5]	X[5]				
Surface quality	X[5]	X[5]				X[5]
Steel quality			EN 10079/ EN10080	EN10218 part 1&2	EN10025	X[5]
Galvanising quality			EN ISO1461	EN10244 part 1&2 EN10245 part 1&2	ISO1461	

2.1.5 Testing of material

GEOSYNTHETIC REINFORCEMENT

Table 2.6 Characteristics needed for the use of geosynthetic reinforcements. Testing should be done according to relevant standards

Parameter	Explanation	Standard
Stress/strain parameters	Normally based on the short term strength	EN ISO 10319
Creep parameters	Related to the design lifetime of the structure	EN ISO 13431 [6]
Manufacture	Related to the production and experience with the product	CE market
Mechanical damage during construction	Related to grain size and form in connection with the construction	ENV ISO 10722-1 [7]
Chemical and biological resistance	Related to environmental effect, which might influence the characteristics of the product	EN 12224, EN 12225, EN 140030, EN 12447, ENV ISO 13438[8]

[5] Requirements are needed but no relevant standard is available
[6] For creep rupture extrapolation to long term (> 25 years) a test duration greater than 10 000 hours is necessary
[7] There are also other field tests that can be performed and can give values suitable for design
[8] The test methods are shortly described in Chapter 2.1.5

STEEL REINFORCEMENT

The following characteristics are needed for use of steel reinforcement:

- Stress/strain parameters
- Chemical and biological resistance (related to environmental effect, which might influence the characteristics of the product)
- Corrosion
- Coefficient of interaction (Defined as coefficient of interaction between the fill/soil and the reinforcement, see table 5.3.4.1)

Testing should be done according to relevant standards.

2.2 SOIL-NAILING

2.2.1 Main parts

A soil-nailed structure consists of the following main parts:

- Reinforcing element – soil-nail
- Natural soil
- Facing
- Drainage

In the following chapters material properties and testing of material properties for the main parts are discussed.

2.2.2 Reinforcing element

2.2.2.1 Different types of reinforcement

The reinforcement (the soil-nail) may be divided into two different groups: driven nail and grouted nail. The main difference is the installation technique (direct installed or drilled) and whether or not grout is included as a part of the reinforcement. The difference in installation technique is described in Chapter 7.

Driven nails are directly driven into the soil and consist commonly of a steel reinforcing element with different types of cross sections e.g. solid bar, hollow bar, or angle bar. Joints and coupling may be used for a long driven nail.

As the name indicates grouted nails consist of the reinforcing element and grout. As for driven nails joints and couplings may be used for longer nails.

2.2.2.2 Material properties for the reinforcing element

In these guidelines only material properties for steel members used as reinforcing elements are considered. Other materials may be used and relevant requirements should in such case be applied.

Depending on type of steel member the requirements in the following standards should be fulfilled, according to prEN 14490.

- EN 10080 for solid steel bars
- EN 10210 or EN 10219 for hollow steel bars
- EN 10025 or EN 10113 for hot rolled steel product
- EN 10138 for pre-stressed steel products

> If a “re-used’ reinforcing element or product is used it shall comply, with the requirements concerning type, size, tolerances, quality and steel grade specified in the design and be free from damage, deleterious matter and corrosion that would affect strength and durability. (Quotation from prEN 14490)

The required properties of the reinforcing element should be guaranteed during its entire design life. Consequently a complementary protection system could be necessary. A number of different types of protection systems exist. The simplest system is to use sacrificial thickness and for more severe conditions double protection system could be used. According to the future European execution standard hot dip galvanised coating and thermal-sprayed zink-aluminium alloy could *e.g.* be used and should then comply with the following standards.

- EN ISO 1461 galvanised, hot dip galvanised coating
- EN 22063 thermal-sprayed zinc-aluminium alloy

Section 2.2.6 gives a suggestion of how to choose the necessary corrosion protection based on known soil parameters and consequences of failure.

The characteristic value of the tensile strength for the reinforcing element is determined according European standard as the 2 percent fractal for the steel. According to the same standard partial factors are applied to determine the design tensile strength of the reinforcing element. The partial factor γ_m =1.0 – 1.1 could be applied for steel in tension. The shearing resistance is determined in a similar manner.

As a rule of thumb the following aspects could be considered when choosing the reinforcing element:

- Steel with ductile failure is preferable
- A high strength steel usually has a higher tendency for corrosion and commonly gives a brittle failure
- If the nail is grouted a ribbed or profiled cross section is preferred since this increases the bond between the steel member and the grout

For design the following parameters of the reinforcing element should be known:

- Cross sectional area (m^2)
- Diameter (m)
- Tensile strength, yield strength (kPa)
- Ultimate strain (%)

2.2.2.3 Material properties for joints and couplings

As a general rule the joints and couplings should fulfil the same requirements as the reinforcing element, consequently have the same tensile strength, mechanical properties and durability. The protection system of the reinforcing element and the coupler should be compatible. The joints and couplings could from the point of durability be a weak point and should be considered during the design.

2.2.2.4 Material properties for grout

According to prEN 14490 the grout should comply with the following standards: prEN 445, prEN 446 and prEN 447. Below some of the main points from these standards are summarised.

- The water-cement ratio in the grout should be less than 0.44.
- The content of chlorides in the grout should be less then 0.1 percent of the weight of the cement.
- The flow rate of the grout should be enough to guarantee that the hole is completely filled with grout and no water or air is encapsulated in the hole.
- The grout should have properties so that separation and settling of the grout is avoided. Grout with low tendency for bleeding should be used.
- Inert filler (sand) may be added to the grout.

It is not always possible to fulfil all of these requirements and in some cases it might be preferable to use grout with other properties.

2.2.3 Soil

One of the main differences between reinforced fill and soil-nailing is that for soil-nailing the natural in situ soil is used, which means that the existing soils properties has to be determined and the soil-nail structure adjusted to the existing condition. The extents of the geotechnical field investigation test depend on the complexity of the geology. The sequence of soil strata in the area from the wall face to a distance of 1.5 times the wall height should be determined with sufficient accuracy for a wall with a flat surface above the wall. For a wall with a slope above the wall the distance is 3 times the wall height (Clouterre, 1991). The investigation should also cover the material below the wall base. Similar recommendations may be given for natural slopes where soil-nailing should be used for increasing the factor of safety.

In necessary soil information for different parts of the soil-nailed structure is listed. Some of the parameters have to be determined in the field by field testing others may be estimated based on experience.

The amount of field tests should be sufficient to determine the soil parameters and their variation in the area. The field tests should be performed according to applicable European standards and national practice.

It has been discussed whether the peak or the residual value of the angle of shearing resistance should be used for design of a soil-nailed structure. The concept of soil-nailing is based on the theory that the soil-nails are activated after a small deformation of the soil. Consequently the soil has experienced some movement when the nails are activated and it is not unreasonable to believe that, for some parts of the failure surface, the soil deformation is greater than the deformation corresponding to the peak-value and a residual value is therefore more reasonable. A recommendation is therefore that the peak angle of shearing resistance should be used with caution for soil-nailing.

The resistivity depends on the water content of the soil. Hence it is recommended that the tests are performed for the most severe condition, *i.e.* the soil is fully saturated. For a field test it is impossible to choose the conditions but the water content should be accounted for when evaluating the test. There are a number of different methods to determine the resitivity both in the laboratory and the field *e.g.* Wenners 4 electrode or a CPT test with restivity sounding.

Groundwater and high porepressure are the most common reasons for difficulties when constructing a soil-nailed structure as well as with the function of the soil-nailed structure. Consequently it is of great importance to estimate the actual groundwater conditions at site.

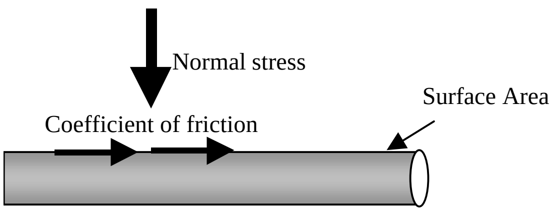
Table 2.7 Soil parameters necessary for design for different parts of the soil-nailed structure.

Soil parameter	Determined by	Soil-nail[9]	Drainage	Facing	Durability
Soil type	Field samples	Yes	Yes	Yes	Yes
Density	Estimated from CPT or undisturbed sampling	Yes		Yes	
Angle of shearing resistance	Estimated or shearbox test	Yes		Yes	
Porewater pressure /groundwater	Measurement in field	Yes	Yes	Yes	Yes
Cohesion intercept	Estimated	Yes			
pH	Test				Yes
Resistivity	Field or laboratory test				Yes
Permeability	Grading curve		Yes		Yes
Degree of compaction	*e.g.* CPT	Yes	Yes		Yes
Chloride and sulphite content or other ion	Soil samples, chemical analyses				Yes
Suitability of the soil to stand unsupported	Test pit			Yes	

2.2.4 Interface between soil and nail

2.2.4.1 Factors influencing the pull-out capacity, T_L

The total pull-out capacity that may be mobilised along the nail will depend on three main parameters: coefficient of friction at the nail interface, normal stress and the perimeter of the nail, see Figure 2.6. Additional information on how different factors influence the pull-out capacity is found in annex A.



$$T_L = m\, s'_N\, q$$

m coefficient of friction

s'_N effective normal stress acting on the nail

q nail perimeter

Figure 2.6 Factors influencing the pull-out capacity

2.2.4.2 Guidelines for a preliminary estimate of the pull-out resistance, q_s

A preliminary estimate needs to be done for the first preliminary design. The final design should however always be confirmed with pull-out tests to verify the estimated value used for design.

In the literature the following guidelines for an estimate of the pull-out resistance may be found.

[9] Including local and global stability of soil-nail and soil-nailed structure

CLOUTERRE (SCHLOSSER ET AL, 1991)

Based on 450-pull-out tests the French national research project Clouterre has established charts for estimating the pull-out resistance, q_s, based on the pressuremeter limit. The tests were performed in the south of France at 87 different sites. The soil has been classified into five different types of soils and both grouted and driven nails have been tested. The chart is presented in Figure 2.7 but the original charts in Clouterre should be used for design.

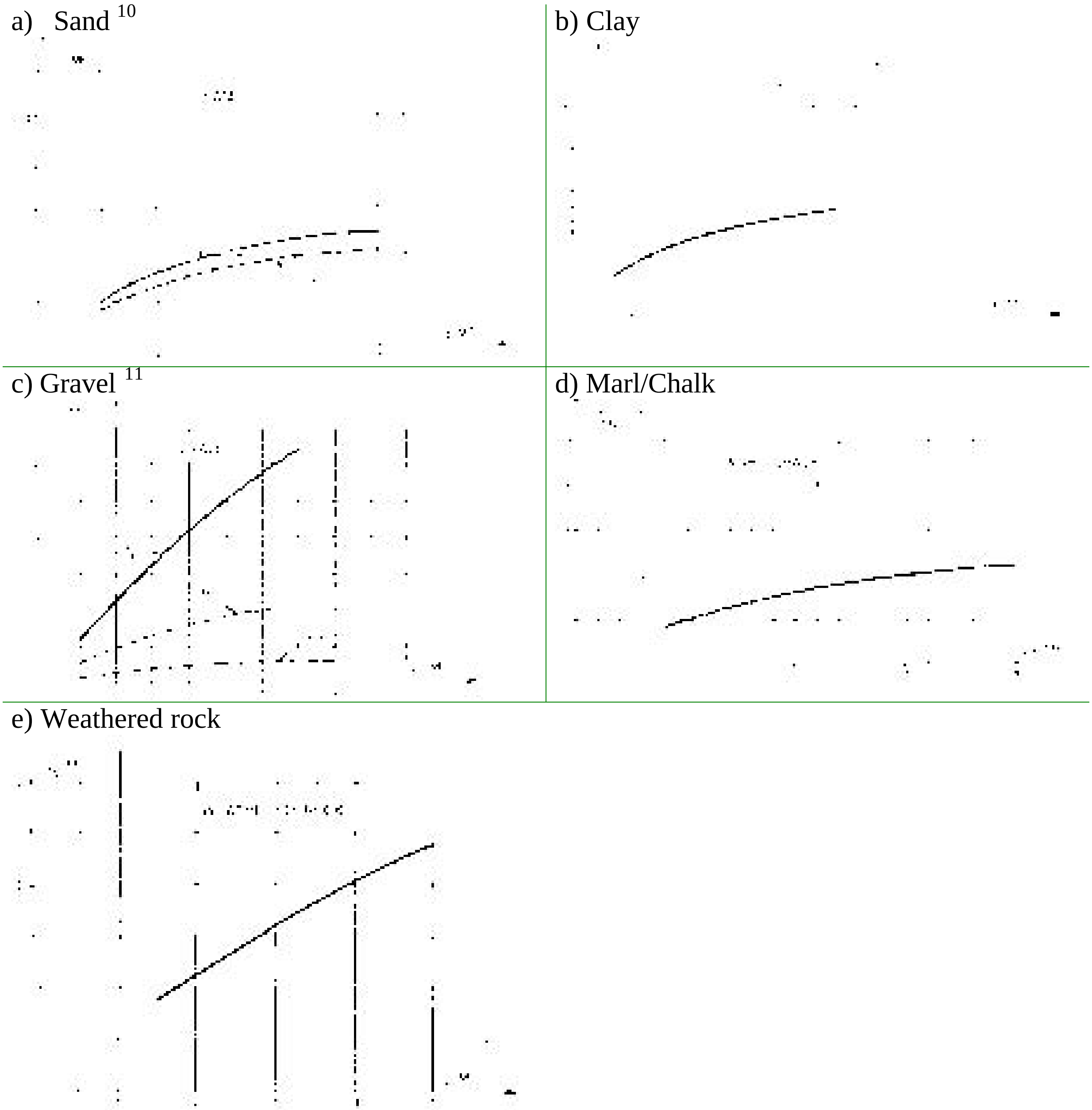


Figure 2.7 Charts for esitmating the pull-out resistance from Clouterre (Schlosser et al. 1991)

[10] S1 gravity grouted, S3 driven
[11] G1 gravity grouted, G2 low pressure grouted, G3 driven

FEDERAL HIGHWAY ADMINISTRATION

In Table 2.8 a suggestion for the mobilised pull-out capacity along the nail depending on the installation method and the soil type, by Mitchell *et al*, 1987, is presented.

Table 2.8 Preliminary Estimate of the Mobilised Pull-out Capacity at the Soil-Nail Interface according to Mitchell et al., 1987

Method of installation	Soil	Pull-out capacity kN/m
Rotary drilled	silty sand	30-60
	silt	18-24
Driven casing	sand	80
	dense sand/gravel	115
	dense moraine	115 - 175
	sandy colluvium	30 - 60
	clayey colluvium	15 - 30
Jetgrouted	sand	115
	sand/gravel	290
Augured	clay	6 -9
	stiff clay	12 - 17
	clayey silt	15 - 30
	limey sandy clay	60 - 90
	silty sand - fill	6 - 9

2.2.4.3 Verification of pull-out capacity

The pull-out capacity should always be verified by pull-out tests at the site. In Chapter 9 a short description of different tests, test equipment, test performances, suggested number of tests and interpretation of tests can be found.

The tests might be performed on a sacrificial nail, *i.e.* a nail that is loaded to failure and consequently it can not be included as a working nail in the final structure. A production nail may also be used, this nail is loaded to its design strength and will continue to be a working nail in the structure after the test.

2.2.4.4 Characteristic and design values

The pull-out capacity is based on a number of load tests. According to the draft of the European execution standard for soil-nailing, the characteristic value is obtained as an average or minimum value from the test multiplied by a factor, η, depending on the number of tests, see Table 2.9. The design value of the pull-out capacity is calculated as:

$$T_d = \eta \frac{T_k}{\gamma_T} \qquad (2.1)$$

γ_T factor that accounts for the natural variation in pull-out capacity due to the soil characteristic and the nail characteristic. In the European standard no recommendation for this can be found. Therefore the following values are suggested to be applied for the pull-out capacity

$$\gamma_T = (\gamma_f \text{ or } \gamma_C) \times \gamma_m \qquad (2.2)$$

The partial factor for the soil strength is multiplied with a factor that accounts for the natural variation in the nail properties (surface area, normal stress and surface roughness). The value of γ_m is suggested to be in the range of 1.2 to 1.4

depending on type of nail. A driven nail with a fixed surface area has a lower value than a grouted nail where the surface area can be expected to vary.

η this factor accounts for the uncertainty in the test method. Recommended values according to the draft of prEN 14490 are given in Table 2.9.

Table 2.9 Reduction Factor, h[12] depending on the Number of Tests.

	Number of test		
η	1	2	>2
Based on the <u>average</u> value from the tests	0.67	0.74	0.77
Based on the <u>lowest</u> test result	0.67	0.80	0.91

2.2.5 Drainage

There are mainly three different types of drainage:
1. Surface drainage (*e.g.* sheeting, channel, trench)
2. Facing drainage (*e.g.* geotextile filter, weep holes)
3. Sub surface drainage (*e.g.* drainage pipe)

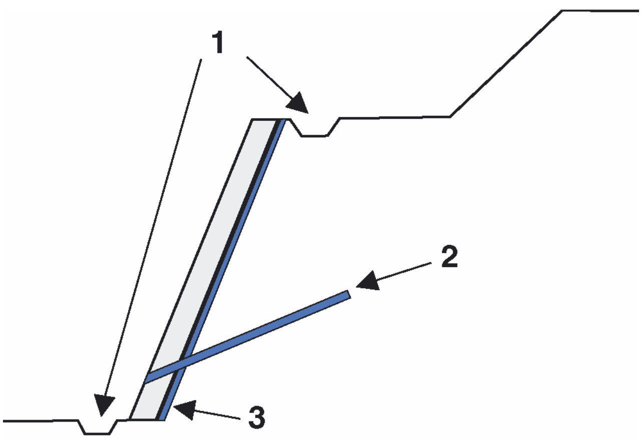


Figure 2.8 Different types of drainage systems

As drainage PVC pipes or metal pipes might be used. The drainage slots could vary between 5 to 20 mm. If metal pipes are used corrosion needs to be considered. Geotextile filter should be designed to have the desired drainage capacity throughout the entire design life.

2.2.6 Corrosion protection system

There are a number of different possible solutions to obtain a soil-nail with satisfactory performance during its entire design life. Below a number of different systems are described, according to prEN 14490. How to choose the necessary system depending on ground condition, type of nail and consequences of failure is further described in Chapter 7.

2.2.6.1 Sacrificial thickness

In the case of sacrificial thickness the decrease in steel area with time is accounted for in the design instead of trying to avoid the corrosion. Depending on the environment at the site where the nails are going to be installed a corrosion rate can be estimated and the cross

[12] η is equivalent to $1/\xi$, where ξ is the reduction factor according to ENV 1991:1

sectional area of the nail sufficiently increased. It is difficult to predict the corrosion rate correctly and it might differ throughout the construction site. Consequently the method is most commonly used in those cases where failure of one nail will not have severe consequences and for short design lives.

2.2.6.2 Surface coating

The steel might be coated both with zink and epoxy. A coating that is not damaged will usually prevent corrosion of the nail according to the design. The difficulty is to avoid damage of the coating during handling, storage and installation.

2.2.6.3 Grout

Encapsulation of the steel in grout will reduce the corrosion. If the grout is evenly distributed along the nail with a thickness of the cover corresponding to the environment and it can be guaranteed that no cracks larger than 0.1 mm will occur, then the grout cover itself might be considered as a satisfactory corrosion protection. However, usually there will be cracks and therefore the grout is usually combined with some other type of corrosion protection system.

2.2.6.4 Impermeable ducts

Encapsulation of the steel in a impermeable duct prevents the corrosion in an efficient way and is commonly used for more severe conditions.

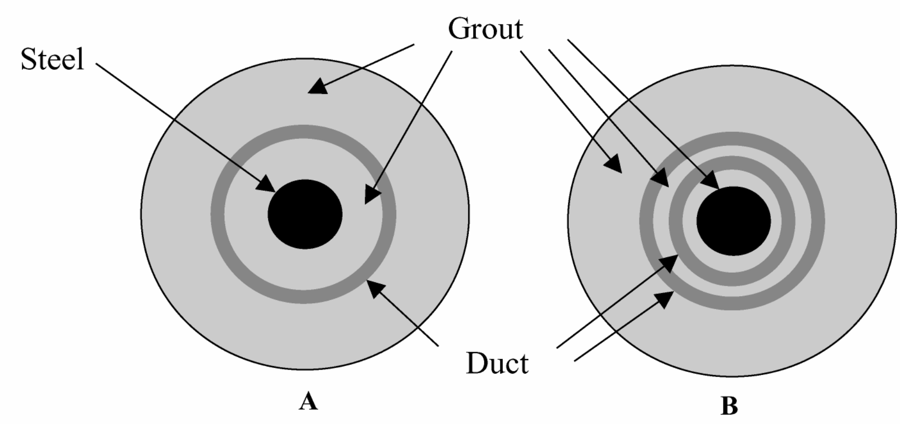


Figure 2.9 Single and double corrosion protection for a nail using impermeable ducts and grout

2.2.7 Facing

A soil-nailed structure may be constructed with or without facing. The inclination of the wall and the consideration of adoption to the surrounding area will determine the choice of facing. Below the different types of facing is divided into four different types according to prEN 14490.

In Section 2.1.4 facing systems for reinforced fill are described. Most of the information in that section can be applied for Soil-nailing as well.

2.2.7.1 No modification of surface / no facing

For a flat slope where soil-nailing has been used to increase the safety it might in some cases be possible to neglect facing. However, it is then necessary to make sure that the natural vegetation is preserved to avoid erosion of the slope.

2.2.7.2 Hard facing

For a steep slope and wall hard facing is usually necessary. The purpose of the facing is to prevent failure of the soil between the nails. The hard facing might be a single layer sprayed concrete, steel mesh combined with sprayed concrete, cast in place concrete or prefabricated concrete blocks.

2.2.7.3 Flexible facing

The flexible facing has a static function and should prevent the soil between the nails to slide out. The facing layout depends on the ground conditions and the layout of the nails. Flexible
facings can be used for relatively steep slopes but not for walls. Usually a steel mesh or a grid is used that is capable of transferring axial and shear forces. It is necessary to connect the flexible facing to the nail head with a suitable head plate to guarantee that the load is transferred. It might be necessary to pre-tension the soil-nail to hold the flexible facing. A vegetation cover as for the soft facing may be used if not the preserved vegetation is enough to guarantee an effective erosion control.

2.2.7.4 Soft facing

Soft facing is another possible solution for a soil-nailed flat slope. The main purpose of this very flexible covering of the slope is erosion control. The soft facing is usually seeded to give a vegetation cover. Climate, altitude and water should be considered when choosing the seed mix.

A light metal mesh or grid combined with a geosynthetic is a possible example of a soft facing. The geosynthetic might be a biodegradable geotextile with seeding. Another solution is geo-grids, geonets or woven open fabrics. The facing should be connected to the nails according to recommendations for the specific system.

3 PRINCIPLES OF DESIGN

3.1 INTRODUCTION

The Commission of the European Communities (CEC) has initiated the work of establishing a set of harmonised technical rules for the design of building and civil engineering works which should in the future replace the different rules in force in the various member states. These rules are known as the Structural Eurocodes. As to these guidelines, 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 guide-lines for design. In each country the Eurocodes might be supplemented with national appli-cation documents, NAD, which together with the ENVs, rule the design in the actual country.

The ENV versions of the Eurocodes are optional to any existing national standards and they are meant to be replaced by EN versions of the Eurocodes, in which case the national standards have to be withdrawn. The work with the EN-versions of the standards is well in progress. From a practical point of view it would be preferable to refer to the latest version of each Eurocode. However, for these guidelines this has been impossible since there exist only provisional versions, prENs, which have no legal validity. Furthermore, the prENs are revised rather frequently.

A formal complication, which can occur in applying ENV 1997-1 for reinforced fill is the definition of fill in the ENV. Fill is defined as a part of the ground, together with soil and rock, when existing in place prior to the execution. On the other hand, the fill is defined as a part of the structure if placed during the execution of the construction works. (ENV 1997-1, 1.5.1(1).) The reason for this separation is not clear and hence, in these guidelines, fills are treated as soil material, regardless whether placed prior to or during the construction work.

3.2 LIMIT STATE DESIGN

In recent decades it has become mandatory in the design to verify structures in two different Limit States, Ultimate Limit State (ULS) and Serviceability Limit State (SLS). These two states give different types of design criteria for determining the calculation model. Verifying Ultimate Limit State corresponds to verifying an inequality, e.g. the resistance (R) must be larger than the action effect (E), thus the criterion is $R-E \geq 0$. As it is not necessary to know exactly when the equality is valid, simplified assumptions of the calculation model can be introduced as long as these assumptions do not lead to an overestimation of the capacity or an underestimation of the action effect. The analysis is said to be on the safe side. In the Serviceability Limit State it is desirable to describe the expected behaviour. In this case, design on the safe side will often lead to uneconomic solutions. In principle this means that the calculation models used in the Serviceability Limit State should be more sophisticated than those used in the Ultimate Limit State. However, a design on the unsafe side will normally not result in a catastrophe. Thus, from a practical point of view, simplified models can also in this case often be used.

Of special interest when using the technique of reinforced soil is that different materials have to work together. For example, the peak strength of the soil and the reinforcement respectively can not normally be combined without any further consideration. If any of the materials has a brittle behaviour, see Figure 3.2, such a practice could result in severe consequences. Especially, mobilisation of geosynthetic reinforcement requires a certain amount of deformation, while steel reinforcement can be regarded as a stiffer material. The basic principle should be to combine strength values of the materials at compatible deformation levels. However, even if justified from a principle point of view, such calculation models are far from day-to-day practice in geotechnical engineering. A simplified way to treat the problem is to introduce model partial factors for simplified calculation models, see also Section 3.5.2 below.

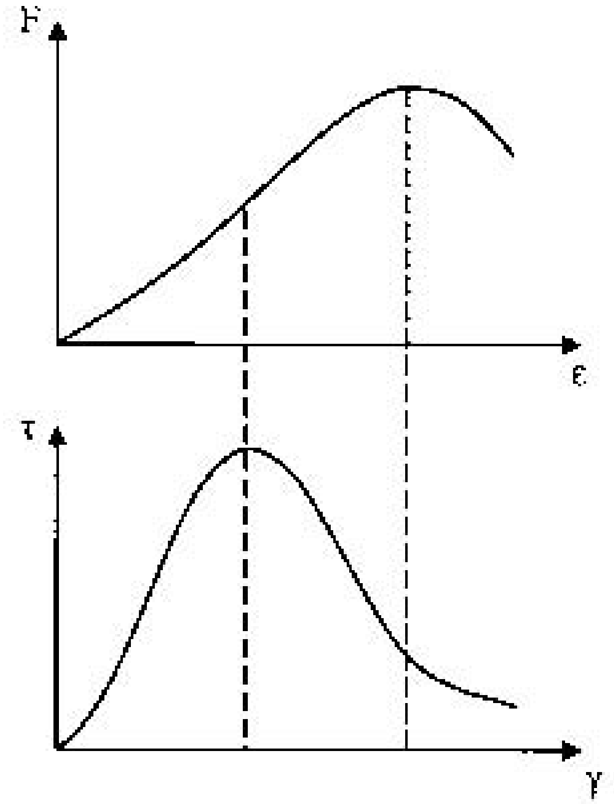


Figure 3.1 Example of compatible deformation levels. Geosynthetic material (top figure) compared to a quick clay (bottom figure). In the figure e is the strain of the reinforcement and ? is the shear of the clay

3.3 PARTIAL FACTORS

Analyses, based upon Limit State design, are in daily practice often combined by a format in which partial factors are used. This format can be seen as a quasi-probabilistic one. The difference between the resistance and the action effect is often called the safety margin, thus the design criterion becomes *M=R-E ³ 0*. The corresponding design criterion in traditional design with a global factor of safety is $R/E \geq F$. By separating the global factor of safety in an action part and a resistance part, called partial factors, the latter design criterion can be rewritten as

$$\frac{R}{E} \geq F = g_R - g_S \qquad (3.1)$$

which again can be rewritten as

$$\frac{R}{g_R} - g_S \cdot E \geq 0 \qquad (3.2)$$

Thus

$$M = \frac{R}{g_R} - g_S \cdot E \geq 0 \qquad (3.3)$$

is the simplest way to describe the design criterion in the partial factor format. By introducing more than two partial factors, more complex relations can be obtained.

To determine a design value of a variable, the actual value to be used as input to the design criterion in the partial factor format, is a process in two steps. First a typical value of the variable is determined formally named a characteristic value, *e.g.* R_k or S_k respectively. This value can be seen as a given fractile of the variable when regarded as a random variable. Secondly, design value of the variable is obtained from the characteristic value by multiplying or dividing it with a partial factor. If the partial factor ≥ unity, a 'safe' design value normally is obtained by multiplying an action variable and dividing a resisting variable, *i.e.* $R_d = R_k / g_R$ and $E_d = g_S \cdot E_k$.

Formally, traditional design with a global factor of safety, (F), can be seen as a simplified procedure with all partial factors except one taken as unity. However, philosophically the two methods are quite different. In the traditional, deterministic method, the resistance and the action are represented by fixed and known values. This means that the resistance *is* R and the action effect *is* E. Hence $F > 1$ implies the inequality $R > E$. A sufficiently safe design is obtained by prescribing the factor of safety sufficiently larger than unity.

In the partial factor format, a probabilistic approach means that both R and E can take a wide range of values. This could be interpreted as the action and the resistance having fixed but unknown values. The same is then valid for M also. The design values should represent one possible combination with sufficiently low probability, *i.e.* the resistance *is* $R_d = R_k / ?_R$ and the action *is* $E_d = ?_S \cdot E_k$. The unlikeness is quantified by the values of the partial factors. The equality $M=0$ is acceptable in the partial factor format but only together with the rare combination R_d and E_d, where the probability of failure becomes sufficiently low. This interpretation of the partial factor format means that the value of a partial factor should be chosen in such a way that a physically impossible design value would not be obtained.

3.4 APPLICATION OF PARTIAL FACTORS

From what is written above the magnitudes of partial factors depend on what design state the calculations are carried out for.

ULTIMATE LIMIT STATE

In an Ultimate Limit State design a low probability of failure is demanded. This must be reflected in the choice of partial factors. Below will be given some principles how partial factors can be determined based upon the problem at hand.

SERVICEABILITY LIMIT STATE

In a Serviceability Limit State design deformations are normally the concern. Partial factors are normally set to unity, g = 1.0, *i.e.* the calculation 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 more rigid structure. For example in a case where the client puts up severe restrictions on deformations, this might be an appropriate option.

3.5 REINFORCED SLOPE/SOIL-NAIL REINFORCED SLOPE

Principally there is no difference when using partial factors in the design of a reinforced slope of fill or a soil-nail reinforced slope, see Figure 3.2.

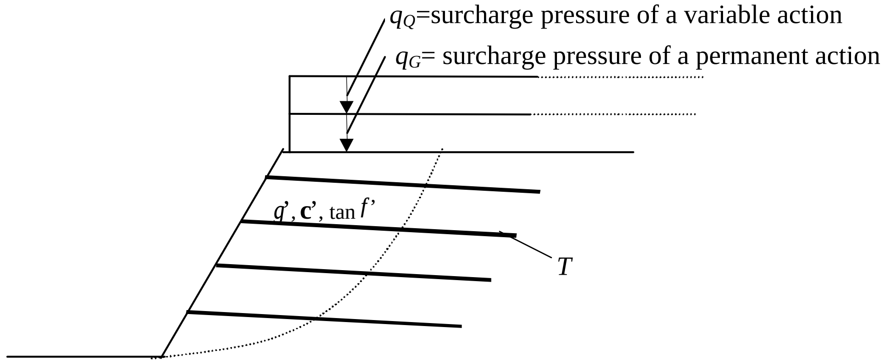


Figure 3.2 Reinforced slope/soil-nail reinforced slope

3.5.1 Traditional factor of safety

In traditional design the factor of safety becomes

$$F = \frac{c_k}{t_k} \tag{3.4}$$

with the shear strength, c_k, at the slip surface

$$c_k = c_k' "+" \tan(f_k') "+" T_k \tag{3.5}$$

and the shear stress, t_k at the slip surface

$$t_k = g_k "+" q_{G_k} "+" q_{Q\,k} \tag{3.6}$$

The expression ″+″ means here the combined effects of the different contributions (*i.e.* not just pure addition). The term T_k= the shear strength of reinforcement/soil-nail including both the effects of increased friction in the soil as well as possible shearing of the reinforcement/soil-nail. Hence:

$$F = \frac{c_k' "+" \tan(f_k') "+" T_k}{g_k "+" q_{G\,k} "+" q_{Q\,k}} \tag{3.7}$$

An alternative formulation is to include the effects of the reinforcement/soil-nail as a decrease of the shear stress, *i.e.* in the denominator of F

$$F = \frac{c_k{}' \text{"+"} \tan f_k{}'}{g_k \text{"+"} q_{G_k} \text{"+"} q_{Q_k} \text{"–"} T_k} \qquad (3.8)$$

It should be noted that the choice of one of the two different formulations has a major influence upon the nominal value of the global factor of safety. A reduction of the denominator as in the latter case gives very high nominal values (or for large amount of reinforcement even negative values). This circumstance can be a reason to avoid the latter formulation.

3.5.2 Partial factor Factor Format

The design criterion becomes the safety margin

$$c_d - t_d \geq 0 \qquad (3.9)$$

or with characteristic values and partial factors any of the two alternatives

$$\frac{c_k}{?_{\mathrm{M}}} - ?_F \cdot t_k \geq 0 \qquad (3.10)$$

$$\frac{1}{g_{Rd}} \cdot \frac{c_k}{g_m} - g_{Sd} \cdot g_f \cdot t_k \geq 0 \qquad (3.11)$$

In the first case model uncertainty and parameter uncertainty are both included in the partial factors, g_{M} and g_{F} respectively, while in the second case these two uncertainties are separated, g_{Rd}: g_{m} and g_{Sd}: g_{f} respectively. With the same notations as in the previous paragraph for the global factor of safety the latter equation can be rewritten as

$$\frac{1}{g_{Rd}} \cdot \left[\frac{c_k{}'}{g_{c'}} \text{"+"} \frac{\tan f_k{}'}{g_{f'}} \text{"+"} h \cdot \frac{T_k}{g_T} \right] - g_{S_d} \cdot \left[g_g \cdot g_k \text{"+"} g_G \cdot q_{Gk} \text{"+"} g_Q \cdot q_{Q_k} \right] \geq 0 \qquad (3.12)$$

<u>Partial factors of materials</u>

$\gamma_{c'}$ partial factor for cohesion intercept, typical value is 1.5

$\gamma_{\phi'}$ partial factor for soil friction, typical value is 1.2

γ_T partial factor for steel or geosynthetic reinforcement. For steel a typical value is 1.1, for geosynthetic reinforcements a likely value is 1.3. In the case of geosynthetic reinforcement, an additional conversion factor h is applied (*c.f.* 3.8.2.2). This factor might be built up of several factors $h_1{\times}h_2{\times}h_3$, considering:

- installation
- creep behaviour
- chemical degradation and biological degradation

For soil-nailing the conversion factor, h, is depending on the number of test performed in field and a partial factor is applied for the pull-out capacity.

<u>Partial factors of actions</u>

Typical values for the partial factors for the action effects, $?_?$, $?_g$ and $?_q$ respectively, are not so easy to prescribe. Values given in Eurocodes as ENV 1991-1 and ENV

1997 1, 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, (*c.f.* the discussion of physically impossible design values in paragraph 3.1; i.e. the unit weight of water becomes ≥ 1350 kN/m^3).

<u>Model factors</u>

γ_{Rd} partial factor for uncertainty in modelling of the resistance

γ_{Sd} partial factor for uncertainty in modelling of the action effect (solicitation) should be regarded as optional factors depending of the problem at hand, i.e. if the model uncertainty has to be separated from other uncertainties

In design of soil reinforcement the strain compatibility of different materials has to be considered. Any other partial factors in the Eurocodes are not aimed for uncertainties in applied calculation models from this respect. Hence, the application of a model factor for the action effect can be an appropriate way to handle such a problem.

3.5.3 Application of partial factor format in stability calculations

From what is said above, the safety margin concept is the common way to use partial factors in design. To use a global factor of safety in conjunction with partial factors would be, if not impossible, at least very confusing. However, a disadvantage in using the concept of safety margin exists in traditional slope stability calculations. The factor of safety is used to establish the critical slip surface. This is not as easy with the help of the safety margin, which is strongly dependent on the volume of soil involved. A way to overcome this difficulty is to use a dimensionless safety margin, where the normal safety margin is scaled with the resistance/shear strength:

$$m = \frac{R-E}{R} = \frac{c-t}{c}\left[= 1 - \frac{1}{F}\right] \tag{3.13}$$

In Figure 3.3 the relationship between the global factor of safety and the dimensionless safety margin is illustrated. There is a one-to-one relation between the dimensionless safety margin and the global factor of safety. Hence, in the same way as with the factor of safety, the safety margin can be used to find the critical slip surface. The dimensionless safety margin, for $R>E$, takes values between 0 - 100 percent, which is a convenient property of a safety margin. If the safety margin is given by the design values of R and E all values of the safety margin larger or equal to zero denote a safe state. When working with existing slope stability computer programs a practical way to use the dimensionless safety margin is to use the approximation $m \approx \ln(F)$. This latter formulation of m has not the advantage to be restricted to 100 percent. In common practice, this is of minor importance, as can be seen from Figure 3.3. To be noted is that this latter formulation does not exist when F, calculated according to Eq. 3.8 is negative.

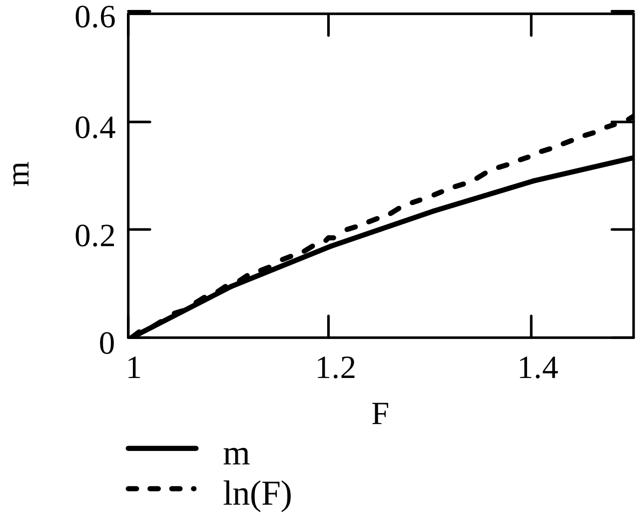


Figure 3.3 Relation between dimensionless safety margin, m and the factor of safety, F.

Summarising what is said above, when applying partial factors in slope stability calculations a dimensionless safety margin is to be preferred. The natural logarithm of the factor of safety, ln (F) calculated with design values of input variables, will serve this purpose.

3.6 DESIGN REQUIREMENTS

3.6.1 Design working life

Table 3.1 Examples of different structures required design working life, 2.4 ENV 1991-1

Class	Required design working life [years]	Example
1	1-5/<2	Temporary structures: reinforced fill/soil-nail
2	25	Replaceable structural parts
3	50	Building structures and other common structures
4	100	Monumental building structures, bridges, and other civil engineering structures

In structures reinforced with geosynthetics the creep strain and the allowed strain in the reinforcement are crucial for the design and the necessary reinforcement strength. For most geosynthetics the creep strains are increasing mostly during the first year after finished construction. *I.e.* the main difference for the design strength is between temporary structures less than 6 months and more “permanent structures”.

3.6.2 Class of safety

In the Eurocode ENV 1997-1 there is only one class of safety, but for example in Sweden there are three different classes. Different classes of safety are defined to consider risks to life and property, where class 1 is defined as little risk and class 3 as great risk to life and property. The safety level used in Eurocodes corresponds to class of safety 3 according to SS-ENV 1991-1. Partial factors corresponding to the different classes are given in Annex B.

3.6.3 Geotechnical category

There are three different geotechnical categories according to 2.1 (5) ENV 1997-1. The various design aspects of a project may require treatment in different geotechnical categories. The three categories have no effect on the values of the partial factors, but imply different demands on the geotechnical investigations, 3.2 ENV 1997-1, as well as the supervision of construction, monitoring and maintenance section 4. ENV 1997-1.

GEOTECHNICAL CATEGORY 1

Category 1 includes small and relatively simple structures.

GEOTECHNICAL CATEGORY 2

Category 2 includes conventional types of structures and foundations without abnormal risks or exceptionally difficult ground or loading conditions.

GEOTECHNICAL CATEGORY 3

Category 3 includes structures that do not fall within the limits of Category 1 or 2. In many cases ground improvement and reinforcement works should be classified in Geotechnical Category 3, 5.5 (3) ENV 1997-1.

Further description of categories of different structures are given in 2.1 (5) ENV 1997-1.

3.6.4 Design method

In some cases, in particular for non-linear analysis, the effect of the uncertainties in the models used in the calculations should be considered explicitly. A factor γ_{Sd} may refer to uncertainties in the action model and/or action effect model, 9.3.2 (2) ENV 1991-1, and a factor γ_{Rd}, covers uncertainties in the resistance model and in the geometrical properties 9.3.5 (2) ENV 1991-1.

To capture different governing conditions different design cases are described below:

CASE A

Static equilibrium

CASE B

Failure of structure or structural elements, including those of the footing, piles, basement walls etc., governed by strength of structural material

CASE C

Failure in the ground

Further description of the different cases is given in Chapter 9.4.1 ENV 1991 -1. For these guidelines case B and case C are relevant[13]. Be aware of when using case B the partial factor of a permanent action is 1.35 which in geotechnical design gives unreasonable results and therefore should be used with care.

[13] The separation of design into the three different design cases A-C, will probably be considerably revised in the final EN:s.

3.6.5 Actions

An Action according to ENV 1991-1 could be either:

a) Direct Action: a force (load) applied to the structure
b) Indirect Action: a deformation or acceleration caused by temperature change, moisture variation, uneven settlement or earthquakes and such

The actions are divided into different categories, among them, permanent action (G) and variable action (Q) and accidental action (A), examples given in Table 3.2.

Table 3.2 Examples of different actions

Item	Symbol (Char. action)	Partial factor	Examples:
Permanent action	q_G	γ_G	Buildings, containers, pallets etc.
Variable action	q_Q	γ_Q	Trucks, cars, trains etc.
Linear or point load Permanent, variable	G, Q	γ_G, γ_Q	Abutments, standing trailers, containers, etc.
Horizontal action	H	γ_H	Handrails, bumpers, wind etc.
Accidental action	A	γ_A	Trucks, cars etc.
Construction action	G_c	γ_{Gc}	Temporary action, during construction, live load and dead load.

Some forces and imposed displacements are actions in some calculations and not in other, 2.4.2 (2) ENV 1997-1. For any calculation the values of actions are known values, 2.4.2 (4) ENV 1997-1. Seismic actions should be considered in addition to the above-mentioned in relevant situations according to EC-8.

3.6.6 Design values of actions

The design values of actions are derived by equation Eq. 3.14. The partial factors for Ultimate Limit State design are tabled in Table 3.3.

$$F_d = g_F \cdot F_{rep} \qquad (3.14)$$

ULTIMATE LIMIT STATE

Table 3.3 Partial factors of actions, g_F – Ultimate Limit State in persistent and transient situations[14], according to 2.4.2 (14)P ENV 1997-1,

Case	Actions			
	Permanent		Variable	Accidental
	Unfavourable	Favourable	Unfavourable	
Case A	[1.00]	[0.95]	[1.50]	[1.00]
Case B	[1.35]	[1.00]	[1.50]	[1.00]
Case C	[1.00]	[1.00]	[1.30]	[1.00]

Comments: In calculation of the design earth pressure for Case B, the partial factors given in table 3.1 are applied to the characteristic earth pressures. For Case C the partial factors

14 These partial factors should be used for conventional structures. In cases with abnormal risks, unusual or exceptionally difficult ground conditions or loading conditions, higher values should be considered.

are applied to the characteristic strength of the ground and to the characteristic surface loads, see further 2.4.2 (17) ENV 1997-1. See further comments below in Section 3.6.7.

Water pressure
For Limit States with severe consequences (generally Ultimate Limit States), design values for water pressures and seepage forces shall represent the most unfavourable values which could occur in extreme circumstances, 2.4.2 (10)P ENV 1997-1.

SERVICEABILITY LIMIT STATE

Partial factors equal to unity (1.0) shall be used for actions in Serviceability Limit State if not specified otherwise, 2.4.2 (10)P ENV 1997-1.

3.6.7 Combinations of actions

Design values of the effects of actions should be determined by combining the values of actions, which can occur simultaneously. In Serviceability Limit States, the combination of actions to be checked also depends of the effects of the actions being checked, e.g. irreversible, reversible or long term effects.

In ENV 1991-1 a number of different combination rules are given. The basic principle is to combine permanent actions, a dominant variable action and combination values for other variable actions. Depending on the Limit State checked, different rules apply as indicated below. The rules are based upon different representative values of actions.

The characteristic value of an action is its main representative value. The design values of permanent actions are:

$\gamma_G G_k$ or G_k

where the characteristic value for a permanent action is used in Ultimate Limit States when the action is favourable and for both favourable and unfavourable actions in the Serviceability Limit States.

For variable actions also reduced values exist to be used in combinations. Based upon the representative values the design values for variable actions are

$\gamma_Q Q_k$,	the design value of the dominant action in Ultimate Limit States
$\gamma_Q \Psi_0 Q_k$,	the combination value for Ultimate Limit States
$\Psi_0 Q_k$,	the combination value for irreversible Serviceability Limit States
$\Psi_1 Q_k$,	the frequent value for reversible Serviceability Limit States
$\Psi_2 Q_k$,	the quasi-permanent value in Serviceability Limit States for long term effects and as combination value for reversible Limit States
Q_k,	the characteristic value for the dominant action in rare combinations in Serviceability Limit States

Combinations of actions are straightforward when the effect of actions is a linear combination of the actions, although it can be cumbersome to check all possible combinations in problems with many variable actions. However, in geotechnical engineering, action effects are often depending on other material properties than the self-weight of the soil, *e.g.* the undrained shear strength when calculating earth pressure. In such cases it is fundamental to make a clear distinction between actions and the effect of the actions. As mentioned above in Section 3.6.6, it is prescribed in ENV 1997-1, that when calculating earth pressure according to case B, partial factors shall be applied to characteristic earth pressure. In such cases with both permanent and combinations of variable actions it is difficult, if not to say impossible, to apply the combination rules in ENV 1997-1. In these guidelines, the procedure for case C is incorporated, that is partial factors are applied to characteristic properties of the soil and surface loads. This procedure makes the application of combination of actions simpler.

3.7 GEOMETRICAL PROPERTIES

In the Eurocode there are no partial factors for geometrical uncertainties, but in the designs it is necessary to include relevant tolerances on all the geometrical input data.

3.8 MATERIAL PROPERTIES

3.8.1 Characteristic values

3.8.1.1 Characteristic values of geotechnical parameters

The selection of characteristic values for geotechnical parameters shall according to 2.4.3 ENV 1997-1 be based on the results of laboratory and field tests, by well-established experience. The parameters shall be selected as a cautious estimate of the values in respect to the following aspects: background information, the extent of the survey, the variation of the results, the actual Limit State being considered, among others.

3.8.1.2 Characteristic material properties of reinforcement materials

The manufacturer gives characteristic material properties X_k, of the reinforcement material, such as:

- Short term tensile strength versus strain,
- Results from creep rupture tests
- Results from tests for durability
- Test results of seam strength

What properties required for the different design methods are described further in the different design chapters as well as in the material chapter.

3.8.2 Design values

3.8.2.1 Design values of geotechnical parameters

The design values of geotechnical parameters shall be derived from characteristic values, using equation 3.15.

$$X_d = X_k / g_M \qquad (3.15)$$

ULTIMATE LIMIT STATE

Tabled partial factors, g_M, Table 3.4, are in accordance with ENV 1997-1. For partial factors according to ruling national document (NAD) see Annex B Chapter B.2.

Table 3.4 Partial material factors[15] g_M - Ultimate Limit State in persistent and transient situations, according to ENV 1997-1

Case	Ground Properties			
	tan ϕ	c′	c_u	q_u[16]
Case A	[1.10][17]	[1.30]	[1.20]	[1.20]
Case B	[1.00]	[1.00]	[1.00]	[1.00]
Case C	[1.25]	[1.60]	[1.40]	[1.40]

SERVICEABILITY LIMIT STATE

In Serviceability Limit States all values of g_M are equal to 1.0, according to 2.4.3 (13) P ENV 1997-1.

3.8.2.2 Design values of reinforcements

PHYSICAL PROPERTIES

The properties for the reinforcement should be documented according to relevant standards see further information in Chapter 2.

CALCULATION OF DESIGN STRENGTH

The allowable tensile force per unit width of the reinforcement usually depends upon the type and safety requirements of the reinforced structure, the stresses the geosynthetic reinforcement is exposed to, on the work execution and on environmental factors. For that reason the ultimate strength parameters determined from short term index tests are multiplied by several conversion factors. Such are used to account for potential creep, installation damage and ageing.

Allowable design strength of the geosynthetic reinforcement (partial factor format):

$$X_d = \frac{h_1 \cdot h_2 \cdot h_3 \cdot X_k}{g_M} \qquad (3.16)$$

The factors h_i (Table 3.5) are here introduced as conversion factor to calibrate data from test conditions, see further Figure 3.4 and Chapter 2, Materials. Different types of structures give different demands on the geosynthetic reinforcement, which result in different values of h_i. If the material properties are based on long-term tests the factor h_i becomes 1.0 (see further Chapter 2.1.1.5).

[15] In ENV 1991-1 γ_M, resp γ_m has different descriptions. In the first case the model uncertainty, γ_{Rd} is included in the partial factor and in the latter case the two uncertainties are separated. Observe that in ENV 1997- γ_m is written, but in prEN this is corrected

[16] Compressive strength of soil and rock

[17] Values in brackets [ ] to be used unless different values are given in NAD.

Table 3.5 Conversion factors of geosynthetic reinforcements

Conversion parameter – material aspect	Conversion factor
Factor of creep (depending on lifetime)	$\eta_1=1/F_{cr}$
Installation damage	$\eta_2=1/F_{id}$
Biological and chemical degradation	$\eta_3=1/F_{env}$

The F_i are taken from "Guide to durability"[19] (CEN-document) to be used in the absence of sufficient test data on long term performance.

F_{env} environmental
$F_{geo}=g_M$ overall material safety factor
F_{cr} creep reduction factor
F_{id} installation damage reduction factor

Determination of characteristic long-term strength should include an evaluation of compatibility with surrounding material. For geosynthetic reinforcement in combination with brittle material, (*e.g.* quick clay) the strain at failure should be limited. See reference under relevant chapter.

Documentation regarding product related reduction factors in general should be available by the supplier and independent institutes or similar.

[19] STG Teknisk rapport 102 CEN CR ISO 134 34:1998 och ISO/TR 134 34:1998

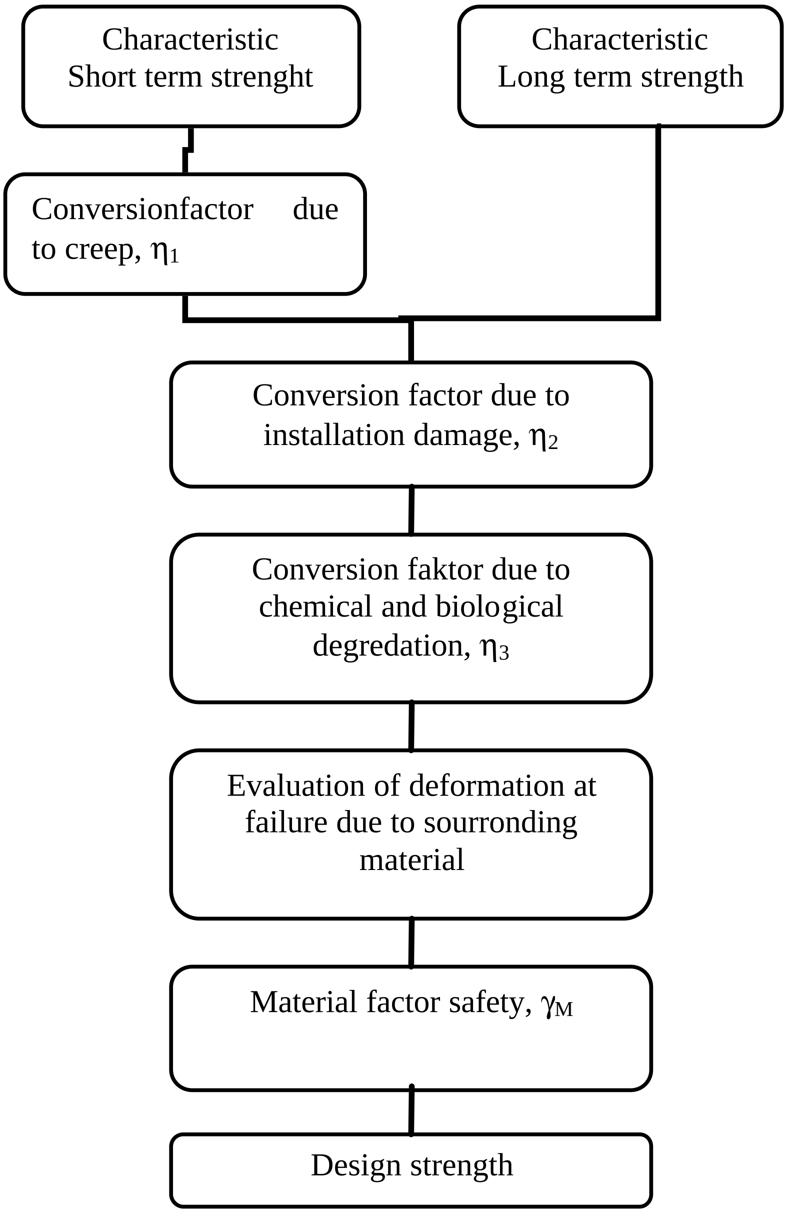


Figure 3.4 How to calculate the design strength for geosynthetic reinforcement Eq 3.16

PROPERTIES RELATED TO FIELD TESTING

When designing a soil-nailed structure the value of *h* (Eq. 3.16) is dependent on the amount of field tests carried out, see further Chapter 2, Materials. A material partial factor g_T is here applied to the pull-out capacity (soil/soil-nail) to take the variation in test results into account.

4 REINFORCED STEEP SLOPES AND WALLS

4.1 INTRODUCTION

Reinforced fill very often appears to be an economically attractive way of constructing walls, abutments, embankments, sound barriers, steep slopes etc.

By introducing horizontal layers of reinforcement into a structure it is possible to stabilise and reinforce the fill.

Since the early eighties a number of projects have been constructed using reinforced fill.

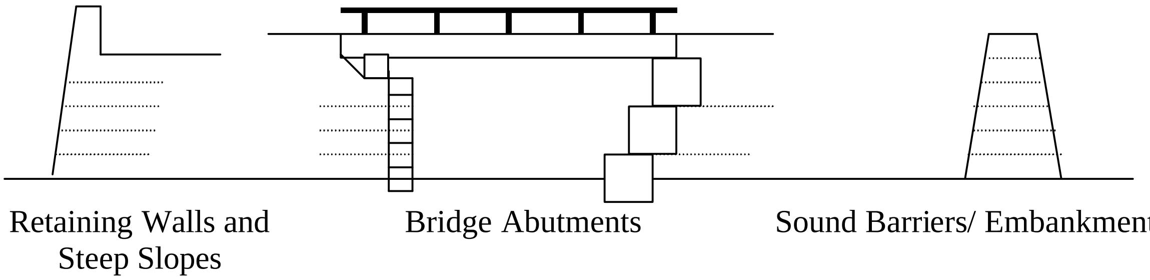


Figure 4.1 Typical applications based on reinforced fill

Walls and Abutments normally cover applications of Reinforced Soil with a slope angle between 70 - 90 degrees,while Steep slopes cover slopes less than 70 degrees. In general, designs based on reinforced soil, must include internal as well as external stability checks.

The design should be based upon well-known and generally accepted design methods, which take the actual site conditions into account in a proper way.

4.1.1 Function of the reinforcement

Reinforced fill structures consist of horizontal layers of reinforcement placed at several levels.

The function of the reinforcement is to provide tensile strength to the soil-reinforcement matrix. In a reinforced steep slope or wall this will prevent the soil masses from falling down by adding tension in the reinforcement at the most critical slip surface. At the same time the anchoring zones in front and behind the failure figure create the necessary pull-out resistance in the reinforcement. This is a simplified model and real behaviour might be a bit more complex.

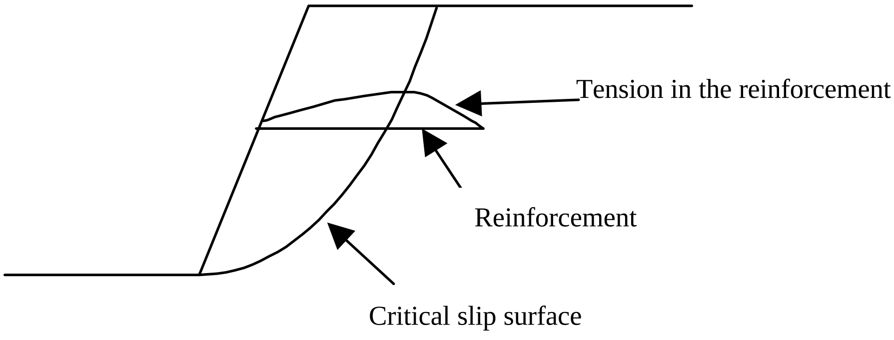


Figure 4.2 Tension in the reinforcement

Depending on the type of structure the facing system is either free-standing or alternatively the reinforcement is used to stabilise the facing. A common way is to wrap-around the reinforcement at the face and re-embed the reinforcement sufficiently deep.

Some deformation of the structure is required to activate the required tension in the reinforcement. Settlements in relation to this issue are discussed in Chapter 4.4.1.

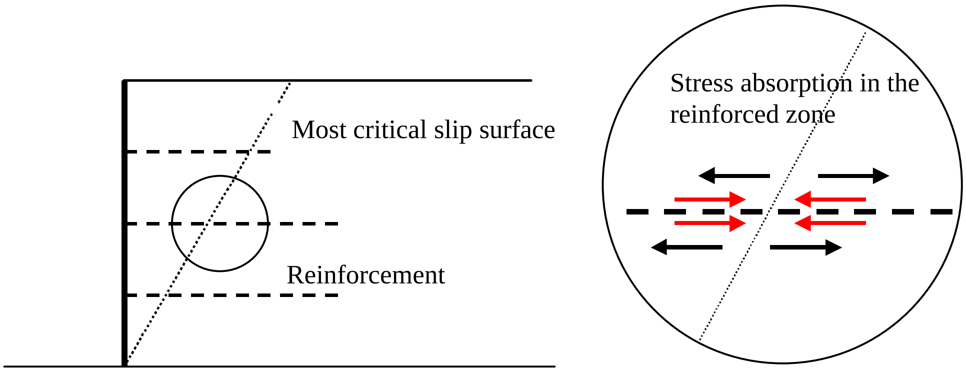


Figure 4.3 Principle showing the function of the reinforcement and the stress absorption in the reinforced zone. Here is the most critical slip surface shown as a straight line, but other more sophisticated slip surfaces may be evaluated depending on the calculation method used.

4.2 SPECIFIC INFORMATION NEEDED FOR DESIGN

For the design of a reinforced retaining structure it is necessary to have a reasonable level of information available to get an accurate result.

The safety level of the structure is depending on the accuracy of the information regarding soil, fill, groundwater conditions, water pressure, drainage, loads, reinforcement *etc.*
It is of great importance to check the external stability as well as the internal stability. For retaining walls and abutments, where facing systems are included, the facing systems as well should be properly checked.

Most failures seen in retaining structures and steep slopes are related to a underestimated water pressure. Water and drainage should therefore be given high attention in the design phase/stage.

For steep slopes, where the facing often is intended to be vegetated, special attention should be given to the growing conditions at the surface.

4.2.1 Geometry and foundation properties

To evaluate a reinforced wall application, a detailed description of the geometry and the actual foundation properties must be carried out. This description also contains information about groundwater, pore water pressure, hydraulic flow etc. Considerations related to cables, pipelines *etc*. in the ground should be included in case of future work near the reinforced fill application.

In the Eurocode there are no partial factors for geometrical uncertainties, but in the designs it is necessary to include relevant tolerances on all the geometrical input data. Irrespective of using Eurocode or national standards, soil properties should be examined properly in a way that any critical stability problem can be avoided. This also includes the design of drainage.

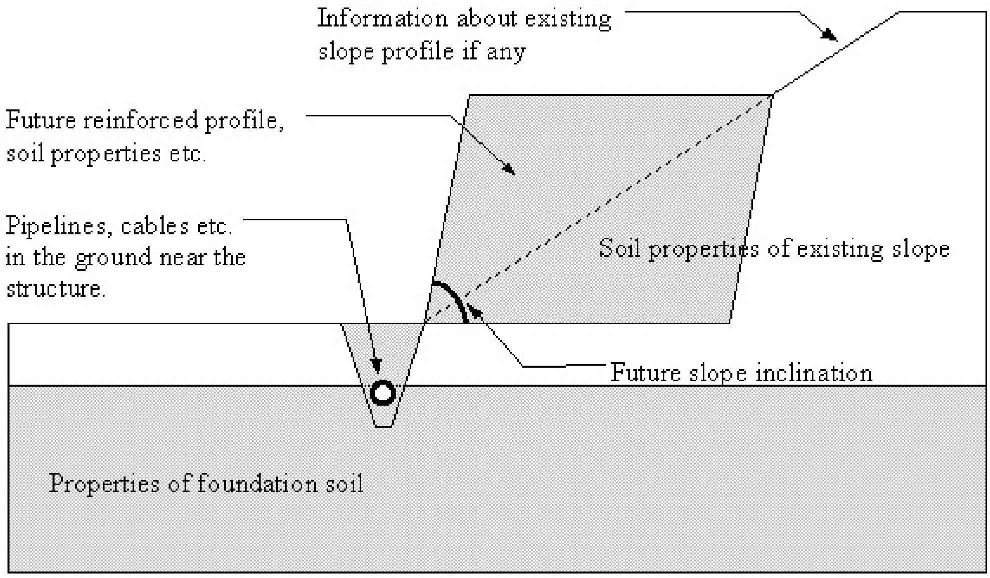


Figure 4.4 Typical geometry for a reinforced fill including foundation properties

4.3 ULITIMATE LIMIT STATE DESIGN

Principles for the Limit State design are defined in ENV 1991-1, ENV 1997-1and other national standards. The Limit State design is based on partial factors of safety applied on loads, reinforcement properties and geotechnical parameters.

Design requirements are described in these guidelines in Chapter 3.63.4. For most common reinforced walls and slopes load case C is relevant as described in Chapter 3.6.4.

According to some national standards structures are divided into different classes of safety to which there are corresponding partial factors, see further in Annex B.Failure modes
During the last decades a great deal of energy has been given to develop new and more accurate design methods for retaining walls especially. Some of the methods are based on simplified models giving reasonable results. Others use advanced computer models either based on iteration or finite element methods.Both simplified models and advanced computer models are valid for the design of many different types of walls.
The main differences between the available design methods are related to the treatment of water, settlement and displacement. The more sophisticated the models get the more accurate the models may handle these items. As a result safer and more economic designs can be expected.

Any design must take into account:
- Internal stability
- Global stability

The following situations should be investigated:

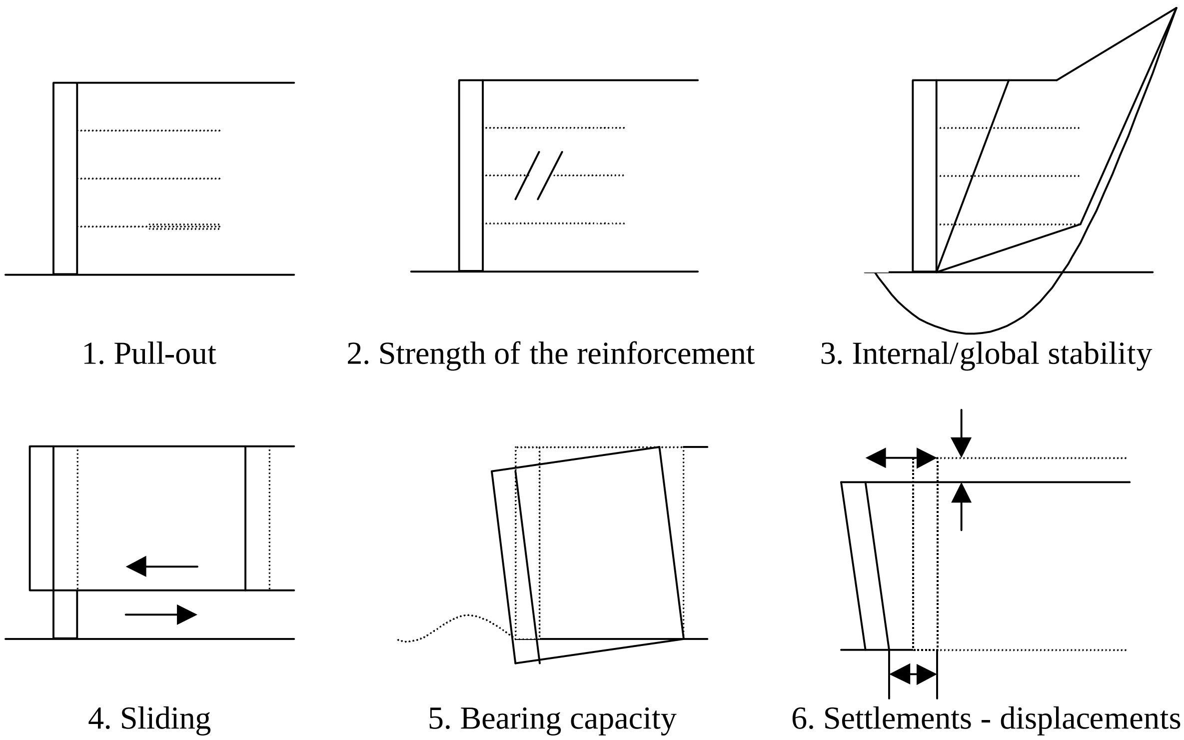


Figure 4.5 Typical failure modes, settlements and displacements to be investigated

Case 1, 2 and 4, above, will be demonstrated here as well as the **internal stability** check in case 3, above. **Global stability** is to be treated according to traditional geotechnical principles. **Bearing capacity** problems should be investigated unless the used design model take into account the issue throughout the external stability analyses. Bearing capacity problems are to be treated according to traditional geotechnical principles.

Many design guidelines are also including overturning as a potential failure mode. The calculation and criterion are similar to what is known for classic gravity retaining structures. The flexibility of the reinforced soil structure should make the potential for overturning failure highly unlikely. However, the overturning criterion, giving a maximum permissible excentricity, aid in controlling lateral deformation by limiting tilting, and may be a good alternative to more advanced analyses such as FEM *etc*.

4.3.1 Design values and design loads

To get a reliable design but also to avoid overdimensioning, all materials should be properly examined according to Chapter 2. Small projects might choose to use conservative values due to economical reasons. In case of larger projects, testing of fill materials should always be included to optimise the design.

4.3.1.1 Design values of parameters

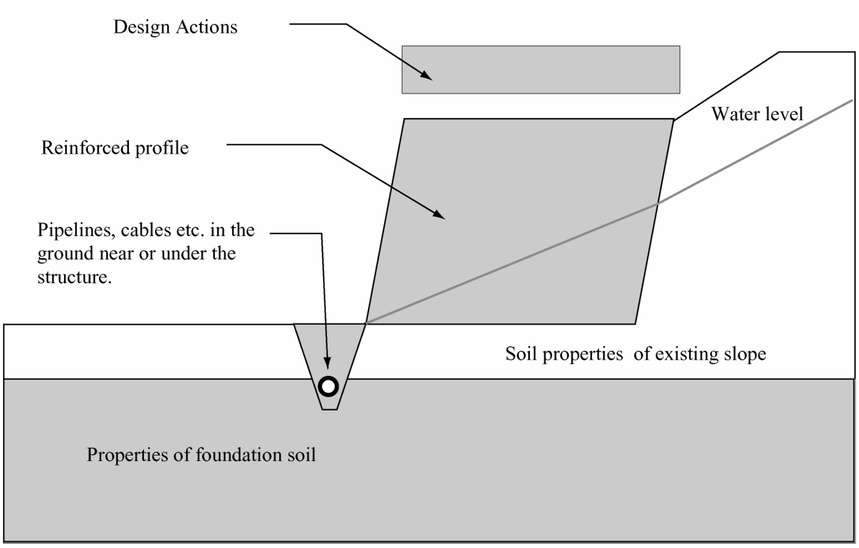


Figure 4.6 Example of a geometry including foundation and load properties

Table 4.1 Typical design parameters for a structure is given by the characteristic values reduced with partial factors given according to Chapter 2.

Characteristic parameters	Partial factor	Design parameters	Unit
γ_k –unit weight of fill/soil	$\gamma_\gamma = 1.0$	γ_d	kN/m^3
ϕ_k - friction angle	?$_\phi$ (tan ϕ_k)	ϕ_d	°
c_{uk} – undrained shear strength	γ_{cu}	c_{ud}	kPa
c'_k – cohesion intercept	$\gamma_{c'}$	c'_d	kPa
q_G, q_Q –surcharge load	γ_G , γ_Q	q_G, q_Q	kPa

In case of pipelines *etc*. near or under the structure, these structures should always be evaluated in order to demonstrate the resistance against future earth pressure etc.

4.3.1.2 Design strength of the geosynthetic reinforcement

Materials to be included in a retaining wall structure should be properly evaluated based on the characteristics given from the supplier, independent research institutes or national certificates, approvals etc.

The properties for the reinforcement should be documented according to relevant standards *cf*. Chapter 2. Documentation regarding product related conversion factors should in general be available by the supplier and independent institutes or similar.

Principally the design strength of the reinforcement (T_d) should be calculated according to procedures outlined in Chapter 3. The deformation of the reinforcement corresponding to the allowed settlement and displacement, see Chapter 4.4.1. Note that use of some types of facings, and/or in combination with fixed structures may imply even more restrictions on the deformation.

Calculations based on FEM (finite element method) are including the E-modulus as a stiffness parameter. The modulus is influenced by degree of strength, deformation and time. The modulus in general should be defined properly and calibrated in the design and documented for the specific product .

Interaction coefficients between fill/soil and the reinforcement will normally vary according to Table 4.2. The partial factors for sliding and pull-out are depending on the certainty in which the coefficients of interaction are determined. If no documentation is provided for a specific type of reinforcement, conservative values should be used.

Table 4.2 Interaction coefficients and partial factors

Interaction fill/soil – reinforcement		Value
Coefficient of interaction fill/reinforcement	α_1	0.5 - 1.0
Coefficient of interaction foundation soil/reinforcement	α_2	0.5 – 1.0
Partial factor for sliding across surface of reinforcement	$?_s$	1.3
Partial factor for pull-out resistance of reinforcement	$?_p$	1.3 – 1.5

Recommended coefficients of interaction are given in Chapter 2 and Annex A, but should in general be based on tests on the actual reinforcement and soil. It is recommend choosing partial factor for pull-out resistance between 1.3 and 1.5. The partial factor for sliding is recommended to 1.3. However, this factor may be reduced if both fill material and reinforcement product is given and site-specific results or relevant pull-out test etc. are available.

Overlapping and sewing of geosynthetics in the prime strength direction in general should not be allowed, unless testing, relevant certificates *etc*. are able to show the capability of the connection.

4.3.2 Design step by step

The design method demonstrated in this chapter is based on a simplified model. Only the internal stability calculation and the sliding calculation are shown. For complicated structures more sophisticated methods are recommended to avoid limitation and confirm the validity of the design method.

Internal failure can occur in two different ways:

- Failure by elongation or breakage of the reinforcement.
  The tensile forces (and, in case of rigid reinforcements, the shear forces) in the reinforcement become so large that the reinforcement elongates excessively or breaks, leading to large movements and possible collapse of the structure.
- Failure by pull-out.
  The tensile forces in the reinforcement become larger than the pull-out resistance, i.e. the force required to pull the reinforcement out of the soil mass. This, in turn, increases the shear stresses in the surrounding soil, leading to large movements and possible collapse of the structure.

The design process to ensure the internal stability therefore consists of determining the maximum developed tensile forces and their location along a locus of critical slip surfaces and the resistance provided by the reinforcements both in pull-out capacity and tensile strength.

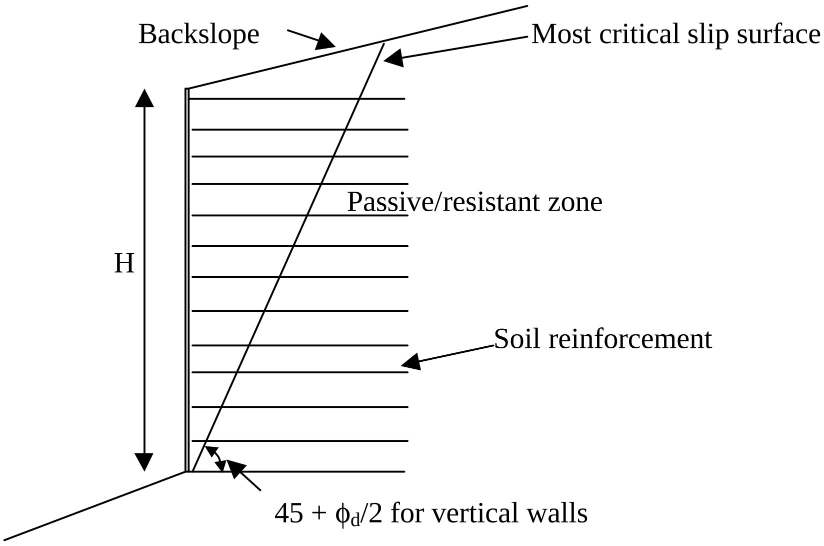


Figure 4.7 Model for design

The most critical slip surface is for a vertical and near vertical wall assumed to be a straight line passing through the toe with an angle of 45 + ϕ_d/2 to the horizontal (assuming no wall friction). For inextensible reinforcement, like steel strips and steel meshes, another slip surface is normally used shown in Annex C. If the wall face batters the most critical slip surface will shift to a smaller angle and also change to a curved line. It is recommended to use a conventional slope stability calculation to find the most critical slip surfaces when the wall face batters more than 10°.

Different diagrams and formulas are available to find the active earth pressure coefficient, K_a. Both the roughness between soil and facing, inclination of the front or possible back slope above the structure may be taken into account. The formula below is valid for a vertical wall assuming no wall friction and no backslope above the fill. For geosynthetic reinforcements the active earth pressure coefficient is used for both internal and external stability calculations. For steel reinforcements/inextensible reinforcements the active earth pressure coefficient should be increased by a factor 1.2 – 2.5 for internal stability calculations, see Annex C.

Definitions:
Active earth pressure coefficient:

$$K_a = \tan^2(45 - \frac{f_d}{2}) \qquad (4.1)$$

where

$$f_d = \arctan(\frac{\tan f_k}{g_f}) \qquad (4.2)$$

q_G = characteristic load

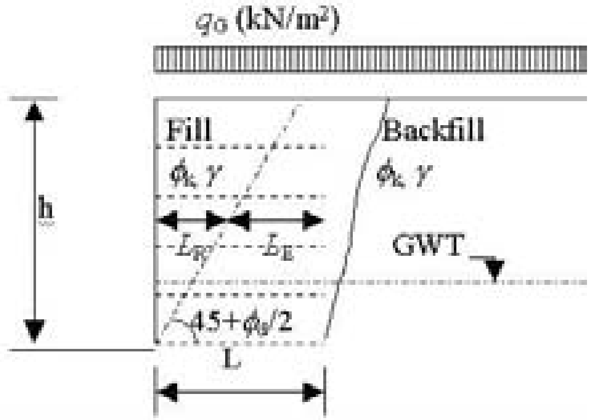


Figure 4.8 Geometry and input parameters for a wall.

4.3.2.1 Stresses

Stresses can be calculated as follows, illustrated in Figure 4.9.

Vertical effective stress:

$s'_{v} = s_{v} - u \qquad s_{v} = g_{d} \cdot h + q_{d} \quad u = \gamma_{w} \cdot h_{w}$

where:

q_d = design action

g_d = design soil unit weight

g_w = design water unit weight

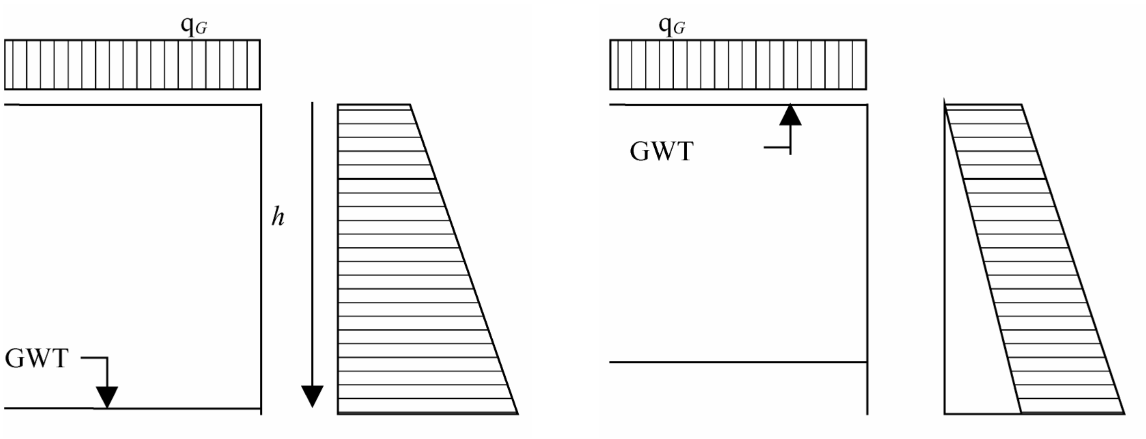


Figure 4.9 Stress diagram without water (left) and with water (right)

The horizontal pressure from soil and water is then calculated as follows:

$$p_{ad} = (K_{ad} \cdot s'_{vd}) + u_d \qquad (4.3)$$

Where

p_{ad} horizontal pressure at given level

s'_{vd} the effective vertical stress level

u_d the water pressure

4.3.2.2 Distance between reinforcement layers

When designing vertical walls and abutments it is possible to vary both the spacing between the reinforcement layers and the strength of the reinforcement.

Normally the critical spacing for reinforcement layers can be calculated as follows:

$$S_{vd} = \frac{T_d}{p_{ad,max}} \qquad (4.4)$$

Where,

S_{vd} distance between reinforcement layers

T_d design strength of the reinforcement

$p_{ad,\,max}$ maximum horizontal pressure from soil and water

The spacing between the reinforcement layers typically varies between 0.2 – 0.6 m and should normally not exceed 1.0 m. Secondary reinforcement layers with lower design strength and/or shorter length, can be implemented in between the primary reinforcementto increase the internal stability (sliding) between the reinforcement layers and to reduce the deformations at the front surface. The length of the secondary reinforcement should be in the range of 2 m to increase the stability and to reduce the deformations of the front.

If a given spacing is required, *e.g.* by use of segmental block wall units, the same formula can be used to calculate the required reinforcement strength.

The same formula may be used to calculate structures with varying spacing and reinforcement strength, where $p_{a,\,max}$ is the maximum horizontal stress at the actual level (= p_a).

4.3.2.3 Calculation of the reinforcement length

The reinforcement length is normally ranging from approx. 60 to 80 percent of the height of the wall. Both higher and lower values might appear. The length of the reinforcement is often selected to an equal length for the whole structure and for non sophisticated design methods an equal length should be used. Several parameters will influence on the necessary reinforcement length such as soil shear strength, wall inclination, back slopes and loads on top of the structure, water/ pore pressure in the fill, sliding along the foundation soil and bearing capacity of the foundation soil.

The largest reinforcement length given from internal stability analysis (at the top), sliding analysis (at the base) and overall stability analysis (at the base) is normally used for all reinforcement layers.

The internal stability check (distance to the critical slip surface and the pull-out capacity) will normally require the longest reinforcement at the top of the structure, while lateral sliding and global stability check often require the longest reinforcement at the base. The required reinforcement length for all layers is normally set to the largest of the reinforcement lengths found from “internal stability”, “sliding” and “overall stability”.

INTERNAL STABILITY

The total length of the reinforcement is calculated as:

$$L = L_R + L_E \qquad (4.5)$$

In the passive zone the anchor length of the reinforcement is calculated to achieve the required pull-out capacity:

$$L_E = \frac{p_{ad} S_{Vd}}{\frac{2a_1}{g_p}(c'_d + g_d \cdot h \cdot \tan f'_d)} \qquad (4.6)$$

where,

a_1	the coefficient of interaction between soil and reinforcement
p_{ad}	horizontal desing pressure at given level (incl. water)
g_p	partial factor for coefficient of interaction (also related to the pull-out resistance of reinforcement)
c_d'	the design cohesion intercept of fill/soil in terms of effective stress
g_d	design unit weight of the fill
h	the depth (from the topof the wall) to the actual level

L_e should, independently of the calculations have a length of 1.0 m expressed as $L_E \geq L_{min} \geq$ 1.0 m. Traffic loads and other live loads should not be included in the calculations of the pull-out capacity.

In the active zone the length of the reinforcement is calculated as the distance from the facing to the most critical slip surface (for the unreinforced structure). For a vertical wall the critical slip surface is assumed to be a straight line through the toe of the slope and with an angle of 45 + ϕ_d/2 degrees to the horizontal, see Figure 4.7and the reinforcement length may be calculated as:

$$L_{\mathrm{R}} = (H - h\ )\tan(45 - \frac{f_{\mathrm{d}}}{2}), \qquad (4.7)$$

If the wall face batters more than 10° the critical slip surface should be found using classical slope stability analyses. The length of the reinforcement in the active zone is then obtained from the geometry.

The total required reinforcement length due to internal stability check is:

$$L = L_{\mathrm{R}} + L_{\mathrm{E}} \qquad (4.8)$$

Note: external stability considerations may require longer reinforcement.

L_{R} should in general have a length of minimum 1.0 m to ensure proper anchoring. The length will for the lower layers become less than 1.0 m, which means that either the reinforcement should be properly fastened to the facing of the wall or alternatively be wrapped around as shown in Figure 4.10.

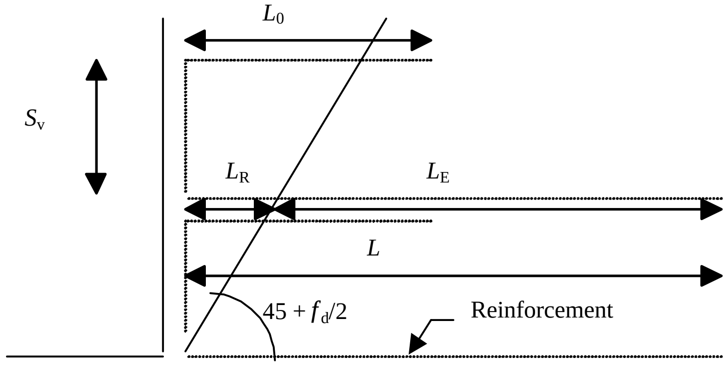


Figure 4.10 Principle of "wrap around", where L_0 -should be min. 1.0 m.

For most practical purposes the minimum wrap-around length L_0 = 1.0 m is sufficient. When the reinforcement spacing is larger than 0.5 m, $L_0 > 2\ S_v$ is recommendedas a rule of thumb . For structures with a high groundwater table the anchoring should always be controlled.

EXTERNAL STABILITY; LATERAL SLIDING

The horizontal thrust from the soil behind the reinforced zone may cause sliding above or below the bottom layer of the reinforcement. A simplified method to calculate the minimum reinforcement length L_e is:

$$L_e \geq \frac{0.5K_{\mathrm{ad}}H\left(g_d H + 2\left(q_{\mathrm{Q_d}} + q_{\mathrm{G_d}}\right)\right)g_{\mathrm{s}}}{g_d h\, a'_2 \tan f'_d} \qquad (4.9)$$

where H is average fill height over the reinforcement length.

L_e is the minimum total reinforcement length at the base level to prevent sliding (the subscript letter e is added to be consistent with other chapters where the same formula is used). The sliding stability should by checked both above and beneath the bottom reinforcement layer using the relevant internal angle of friction and interaction coefficient. For more complex structures, e.g. if the reinforcement length or the fill material is varying, more layers may be checked and calculated (*i.e.* varying H).

Note: global stability considerations may require longer reinforcement.

EXTERNAL STABILITY; GLOBAL STABILITY AND BEARING CAPACITY FAILURE

Global stability and bearing capacity should always be evaluated according to traditional geotechnical principles *e.g.* Bishop, Janbu *etc*.

Global stability could be determined using force and/or moment equilibrium analyses which could be performed using a classical slope stability analysis method. The reinforced soil wall is first considered as a rigid body and only failure surfaces completely outside the reinforced mass are considered. Then compound failures, passing both through the reinforced and unreinforced zones are considered. For simple structures (near vertical face, uniform reinforcement length and spacing, one reinforced soil type, no significant slopes at the toe or above the wall) compound failures will generally not be critical.

If the minimum safety factor is less than the required minimum, increase the reinforcement length or improve the foundation soil.

Generally two modes of ***bearing capacity failure*** exist, general shear failure and local shear failure or 'squeezing'.

To prevent bearing capacity failure of the mode ***general shear*** it is required that the vertical stress at the base does not exceed the allowable bearing capacity of the foundation soil. It is important not to underestimate the vertical stress at the base (remember that some of the load from the soil pressure behind the reinforced structure will have both a vertical and a horizontal component acting on the base). The ultimate bearing capacity could be determined using any classical soil mechanics method.

Local shear or 'squeezing' of the foundation soil could be a failure mode in weak cohesive soils. To prevent large horizontal movements the maximum height is approximately given by:

$$g_d \cdot H \leq 4 \cdot c_{ud} \qquad (4.10)$$

4.4 SERVICEABILITY LIMIT STATE DESIGN

The different types of problems to be considered in the Serviceability Limit Stateare as follows:

- Settlement of the fill
- Displacement of the front (creep strains in the reinforcement, acceptable deformations of the facing system)

4.4.1 Settlements and displacements

Vertical settlements can be calculated according to traditional methods based on the effective stress level and modulus of consolidation.

Horizontal displacement in the range from 0.1 - 0.3 percent of the height can be expected for geosynthetic reinforced walls and abutments. This, however, may depend on the stiffness of the reinforcement used, type of material and the compaction of the fill.

The designed creep strains (post constructional) in the reinforcement should normally not be allowed larger than 2 percent during the design lifetime, see Figure 4.11. For constructions where it is important to minimise the post constructional deformations the creep strains should be minimized.

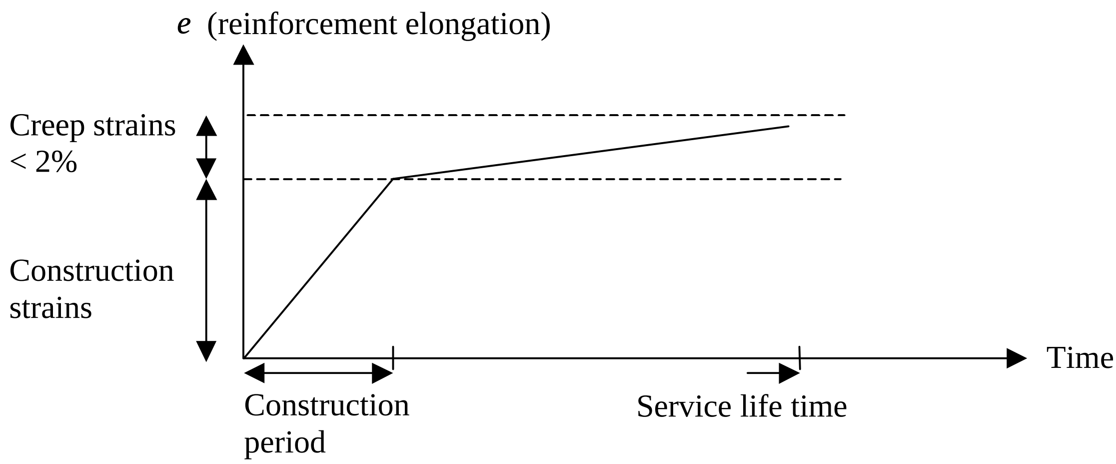


Figure 4.11 Construction strains and creep strains

Vertical walls should, in order to avoid overhang walls, be designed in a way that horizontal displacements can be accepted. A slight backward tilting 1-2 degrees is recommended.

4.5 DRAINAGE AND WATER PRESSURE

The appearance of water in a reinforced fill structure might lead to unexpected failures if not properly implemented in the design. This issue should therefore be given special attention in any project. For any structure temporary or permanent, it is recommended to evaluate the magnitude of water pressure on the structure.

Parameters that have an influence on the existence of water in the structure and include the following:

- Surface run off – water entering the reinforced fill from above
- Internal water table – water entering the reinforced fill from behind/inside
- The use ofcohesive fill

For vertical or near vertical walls and abutments with a facing it is normally recommended only to use well-drained friction fill and further to ensure the implementation of an effective drainage system in the structure.

By looking at the individual stress profiles for at typical soil profile based on traditional theory, see Figure 4.12 it is well known that the effective stress level, σ', will be influenced by the groundwater level (GWT) and the capillary level (CWT).
The effective vertical stress level is defined by s'_{vd} as:

$$s'_{vd} = s_{vd} - u_d \qquad (4.11)$$

In the capillary zone the effective stress is increased, while the effective stress is reduced below GWT. The increased effective stress should only be used to calculate additional active earth pressure and not to calculate an increased pull-out resistance or sliding capacity.

When calculating the total horizontal pressure the water pressure has to be added to the active earth pressure.

$$p_{ad} = (K_{ad} \cdot s'_{vd}) + u_d \qquad (4.12)$$

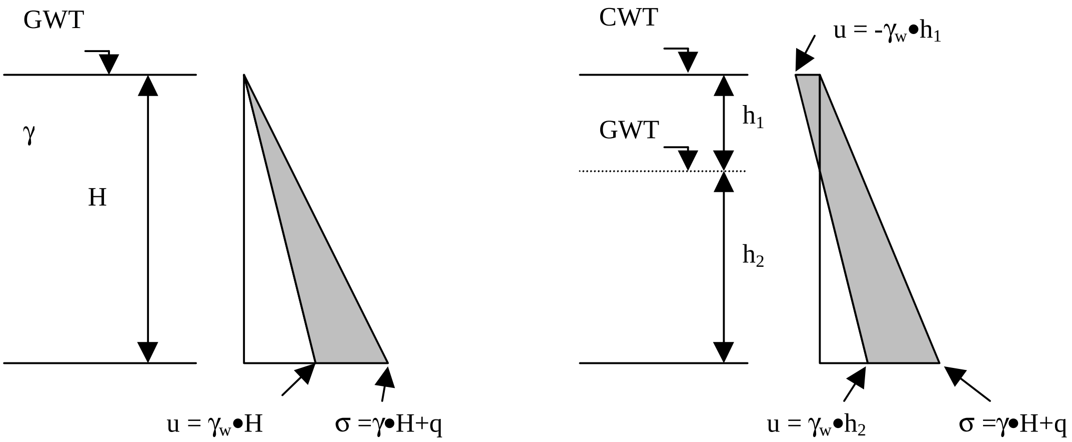


Figure 4.12 Two typical vertical stress profiles for fully saturated soils. Soil thatmobilises a certain level of capillarity will have an increase in the effective stress level, illustrated in the right hand picture (only cohesive soils as clay and silt).

Another way of handling pore water pressure is based on the R_u-coefficient defined as:

$$R_{ud} = \frac{g_w \cdot h_w}{g_d \cdot h} \qquad (4.13)$$

or

$$R_{ud} = \frac{u_d}{g_d \cdot h} \qquad (4.14)$$

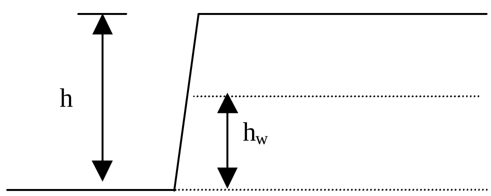


Figure 4.13 Definition of Ru-coefficient

The R_u-value varies in the range from 0.0 (dry fill) till 0.5 (fully saturated fill). Quite a few design methods include special diagrams given the relation between R_u and the active earth pressure.

Water pressure should always be implemented for the worst case during construction and serviceability of the reinforced slope/wall.

4.6 FACINGS

In relation to aesthetic matters, facing is a very individual issue, which in practice is related to the surface, way of integration, tolerances, planting *etc*.

Facings can be divided into the following groups:

Table 4.3 Definition of facings.

Soft	"*e.g.* wrap-around structures etc."
Flexible	"*e.g.* gabions, steelmesh based facing units etc."
Hard	"*e.g.* segmental block walls etc."

Facings are produced in a great number of varieties, which include lots of different materials. Most facing systems include connections between the facing unit and the reinforcement. Any joint should in general be properly evaluated for the specific facing system.

Facing systems should only be used if their suitability as a facing has been proven by comparable experience. Other facing units can be used provided that the serviceability of the system and the durability of the materials used can be proven by tests, *c.f.* draft prEN 14475.

General information about facing systems is described in the draft prEN 14475– Execution of Special Geotechnical Works – Reinforced Fill.

Generally the facings should be properly implemented in the design and evaluated according to Chapter 4.3.2. Many design method consider the facing units as an integrated part of the structure. This leads to a simplified design geometry, based upon the geometry of the reinforced fill and the facing. Not all facings allow this way of integration, and this issue should always be properly examined.

4.7 DURABILITY

In general any design should be properly evaluated in such a way that the durability complies with the design lifetime according to Chapter 2. The evaluation should focus on all relevant issues, including reinforcement, facing units and other structural components in the reinforced structure.

Depending on the type of structure the following aspects should be examined:
- Durability of the reinforcement (biological and chemical degradation)
- Durability of the facing unit based on frost, corrosion and UV-degradation
- Durability in case of fire (whole structure)
- Durability in case of mechanical damage or vandalism (primarily the facing units)
- Vegetation (relevant to evaluate in connection to UV-degradation of synthetics reinforcement and geotextiles exposed to direct sunlight *etc*.)

For steep slopes the vegetation is the most important issue in relation to durability. The protecting effect of the vegetation minimises the UV-degradation of geotextiles and synthetics reinforcement on the surface of the slope.

For walls, abutments *etc*. including the connection between the reinforcement and the structural facing unit *e.g*. modular concrete block, gabions *etc.* the durability must be evaluated as one unit.

4.8 EXECUTION, QUALITY CONTROL AND PROCUREMENT

These guidelines describe execution, quality control and procurement in the following chapters:

- Execution — Chapter 8
- Quality control: — Chapter 9
- Procurement: — Chapter 10

Based on the specific design it is recommended to ensure that any open question regarding project responsibility, execution and quality control is clearly defined in the contract..

5 EMBANKMENTS ON SOFT SUBSOIL

5.1 INTRODUCTION

5.1.1 Function of the reinforcement

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), illustrated in Figure 5.1.

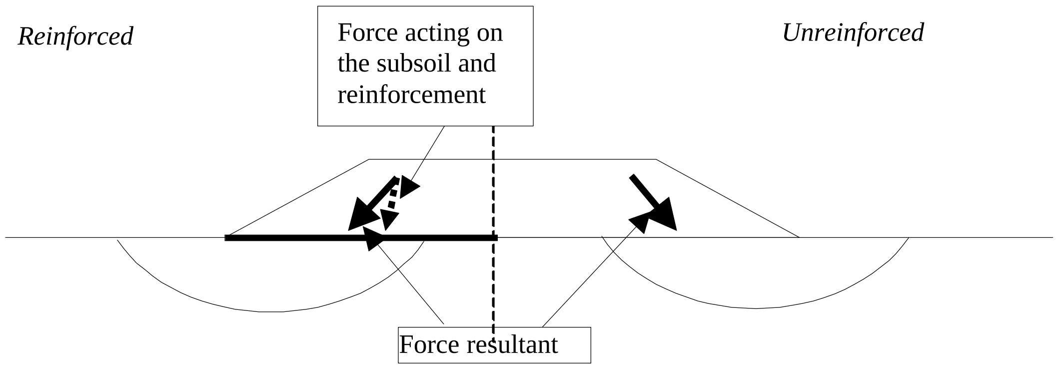


Figure 5.1 The effect of the reinforcement

Limitation: Maximum bearing capacity: $q = N_c{*}c_u$
Unreinforced: $r = 1 \Rightarrow N_c = 2.8$
Reinforced: $r = 0 \Rightarrow N_c = 5.14$

i.e. maximum theoretical improvement is 83 percent (The total horizontal force component is taken as tensile force in the reinforcement).

5.1.2 Calculation principles

The calculation method described is similar to the method in British standard BS8006. The formulas are given for the case of reinforcement between the embankment fill and the soft foundation soil. This method controls the initial stability of the embankment, but the settlements are not controlled.

In addition to the formulas given, it is important that the user defines the Limit States in order to design the reinforced embankment. Permanent actions should always be included. Traffic loads (variable actions) may not be included in the Ultimate Limit State on long term basis, but short term stability and tensile strength should be checked for large variable actions (*e.g.* railroads in the case of relatively low embankments).

5.2 SPECIFIC INFORMATION NEEDED FOR DESIGN

For the design of a reinforced embankment structure it is necessary to have a reasonable level of information available to get an accurate design.

The safety level of the structure is depending on the accuracy of the information regarding soil, fill, groundwater conditions, actions, reinforcement etc. It is of great importance to check the external stability as well as the internal stability.

5.2.1 Design life time

The design lifetime should be considered when calculating the reinforcement design strength. The use of reinforcement reduces the mobilisation of shear strength inthe subsoil and hence increases the bearing capacity. The use of reinforcement may commonly increase the bearing capacity in the range of 30-50 percent depending on the type of subsoil. The most critical situation is generally at the completion or shortly after the 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 5.2, but settlements will increase the tensile strain and hence the load, in the reinforcement. therefore the reduction in reinforcement load due to consolidation and increase of the stability of the embankment can not always be accounted for.

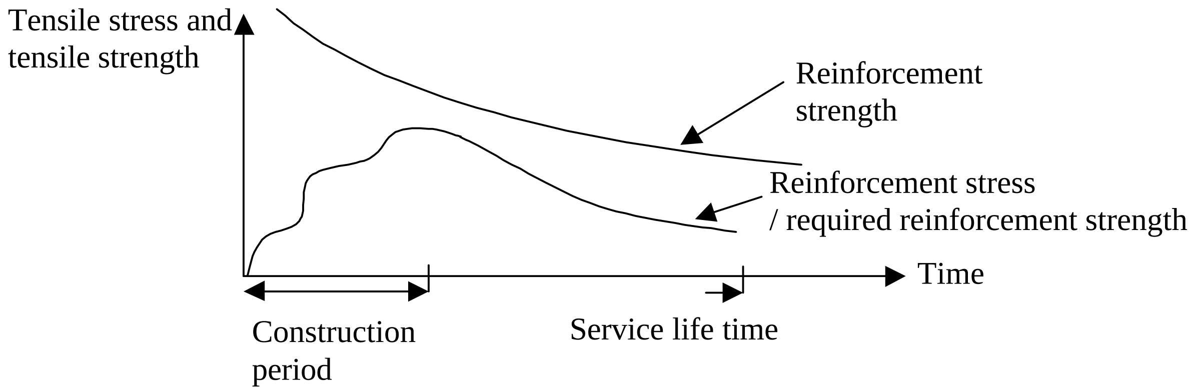


Figure 5.2 Required reinforcement strength

5.2.2 Geometry and foundation properties

Low embankments, less than 2 metres, may be evaluated based on limited information about geometry and foundation properties (however, some information *is* required). To evaluate a high reinforced embankment application, a detailed description of the geometry and the actual foundation properties must be carried out including information about groundwater etc. Considerations related to *e.g.* pipelines in the ground should be included to avoid damaging settlements.

In the Eurocode there are no partial factors for geometrical uncertainties, but in the designs it is necessary to include relevant tolerances on all the geometrical input data.

Soil properties in general should be examined properly, in a way that any critical stability problem can be avoided.

5.3 ULTIMATE LIMIT STATE DESIGN

5.3.1 Failure modes

The different types of problems to be considered in the Ultimate Limit State are as follows:

- local stability of the embankment fill (see Figure 5.4)

- lateral sliding stability of the embankment fill (see Figure 5.5)
- rotational stability/overall stability of the embankment (see Figure 5.8)
- foundation extrusion stability of the embankment fill (see Figure 5.6 and Figure 5.7)

5.3.2 Design values and design loads

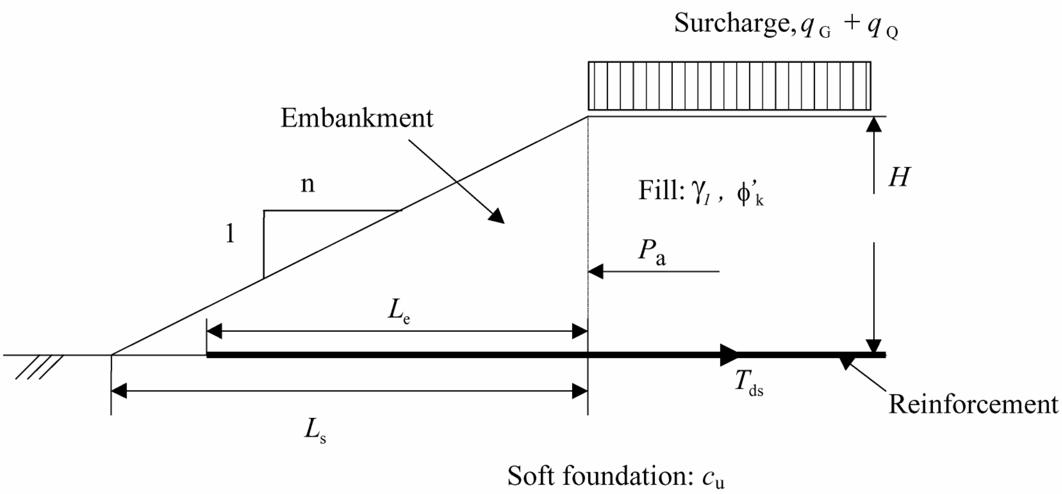


Figure 5.3 Typical parameters for design

Typical parameters are shown in Figure 5.3 and Table 5.1. Typical design parameters for a structure are given by the characteristic values reduced with partial factors given according to Chapter 3.

Table 5.1 Typical geotechnical design parameters for a structure.

Characteristic values	Partial factor	Design parameters	Unit
γ_k –value for unit weight of fill/soil	$\gamma_\gamma = 1.0$	γ_d	kN/m^3
ϕ_k - value of friction angle	$?_\phi$ (tan ϕ_k)	ϕ_d	°
c_{uk} – undrained shear strength	γ_{Cu}	c_{ud}	kPa
c'_k – cohesion intercept	$\gamma_{c'}$	c'_d	kPa
q_G , q_Q –surcharge load	γ_G , γ_Q	q_G , q_Q	kPa

5.3.3 Design step by step

The formulas given do not include any risk factor as in ENV 1997-1 (Eurocode 7). However, some national standards apply a risk factor on either the action or the reinforcement and it is left to the user to include such factor according to the standard used, see further Chapter 3 with Annex.

5.3.3.1 The design tensile force (the maximum Ultimate Limit State tensile force) T_r:

The design force (maximum Ultimate Limit State tensile force) T_r to be resisted by the basal reinforcement is the greater of:

a) the maximum tensile force needed to resist the Rotational Limit State T_{ro} per metre 'run' (see section about Rotational stability); or
b) the sum of the maximal tensile force needed to resist lateral sliding T_{ds} per metre 'run' (see section about Lateral sliding stability) and the maximum tensile force needed to

resist foundation extrusion T_{rf} per metre 'run' (see section about Foundation extrusion stability). (*i.e.* $T_{ds}+T_{rf}$).

The design strength for the reinforcement T_d , should not be smaller than the calculated design force T_r (*i.e.* $T_d \geq T_r$).

5.3.3.2 Reinforcement bond length

Necessary bond length outside the embankment shoulder, L_b, is the greater of L_b due to rotational stability, L_e due to lateral sliding and L_{ext} due to foundation extrusion. *A good practice is to install the reinforcement all the way out to the foot of the embankment and if required also with a wrap around to provide side slope stability.*

5.3.3.3 Calculation of the different failure modes

LOCAL STABILITY OF THE FILL

The local stability of the embankment sideslope should be checked according to

$$\left(\frac{1}{n}=\right)\quad \frac{H}{L_s} \leq \tan f'_d \qquad (5.1)$$

where

1/n	is the slope inclination
H	is the embankment height
L_s	is the horizontal length of the sideslope of the embankment
f_d	is the design value of internal friction angle for the fill masses

If this requirement is not fulfilled, either the slope inclination should be reduced (*i.e.* increase n) or the slope should be reinforced, *e.g.* by wrap-around (see Chapter 4).

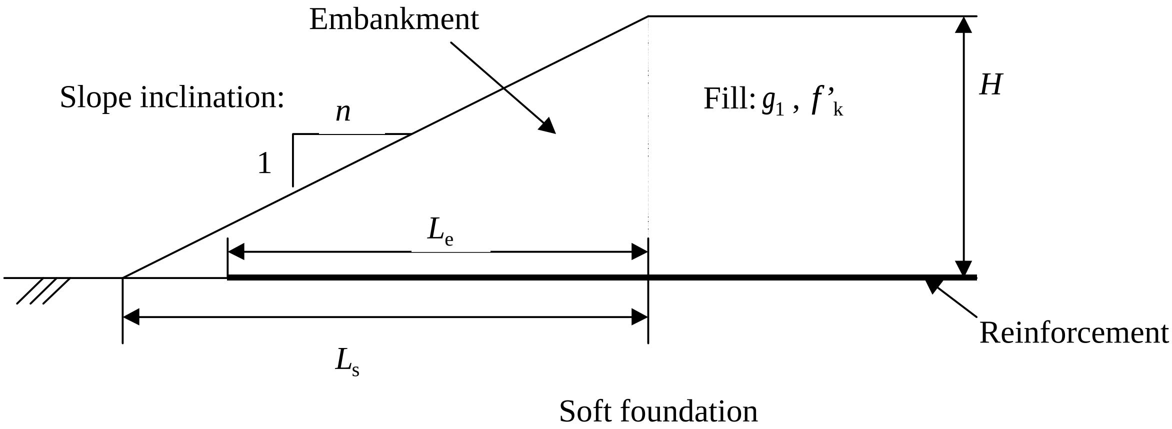


Figure 5.4 Local stability of embankment

LATERAL SLIDING STABILITY

The reinforcement should resist the horizontal force due to lateral sliding (active earth pressure). The reinforcement tensile load T_{ds} needed to resist the outward thrust of the embankment is calculated according to Figure 5.5.

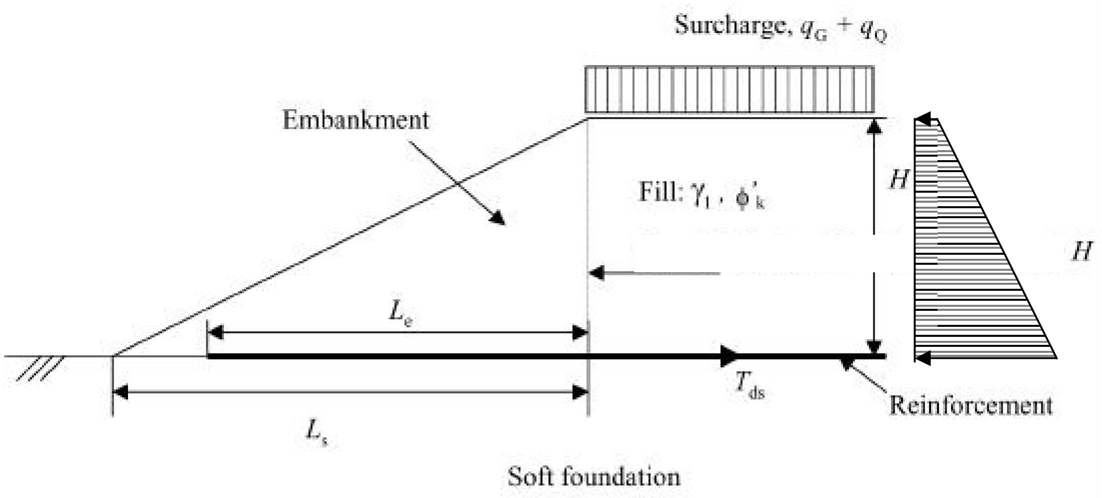


Figure 5.5 Lateral sliding stability

$$T_{ds} = P_a = 0.5K_a\left(g_{1d}H + 2(q_{Q_d} + q_{G_d})\right)H \qquad (5.2)$$

where

g_{1d} is the design unit weight of the embankment fill

q_{Qd} is the design surcharge intensity from variable load on top of the embankment

q_{Gd} is the design surcharge intensity from permanent load on top of the embankment

b is half the width of the embankment at the bottom line =(top-width)/2+n*H

H is the embankment height

K_a is the active earth pressure coefficient defined as

$$K_a = \tan^2\left(45° - \frac{f'_d}{2}\right) \qquad (5.3)$$

where

$$f_d = \arctan\left(\frac{\tan f_k}{g_f}\right) \qquad (5.4)$$

<u>Necessary reinforcement bond length:</u>

To generate the tensile load T_{ds} in the reinforcement the embankment fill should not slide outwards over the reinforcement. To prevent this horizontal sliding the minimum reinforcement bond length L_e should be:

$$L_e \geq \frac{0.5K_a H\left(g_d H + 2\left(q_{Qd} + q_{Gd}\right)\right)g_s}{g_d h\, a'_1 \tan f'_{d1}} \qquad (5.5)$$

where

h is average fill height over the reinforcement bond.

$h = H/2$ is a conservative assumption and is recommended to find whether or not a suggested slope inclination is ok (*i.e.* $h = H/2$ for $L_e = L_s$).

Iteration on h is necessary to find the minimum required bond length.

If calculated $L_e > L_s$, either the slope inclination should be reduced (*i.e*. increase *n*) or the slope should be reinforced, e.g. by the use of wrap-around of the reinforcement. *Note that increasing the slope angle may result in increased mobilisation of the subsoil at the toe of the embankment. In case of very soft subsoil the possibility for extrusion should be considered.*

FOUNDATION EXTRUSION STABILITY

The geometry of the embankment induces outward shear stresses within the soft foundation soil, Figure 5.6-5.7. Where the foundation soil is very soft and of limited depth the outward shear stresses may induce extrusion of the foundation. To prevent this extrusion the sideslope length of the embankment L_s and reinforcement bond L_{ext} have to be long enough to mobilise enough force in the reinforcement (R_R). In equation 5.6 assume $L_{ext}=L_s$ is assumed. The user has to do iterations on the subsoil layer thickness z_i to find the maximum value. Because this is a failure mode assumed possibly close to the edge in the upper subsoil layer, *it is recommended to limit the subsoil layer thickness in the calculation to maximum z_i max = 1.5×H for slope inclination in the range 1.5<n<3.0.*

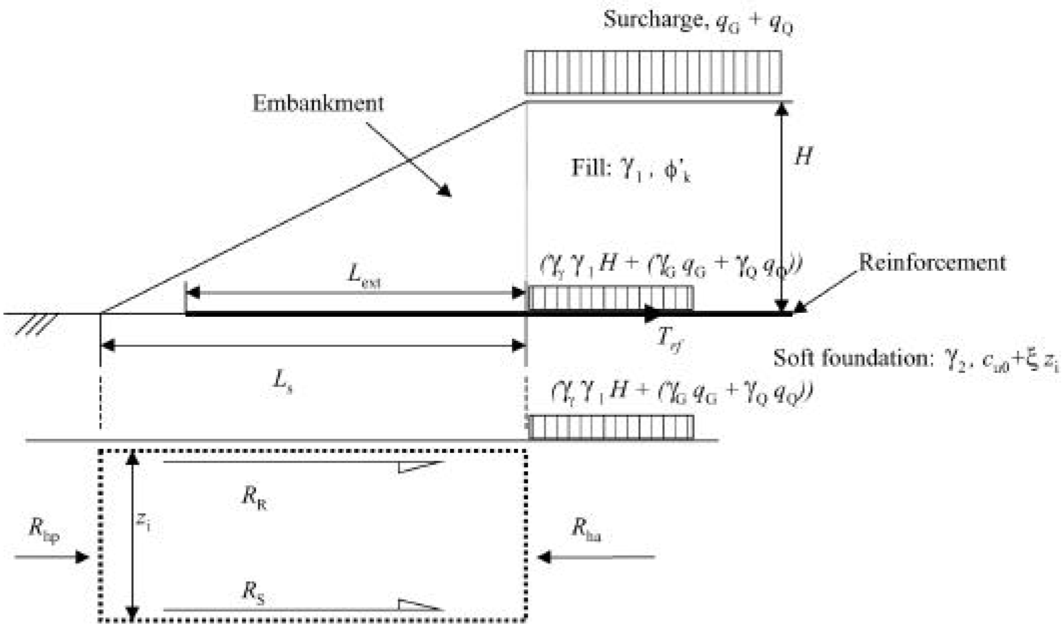


Figure 5.6 Forces in extrusion stability analyses

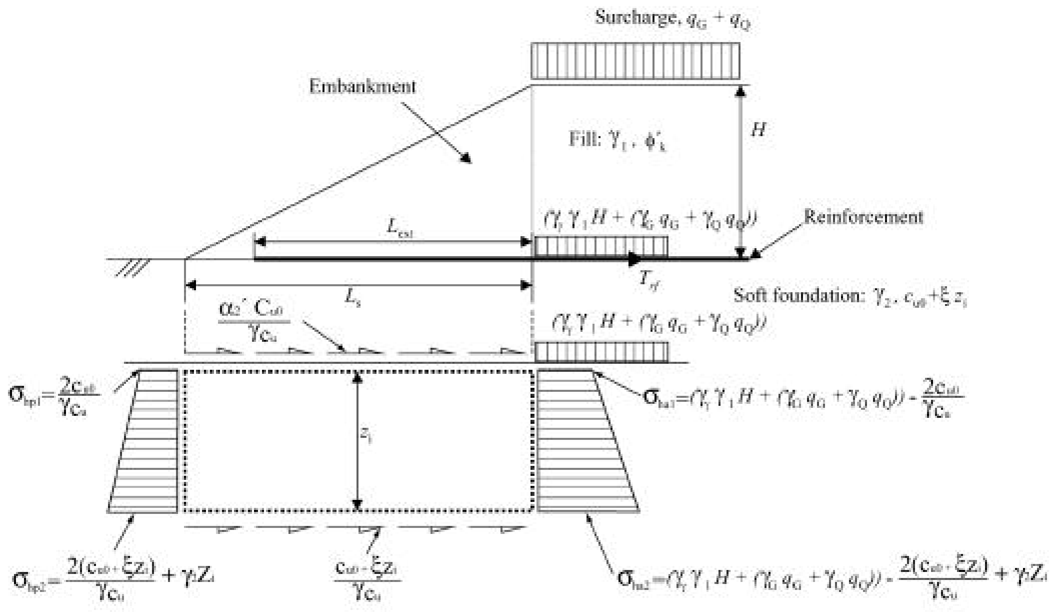


Figure 5.7 Stresses in foundation extrusion stability analyses

$$L_{\text{ext}} \geq \frac{\left(g_{d1}H + q_{\text{G}d} + q_{\text{Q}d} - \left(4c_{u0d} + 2x_d z_i\right)\right)z_{\text{i}}}{\left(1 + a_2\right)c_{u0d} + x_d z_i} \geq 0 \qquad (5.6)$$

where

z_i is the depth to the lower slip surface
If the soft foundation is of limited depth, and has constant undrained shear strength, *i.e.* $e = 0$, then set $z_i = t$, where t is the total thickness of the soft foundation layer.
For $e \neq 0$ the user has to calculate for different $z_i < t$ and find maximum required sideslope length L_{ext}

t is the thickness of the soft soil layer

c_{u0d} is the design value of the undrained shear strength of the foundation soil at the underside of the reinforcement

x_d is the increase in the design value of the undrained shear strength per metre depth below the embankment

a_2 is the interaction coefficient relating the foundation soil/reinforcement adherence to c_u . NB! strain compatibility is necessary in order to achieve a maximum interaction coefficient (sensitive foundation soil)

H is the embankment height

g_{1d} is the design unit weight of the embankment fill

q_{Qd} is the design surcharge intensity from variable load on top of the embankment

q_{Gd} is the design surcharge intensity from permanent load on top of the embankment

Necessary sideslope length L_s and bond length L_{ext} is (when assuming L_s=L_{ext}):
For constant undrained shear strength, c_u, the maximum bond length is found for z_i equal to the total layer thickness, *t*, i.e. no iterations are necessary. Be aware that for constant shear strength the extrusion force, and thereby the necessary bond length, is increasing with increasing layer thickness. Note the recommendation given prior, *"limit the subsoil layer thickness in the calculation to maximum $z_{i\,max}$ = 1.5·H for slope inclination in the range 1.5<n<3.0"*, and do not use average shear strength over a thick soil layer. Especially not if the shear strength in reality is increasing with depth.

If calculated L_{ext} >L_s , the slope inclination should be reduced (*i.e.* increase *n*).

TENSILE FORCE IN REINFORCEMENT DUE TO EXTRUSION:

The maximum required reinforcement bond length, L_{ext} (calculated for L_{ext}=L_s), is used to calculate the tensile force, T_{rf}, generated in the basal reinforcement per metre 'run' due to outward foundation shear stress, even if L_s or L_{ext} is chosen longer than the maximum required value:

$$T_{rf} = a_2 c_{u0d} L_{ext} \qquad (5.7)$$

where

c_{u0d} is the design value of the undrained shear strength of the foundation soil below the reinforcement

L_{ext} is the calculated necessary length of the reinforcement outside the embankment crest

a_2 is the interaction coefficient relating the foundation soil/reinforcement adherence to c_u . NB! strain compatibility is necessary in order to achieve a maximum interaction coefficient (sensitive foundation soil)

However, if the real, selected sideslope length is substantially longer than the required value, equation 5.7 might be conservative. The safety level in the soft soil, g_{Cu}, is increased with increased sideslope length, L_s. No simple calculation methods are available at the moment, and a computer analysis (*e.g.*. FEM) is recommended for a more accurate calculation.

ROTATIONAL STABILITY

The stability of the fill could be analysed by a conventional circular slip surface method, *e.g.* as given by Janbu *et. al.* (1956). A conventional computer program could be used (and is recommended). Janbu et. al. are giving the following formulas (adapted to the partial factor calculations):

$$F_+ = \frac{R\sum\left(\dfrac{c_d + (p-u)\tan f_d}{m_a}\,\Delta x\right)}{-T_{Rc}\cdot a_T + \sum \Delta W \cdot x} \qquad (5.8)$$

in which

$$m_a = \cos a \;(1 + \tan a \;\tan f_R / F_+) \qquad (5.9)$$

and

R is the radius of the circular slip surface.

DW	is the weight of each slice including surcharge action, and including the partial factors $g_{?}$, g_{G} and g_{Q}
c_d	in fill material: $c_d = c' / g_{c'}$ is the design value of the cohesion intercept in subsoil: $c_d = c_u / g_{Cu}$ the design value of the average undrained shear strength along slip surface
$tan f_d$	in fill material: $tan f_d = tan f_{\kappa} / g_f$ is the design value of the internal friction angle
	in subsoil: $tan f_d = 0$ (for short term stability)
p	is average vertical pressure at shear surface (no load factors applied)
u	is average pore pressure at shear surface (no load factors applied)
T_{Rc}	is the tensile force in the reinforcement required for stability
a_T	is moment arm about circle centre
F_+	is a strength reserve factor (not a lumped safety factor)

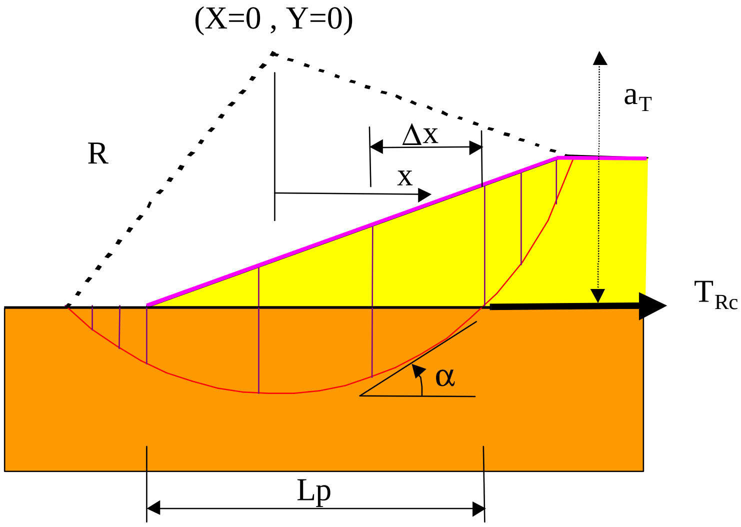


Figure 5.8 Slip circle analysis

Iterations are first made with $T_{Rc} = 0$ in order to find the critical shear surface. The calculated F_+ is a strength reserve factor (not a lumped safety factor). If $F_+ \geq 1$ is found for $T_{Rc} = 0$, the slope is stable and has the required safety.

For the case when $F_+ < 1$, the required T_{Rc} to ensure stability, i.e. to make $F_+ = 1$, has to be calculated. The resulting T_{Rc} has to be lower than the design strength for the reinforcement.

<u>Necessary reinforcement bond due to stabilising the shear surface; Figure 5.9:</u>
The reinforcement should achieve an adequate bond with the adjacent soil to ensure that the required T_{Rc} tensile loads can be generated. The necessary reinforcement bond length, L_{pj}, is calculated according to equation 5.10.

$$L_{pj} \geq \frac{g_p (T_{Rc})}{g_d h a'_{k1} \tan f'_{d1} + a'_{k2} c_{ud2}} \qquad (5.10)$$

h	is average fill height over the reinforcement bond (L_{pj})
g_{1d}	is the design unit weight of the embankment fill

The bond length should also be calculated due to the friction angle for material below the reinforcement (f'_{d2}), equation 5.11. If both f'_{d2} and c_u are determined, the larger value of equation 5.10 and 5.11 should be used.

$$L_{pj} \geq \frac{g_{p}(T_{Rc})}{g_{d} h(a'_{k1} \tan f'_{d1} + a'_{k2} \tan f'_{d2})} \qquad (5.11)$$

To find the minimum necessary bond length, the user should calculate the necessary bond length outside the embankment shoulder, L_b , for each slip circle with $T_{Rc} > 0$, see Figure 5.9. The shortest way to the embankment foot is to be used. *Note! It is not necessarily the slip circle that requires the largest T_{ds} that gives the largest L_b..*

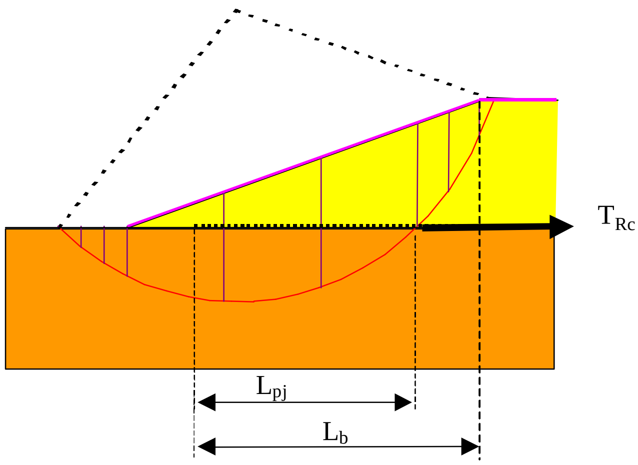


Figure 5.9 Required bond length, L_{pj} , and bond length outside the embankment shoulder, L_b.

5.4 SERVICEABILITY LIMIT STATE DESIGN

The different types of problems to be considered in the Serviceability Limit State are as follows:

- settlement of the foundation soil
- excessive strain in the reinforcement

5.4.1 Foundation settlements

The reinforcement alone does not significantly influence on the settlements of the embankment. Settlements analyses could therefore be performed using conventional procedures based on the effective stress level and modulus of consolidation.

Foundation settlements can induce strains and hence load, in the reinforcement.

5.4.2 Reinforcement strains

Strains in the reinforcement are determined from the applied loads, see Figure 5.10. Foundation settlements and construction works may also induce strains in the reinforcement, but these strains are difficult to quantify. Numerical analyses may be used for a better estimate of the real behaviour of the embankment.

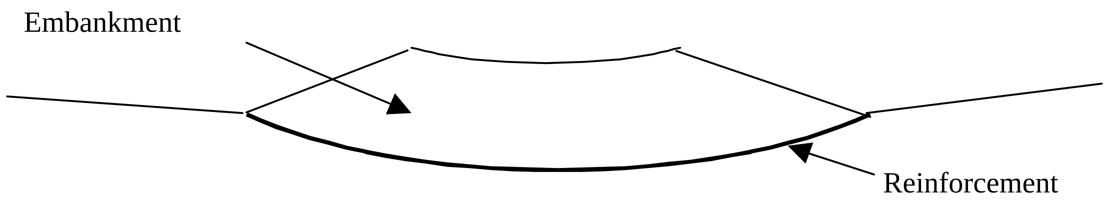


Figure 5.10 Reinforcement strain, Serviceability Limit State

The strains developed should not exceed values derived from Serviceability Limit State considerations,Figure 5.11. Normally it is not critical if the total strains creep strains included are up to 10 percent on long term basis. However, the total strains, post constructional strains as creep strains and strains due to foundation settlement included, should not during the design life time exceed 70 percent of the strain at failure for the actual reinforcement.

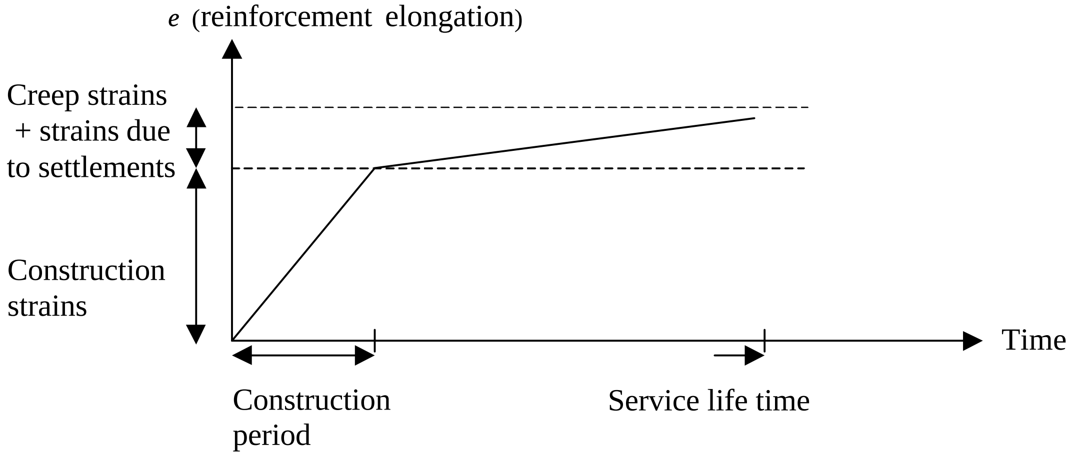


Figure 5.11 Construction strains and creep strains

If the foundation soil is brittle (*e.g.* quick clay) the reinforcement strain at failure (including the creep strains) should be limited to maximum 4 percent to ensure compatibility with adjacent soil.

5.5 DURABILITY

In most cases the shear strength in the subsoil will increase during consolidation after the construction period. The reinforcement may therefore not be necessary on long term basis. However, it is normal to ensure that the required tensile strength is available during the design life time.

5.6 EXECUTION, QUALITY CONTROL AND PROCUREMENT

These guidelines describe execution, quality control and procurement in the following chapters:

- Execution — Chapter 8
- Quality control: — Chapter 9
- Procurement: — Chapter 10

Based on the specific design it is recommended to ensure that any open question regarding project responsibility, execution and quality control is clearly defined in the contract.

6 EMBANKMENT ON IMPROVED SOIL (REINFORCED PILED EMBANKMENTS)

6.1 INTRODUCTION

For embankments on improved soil reinforcement may be used in the fill in the lower part of the embankment. It is important to decide the purpose with the reinforcement, prior to the design.

6.1.1 Function of the reinforcement

Reinforcement above lime cement columns may have two functions. For **soft columns** the function is to prevent sliding. Calculations could be done almost according to the previous Chapter 5, but a complement is needed because of the lime cement columns resistance to sliding. The load is carried both by the columns and the soil inbetween the columns, which leads to a small difference between the displacement of the columns and settlements in the soil., This results in a small strain in the reinforcement and therefore the vertical load shedding effect will also be small. . In the design of lime cement columns both settlements and sliding have to be considered. In the case where the settlements are dimensioning to the spacing of the columns and this gives a safe construction from stability point of view, then the reinforcement is unnecessary.

For **stiff columns**, see Figure 6.1, the function of the reinforcement may be both to prevent settlements of the embankment and to prevent sliding. In this case the function is the same as for **reinforced piled embankments** and calculations could be made according to this chapter. The rest of the chapter are only showing the method with reinforced piled embankments but can be used in the same manner for stiff columns.

REINFORCED PILED EMBANKMENTS

The arching effect between the pile caps reduces the incremental portion of load carried by the reinforcement and transfers the embankment loading onto the piles. The main purpose with the reinforcement above a piled embankment is to prevent settlements of the embankment.

In the Nordic countries the piles are often installed inclined, for example 4:1 beneath the embankment slopes to provide lateral support. The use of reinforcement is often an economic solution to reduce the size of the pile caps and it may also make it possible to avoid the inclined piles.

6.1.2 Calculation principles

There are different calculation models for reinforced piled embankments that can be used for design. In Sweden a comparison has been made between the British Standard BS 8006 (1995) and the model in these guidelines. The results showed that the suggested model gives a better agreement with finite element calculations for the degree of cap coverage normally used in Sweden than BS 8006, Rogbeck et al (references from 1995-2000). In Norway the foundation SINTEF also has a model for design, Svanø *et al* (2000). The results are comparable with the suggested model when the restrictions of the suggested model are considered.

In this chapter the calculations are related to the function to prevent settlements and lateral sliding. If the reinforcement also is used to control stability, calculation for this part should be made according to Chapter 5. This chapter shows the design of the reinforcement, the piles and pile caps should be designed according to national regulations. Load case C according to ENV 1991-1, is used in the design, for further information see Chapter 3.

Det finns modeller som beräknar armering i flera lager. Det är då viktigt att ta hänsyn till att töjningarna skiljer sig åt i de olika lagren.

The model in the guidelines has been used for design with geosynthetic reinforcement. It may be applicable also for steel reinforcement, but there are no documented practical experience on this matter.

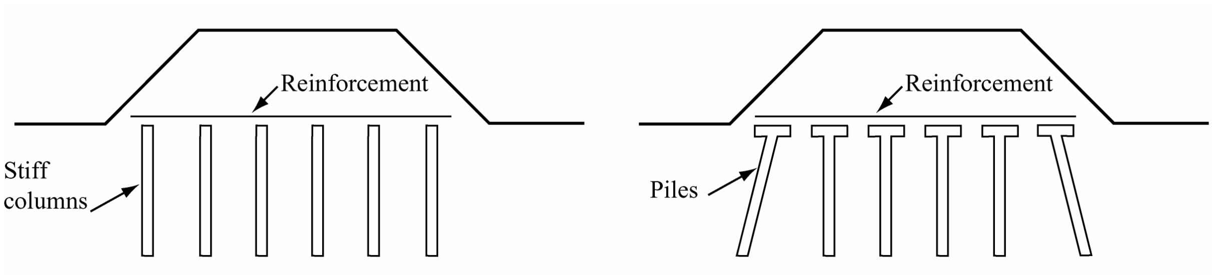


Figure 6.1 Stiff columns and piled embankment with basal reinforcement.

6.2 SPECIFIC INFORMATION NEEDED FOR DESIGN

The safety level of the structure is depending on the accuracy of the information regarding soil, fill, groundwater conditions, porewater pressure, loads, reinforcement, design life etc.

More information about the material properties is given in Chapter 2. The design formulas are using a principle of partial factors of safety, which is described in general in Chapter 3. Below the parameters needed for design of a reinforced piled embankment are given.

6.3 ULTIMATE LIMIT STATES DESIGN

Principles for the Limit Statedesign is defined in ENV 1991-1 and other national standards. The Limit State design is based on partial factors of safety applied on loads and material properties of the reinforcement and geotechnical parameters.

6.3.1 Failure modes

The Ultimate Limit States to be considered are 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, see Figure 6.2. 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. Calculations of lateral sliding and vertical load shedding are described in this chapter.

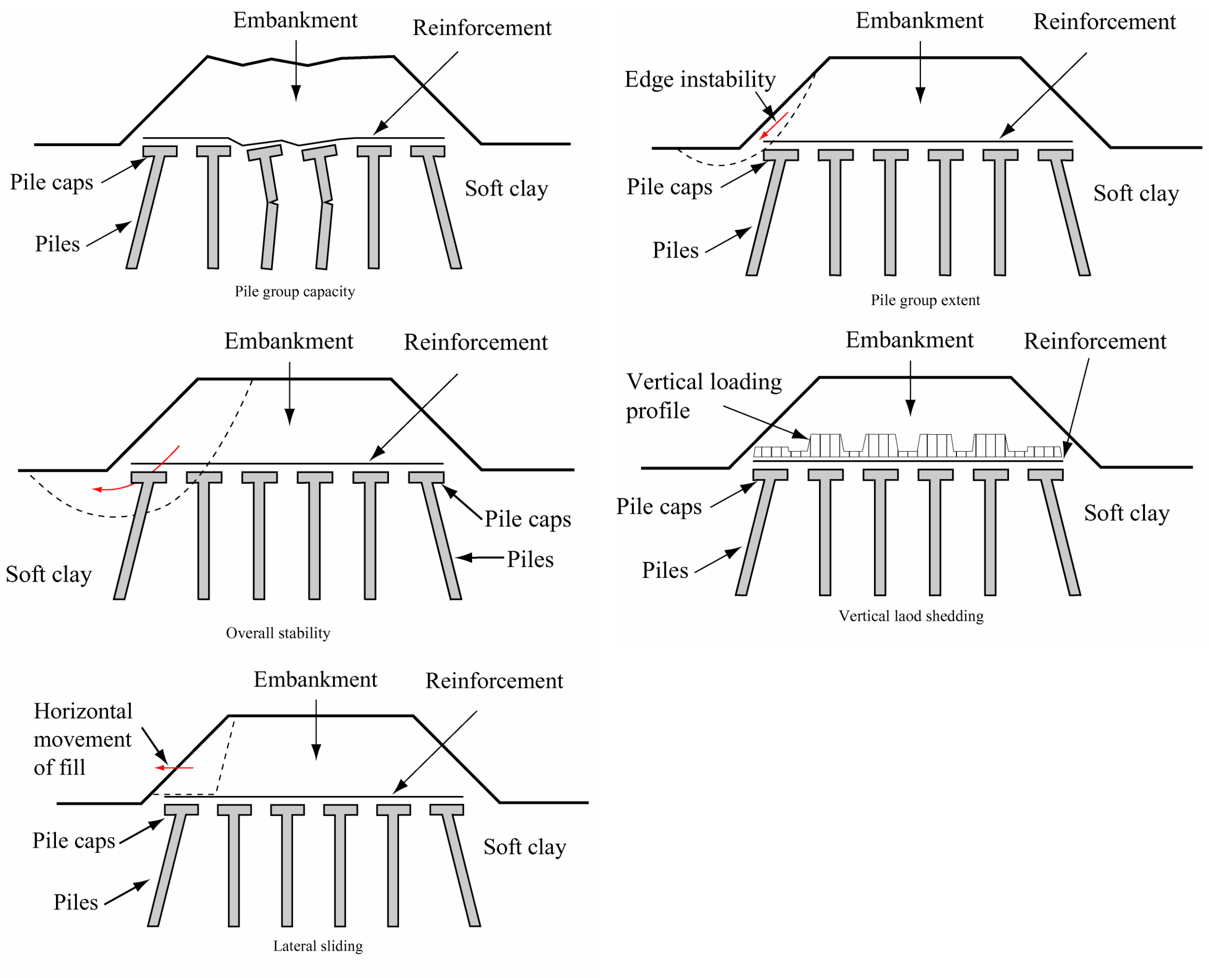


Figure 6.2 Ultimate Limit States for reinforced piled embankments.

6.3.2 Design values and design loads

Typical design parameters are shown in Table 6.1. Design parameters for a structure are given by the characteristic values reduced with partial factors given according to Chapter 2 and 3.

Table 6.1 Geotechnical design parameters for a structure.

Characteristic values	Partial factor	Design parameters	Unit
γ_k –value for unit weight of fill/soil	$\gamma_\gamma = 1.0$	γ_d	kN/m^3
ϕ_k - value of friction angle	$?_\phi$ (tan ϕ_k)	ϕ_d	°
q_G , q_Q – surcharge load	γ_G , γ_Q	q_G , q_Q	kPa

6.3.3 Partial factors of safety

The design formulas are using the principle of partial factors to give the structure a proper safety against collapse. The principle is described in Chapter 3.

In this application the creep behaviour in the reinforcement is very important to prevent settlements after the construction time. The reinforcement properties should therefore be properly evaluated based on the characteristics given from the supplier, independent

research institutes or national certificates, approvals etc., see Chapter 2. The long-term characteristic strength has to be evaluated by long-term creep test and therefore the conversion factor for creep behaviour is equal to 1.0.
Partial factors for damage during construction could be determined by tests on reinforcement and soil. Normally the tests are carried out on a certain reinforcement in different test soil types. There are no European standards for this type of tests and the designer has to evaluate if the methods used give reliably results for design. In complex designs where this kind of tests have not been done before it could be of interest to perform the tests on the material used in the specific project. If information from the relevant reinforcement and soil is not available, typical values based on experience may be used, see Chapter 2. .

6.3.4 Restrictions of the model

The calculation assumes an arch formation and that the reinforcement is deformed during loading. The model is based on a reinforcement placed in one layer, but an approximation is given for reinforcement in two layers. The function of the reinforcement is greatest if it is placed onto the pile caps, but it should for practical reasons be about 0.1 m above the pile caps. In order to ensure that the deformation of the road surface will not be too large, the embankment height should be at least as large as 1.2 the distance between the pile caps. Which also could be expressed as the maximum distance between the pile caps should not be greater than $0.8 \cdot H$. The degree of cap coverage should be at least 10 percent.

The model pressumes a top angle of 30º of the arch 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 greater friction angles the needed strength in reinforcement is lower than calculated in this model. Fill material with a smaller friction angle should not be used in this type of construction

It is recommended that calculations should be carried out for an initial strain of a maximum of 6 percent and with a remaining creep strain after the construction period and during the lifetime of the construction of an additional 2 percent at the most. The same E-modulus is used both in Ultimate Limit States and Serviceability Limit States. The strain has to be checked for the specific product and compared with the design strength at the chosen strain. The total strain should not during the design lifetime exceed more than 70 percent of the strain at failure for the actual reinforcement.

Tolerances for the centre distance between the piles should be considered and the design distance should be chosen as the worst case according to accepted tolerances.

If more than one layer of reinforcement is considered or a lower embankment height than the restriction of the model, it is recommended that finite element calculations are used.

The analytical calculation model proposed is judged reasonable if there is a risk of cavities arising under the reinforcement, a future change of the load situation by *e.g.* groundwater 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 achieved if finite element calculations are used to model the complex interaction behaviour.

6.3.5 Design step by step

The calculations below are made for the design of the reinforcement considering horizontal force and vertical load shedding. Lateral sliding due to the horzontal force is only relevant if vertical piles are used beneath the embankment slopes. The local stability of the embankment side slope outside the pile caps should be checked according to Chapter 5. Figure 6.3 shows the symbols used in the calculation model.

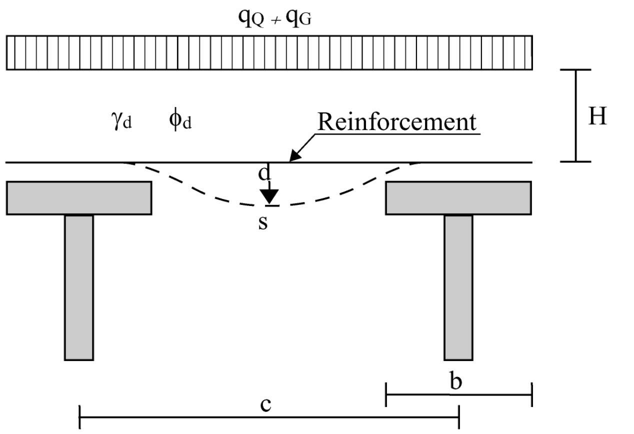


Figure 6.3 Symbols used in the calculation model.

SYMBOLS SPECIFIC TO THIS CHAPTER:

H	embankment height (m)
b	pile cap width (m)
c	centre distance between piles (m)
d	displacement (m)
s	arc length (m)

6.3.5.1 Design of horizontal force

If vertical piles are used beneath embankment slope instead of inclined piles, see Figure 6.4, the tensile force in the reinforcement can be calculated in the same way as described in Chapter 5, Lateral sliding stability (active soil pressure):

$$T_{ds} = P_{ad} = 0.5K_{ad}\left(g_d H + 2(q_{Q_d} + q_{G\,d})\right)H \qquad (6.1)$$

$$K_{ad} = \tan^2(45 - \frac{f_d}{2}) \qquad (6.2)$$

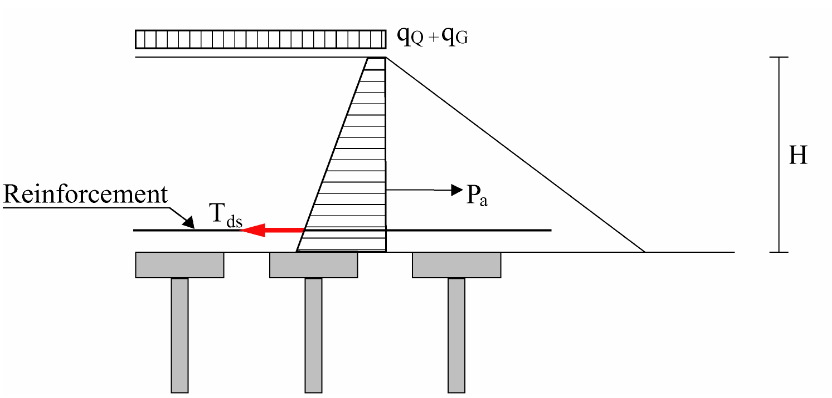


Figure 6.4 Horizontal force in reinforcement using vertical piles beneath embankment slope.

6.3.5.2 Design of vertical load transfer

The method is based on the formation of an arch inbetween the pile caps., 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 using the soil wedge described in Figure 6.5. 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 should be maximum 6 percent or less if there is a risk of exceeding the maximum of 2 percent creep strain. The total strain should not exceed 70 percent of the strain at failure for the actual reinforcement.

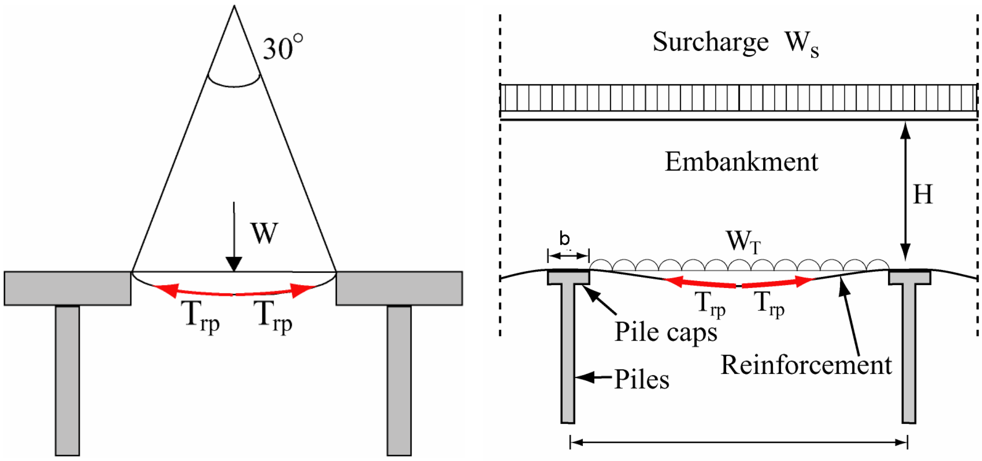


Figure 6.5 The soil wedge carried by the reinforcement.

The weight of the soil wedge, W, according to Figure 6.5 is:

$$W_{2Dd} = \frac{(c-b)^2}{4 \cdot \tan 15°} \cdot g_d = 0{,}93(c-b)^2 \cdot g_d \quad \text{kN per metre in length} \qquad (6.3)$$

The three-dimensional effects are estimated through load distribution according to Figure 6.6. The reinforcement transfers the load to the pile caps. The weight of the soil in three dimensions, W_{3D}, is calculated as follows:

$$W_{3Dd} = \frac{1+\frac{c}{b}}{2} \cdot W_{2Dd} \qquad (6.4)$$

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

$$s = (1+\varepsilon)(c-b) \approx c-b+\frac{8}{3}\frac{d^2}{c-b} \qquad (6.5)$$

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

$$d = (c-b)\sqrt{\frac{3}{8}\varepsilon} \qquad (6.6)$$

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 for 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 experiences in the Nordic countries that shows that a larger displacement can be tolerated.

The force, $T_{rp\ 3D}$, in the reinforcement due to the vertical load in three dimensions according to Figure 6.5 and Figure 6.6, is calculated using the equation:

$$T_{rp3D} = \frac{W_{3Dd}}{2} \cdot \sqrt{1 + \frac{1}{6e}} \qquad (6.7)$$

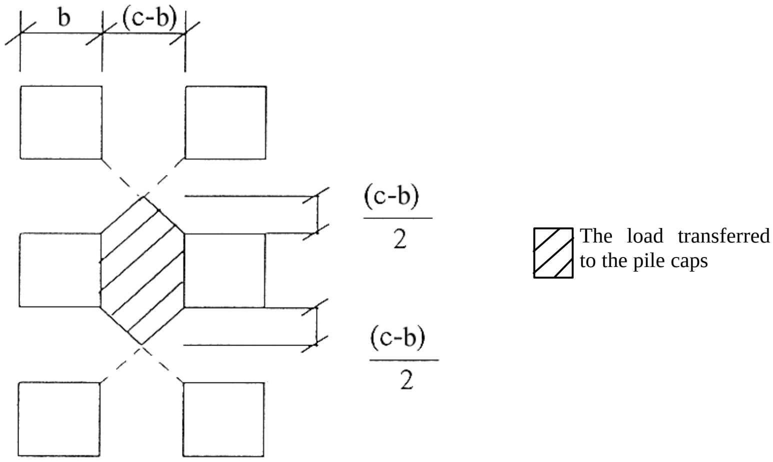


Figure 6.6 Load distribution to estimate the forces in the three-dimensional case

6.3.5.3 Design of total force

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

$$T_{tot} = T_{ds} + T_{rp\,3D} \qquad (6.8)$$

If the force, T_{Rc} from rotational stability calculation according to Chapter 5 has shown to be greater than T_{ds} the total force, T_{tot} , should be the sum of the force, T_{Rc} and T_{rp3D} .

In the calculations the strength of the seam has to be considered. If overlapping is used instead of seams the calculations for transverse sliding and pull-out in this chapter might be used. In that case the friction between the reinforcement layers in the reinforcement overlap has to be considered.

6.3.5.4 Design of reinforcement

Two principles apply to the design of the reinforcement:

- during the 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:

$$T_d = T_{cr} \cdot h_1 \cdot h_2 \cdot h_3 \qquad (6.9)$$

or

$$T_d = T_{cs} \cdot h_1 \cdot h_2 \cdot h_3 \qquad (6.10)$$

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	according to Chapter 2

The design strength of the reinforcement should be greater than 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 is recommended that they are placed close to each other, but not on top of each other due to loss of friction, a distance of 0.1 metres could be chosen. The additional design strength because of the second layer may approximately be chosen as 40 percent of the first layer. If more economical solutions should be achieved with two layers finite element calculations are recommended.

Depending on the embankment height the reinforcements *frost durability* has to be considered.

6.3.5.5 Design of the bond length of the reinforcement

The reinforcement should achieve an adequate bond with the fill at the outer edge of the piled area and all vertical sections should be verified. For the necessary reinforcement length, illustrated in Figure 6.7, on account of transverse sliding and pull-out force across the bank, the bond length$_b$, of the reinforcement can be determined according to the following calculations

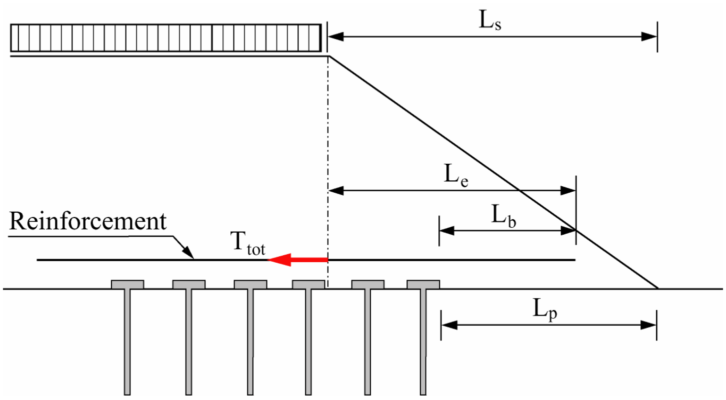


Figure 6.7 The bond length according to transverse sliding across the bank and the pull-out length of the reinforcement

Where:

L_e	bond length due to transverse sliding
L_b	bond length due to pull-out force
L_s	is the horizontal length of the sideslope of the embankment

REQUIRED BOND LENGHT DUE TO TRANSVERSE SLIDING

Necessary bond length, L_e, due to transverse sliding across the bank can be calculated as:

$$L_e \geq \frac{T_{ds} \cdot g_s}{g_d h a \tan f_d} \tag{6.11}$$

$$L_e \geq \frac{0.5 K_{ad} H \left(g_d H + 2\left(q_{Qd} + q_{Gd} \right)\right) g_s}{g_d h a \ \tan f_d} \tag{6.12}$$

where:

h   average height of fill above reinforcement
H/2 is a conservative assumption and is recommended used to find whether or not a suggested slope inclination is ok (*i.e.* h=H/2 for $L_e = L_s$). Iteration on h is necessary to find the minimum required bond length more exactly.

If calculated $L_e > L_s$, either the slope inclination should be reduced or the slope should be reinforced using for example wrap around.

REQUIRED BOND LENGTH DUE TO PULL-OUT FORCE

Necessary bond length, L_b, due to the pull-out force across the embankment, , is calculated as:

$$L_b \geq \frac{(T_{rp3D} + T_{ds})\gamma_p}{\gamma_d h(\alpha_1 \tan\phi_{d1} + \alpha_2 \tan\phi_{d2})} \qquad (6.13)$$

where

$\tan\phi_{d1}$ design friction angle above reinforcement
$\tan\phi_{d2}$ design friction angle beneath reinforcement[20]
h   average height of fill above reinforcement

The corresponding necessary bond length along the embankment could be calculated by the same equation where $T_{ds}=0$.

If it isn't possible to achieve the adequate bond length some solutions are suggested:

- flatter slopes
- wrap-around with reinforcement
- use a row of gabions as a thrust block and wrap the reinforcement around the gabions

6.4 SERVICEABILITY LIMIT STATES DESIGN

In the Serviceability Limit States excessive strain in the reinforcement and settlement of the piled embankment have to be considered. The Ultimate Limit States will be dimensioning for the tensile strength in the reinforcement when restricting the strains in the reinforcement in the calculations.

6.5 DURABILITY

The reinforcement has to be chosen to ensure that the required tensile strength is available during the design life time. Polyester is more sensitive to pH-values greater than 9 than other polymers and that has to be considered when placing the reinforcement above the pile caps.

[20] The possibility to take the friction angle below the reinforcement into account might be reduced and it could be better to neglect this part or to use values for the foundation soil

6.6 EXECUTION, QUALITY CONTROL AND PROCUREMENT

These guidelines describe execution, quality control and procurement in the following chapters:

- Execution Chapter 8
- Quality control: Chapter 9
- Procurement: Chapter 10

Based on the specific design it is recommended to ensure that any open question regarding project responsibilty, execution and quality control is clearly defined in the contract.

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 sub-sections.

7.1 EXCAVATED SOIL NAILED SLOPE

The design of a soil nailed excavated slope comprises the following
1. selection of length, type and spacing of soil nails, which most commonly are based on an analysis of external and internal stability (ultimate limit state design)
2. comparison of acceptable displacement versus expected displacement (serviceability state design)
3. design of facing
4. design of drainage
5. consideration of durability requirements for the nails and facing
6. consideration of the adaptation of the structures to the environment

The different steps are further discussed below.

7.1.1 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. The extent of the geotechnical site investigation should be thorough enough to guarantee that the site characteristics could be determined in accordance with the requirements in ENV 1997-1. It is especially important to note layers with different soil characteristics.

One of the most important parameters for design is the groundwater and surface water situation at the site. Neglecting the problematic with water may result in failure of the structure.

As for all other structures a number of different loads and load combinations could be applied to a soil nailed structure. The following loads should be considered, if they are applicable:
- Permanent action
- Variable action
- Seismic load (accidental load)

For excavated slopes, both the final geometry and the geometry of each excavation step is a result of the design process. The input to the design is the client's request of height and ex-tension of the wall.

Based on information about the design life, ground conditions and the wall geometry a suitable soil nail system is proposed. To finalise the design information about the strenght of the reinforcing element, the installation techniques, durability, geometry of the nail and pullout capacity is needed.

In Chapter 2 the different materials are further described and suitable requirement discussed. In Table 7.1 necessary information for the different design steps is summarised.

Table 7.1 Information needed for design

Design step	Soil information (ϕ, γ, c, C_u, w_l,)	Soil electrochemical information (pH, resistivity, chlorides)	Groundwater (gw, pp) and surface water	Loads	Wall geometry	Soil Nail system (strength, installation technique)
Ultimate limit state design	X		X	X	X	X
Serviceability state design	X			X	X	
Facing	X		X	X	X	
Drainage	X		X	X	X	
Durability	X	X	X			X
Adoption to environment					X	

For construction in urban areas it is important to obtain information about foundation of neighbouring structures and if there are any installations in the ground that might interfere with the soil nailing.

7.1.2 Ultimate limit state design

7.1.2.1 Failure modes

The failure in a ultimate 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 are relevant for a reinforced wall may also be applicable for a soil nailed wall (bearing capacity below the wall, tilting, sliding and overall stability). These failure modes have been discussed in previous chapters and will not be discussed further here. However, they should not be neglected in the design.

7.1.2.2 Design values and design loads

Design values for the soil and the soil nails are chosen in accordance to the suggestions in Chapter 2 and 3.

7.1.2.3 Design step by step

Ultimate limit stage design includes the following;

1. Preliminary layout of the soil nails and choice of soil nailing system
2. Stability analyses
3. Verification of the chosen soil nailing system
4. External failure modes and global stability
5. Stability analysis of each excavation phase

1. PRELIMINARY LAYOUT

A preliminary choice of soil nailing system is made based on information about the site and what type of construction that is requested. The answear to the following questions give an indication of what system that should be chosen;

- Permanent structure or temporary? For permanent structures grouted nails may have an advantage considering the durability but a driven nail with sacrificial thickness may be sufficient.
- Self-supporting soil or not? This indicates what kind of flushing medium that should be used and if casing should be applied or not.
- Available systems.
- Type of soil. In case of boulders it may be less suitable to use a driven nail.
- Environmental aspects. Adaptation to the surroundings.
- Corrosion potential in the area. The necessity of a corrosion protection system.
- Could installation method cause increase of pore-pressure that would have a negative effect on the stability during the execution?

Next step is to make a preliminary estimate of the nail length and nail density. In the paper Ground Engineering (November, 1986) Bruce *et al.* present a number of empirical correlations which may be used. Three different parameters are defined;

1. The ratio between the length and the height of the slope, L/H
2. Available area where friction may be mobilised, $C\,L\,/S_h\,S_v$
3. The strength of the nail compared to the area it will reinforce $A\,/S_h\,S_v$

Depending on which type of nail that has been chosen typical values are given in Table 7.2. For grouted nail there is additional empirical correlation based on type of soil that is given in Table 7.3.

Table 7.2 Preliminary estimation of nail spacing, nail lenght and layout (Bruce et al, 1986)

	Grouted nail (not simultaneously drilled and grouted)	Driven
1. L[21]/H[22]	0.5 – 0.8	0.5 – 0.6
2. C[23] L / S_v[24] S_h	0.3 – 0.6	0.6 – 1.1
3. A[25] / $S_v\,S_h$	$(0.4 – 0.8)\cdot 10^{-3}$	$(1.3 – 1.9)\cdot 10^{-3}$

Table 7.3 Typical values for spacing, nail length and layout for grouted nail is different soils according to Bruce (1986).

	Granular soil	Moraine and marl
1. L/H	0.5 – 0.8	0.5 – 1.0
2. CL / S_vS_h	0.3 – 0.6	0.15 - 2.0
3. A/ $S_v\,S_h$	$(0.4 – 0.8)\cdot 10^{-3}$	$(0.1 - 0.25)\cdot 10^{-3}$

Results presented by Transportation Research Laboratory indicates that for soil nailing walls performed for roads more conservative values than the values calculated by the

[21] L is the nail length
[22] H is the effective retained height
[23] C is the characteristic circumference of the hole in which the nail and grout (if any) is placed
[24] S_v and S_h are the vertical and horizontal spacing of the nail
[25] A is the characteristic cross-sectional area of the driven nail and the grouted nail diameter for grouted nails.

empirical correlation above are used (P. Johnsson *et al*, 1998). In France nails commonly are divided into two different groups; Hurpoinise (closely spaced nails usually driven) and drilled/grouted nails more widely spaced. For the first group the nail length is about 0.5 to 0.7 times the height of the slope (*H*) and in the second case 0.8 to 1.2 *H*.

2. STABILITY ANALYSES

To verify that the assumed nail layout is sufficient traditional slope stability methods are used to analyse soil nail structures by incorporating the nail force at the intersection with the failure surface in the equilibrium equations.

Due to a small movement of the active wedge of the slope a tension force in combination with shearing/bending will be mobilised in the nail. This nail force will contribute to resist further movement of the slope.

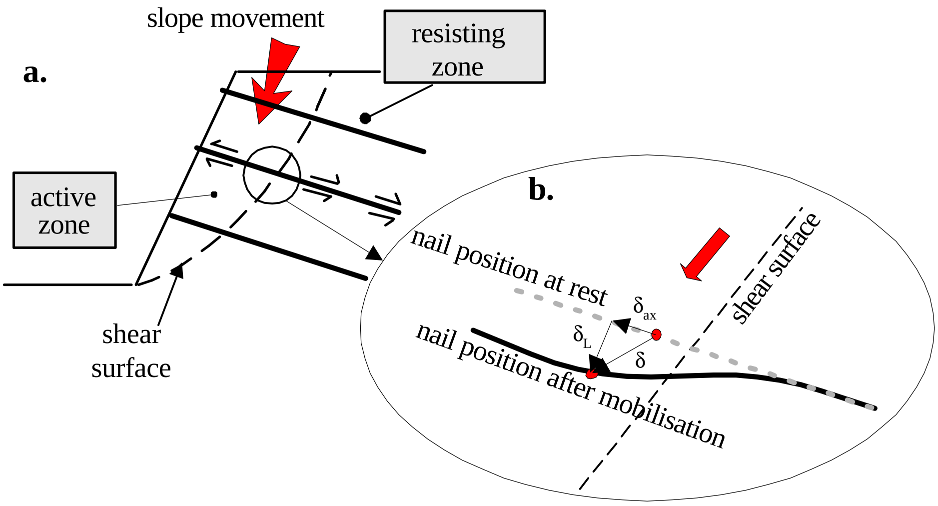


Figure 7.1 Principal behaviour of a soil nailed wall

a) active and resisting zone
b) assumed nail displacement at the shear surface

For most practical applications the available research information suggests that the contribution from the shearing/bending mobilised in the nail might be neglected, resulting in just a marginal conservatism. Consequently only the tension force is considered in this document.

The mobilised tension force can be divided into two components; one that is normal to the failure surface, P_N and one parallel to the failure surface, P_P.

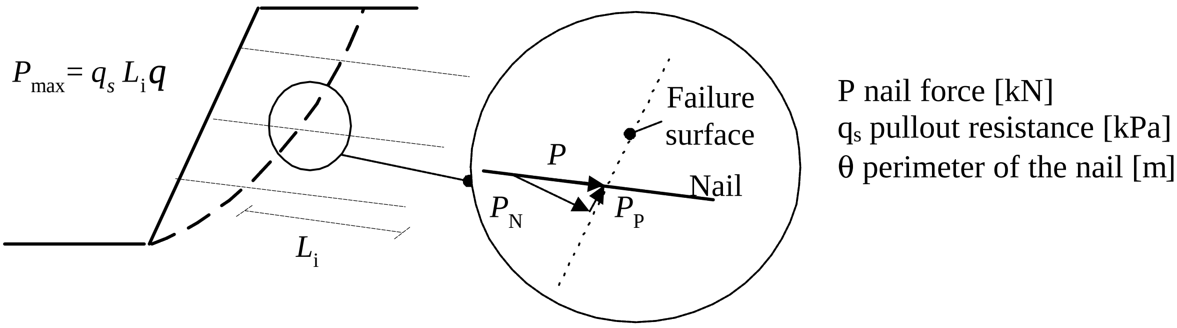


Figure 7.2 Definition of nail forces

The factor of safety can be expressed as follows according notation in Chapter 3

$$\frac{1}{\gamma_{Rd}} \cdot \left[\frac{c'_k}{\gamma_{c'}} "+" \frac{\tan \phi'_k}{\gamma_{\phi'}} "+" \eta \cdot \frac{R_{N_k}}{\gamma_{R_N}} \right] - \gamma_{S_d} \cdot \left[\gamma_\gamma \cdot \gamma_k "+" \gamma_G \cdot q_{Gk} "+" \gamma_Q \cdot q_{Q_k} \right] \geq 0 \quad (7.1)$$

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. Symbolically the equation can be written as follows. (The actual equation depends on the analysis method used; *e.g.* Bishop, Morgenstern and Price, Janbu)

$$F = \sum_i \frac{\frac{c'_{k_i}}{\gamma_{c'}} + \left(\sigma'_{N_i} + \Delta\sigma'_{N_i}\right) \frac{\tan \phi'_{k_i}}{\gamma_{\phi'}} + \eta \cdot \frac{T_{k_i}}{\gamma_T}}{\gamma_{S_d} \cdot \left[\gamma_\gamma \cdot \gamma_{k_i} "+" \gamma_G \cdot q_{Gk_i} "+" \gamma_Q \cdot q_{Qk_i} \right]} \quad (7.2)$$

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 due to the component P_p of the nail force parallel to the shear surface.

partial factors

$\gamma_{c'}$	partial factor for cohesion intercept, for typical values *c.f.* Chapter 3 and Annex B
$\gamma_{\phi'}$	partial factor for soil friction, for typical values *c.f.* Chapter 3 and Annex B
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
η	factor related to numb of pullout tests performed
γ_γ	partial factor for action, *c.f.* Chapter 3
γ_Q	partial factor for action, *c.f.* Chapter 3
γ_G	partial factor for action, *c.f.* Chapter 3
γ_{Sd}	model factor, *c.f.* Chapter 3

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). The pullout capacity is influenced not only of the soil but also by the type of nail and installation technique and an initial estimate may be done, but should be used with care in the design. The influence on the pullout capacity by different factors are further discussed in Annex A. Suggestion for how to make the initial estimate is also found in Chapter 2. The value used in the design should be equal to the minimum value of the capacity that may be mobilised in the active or resisting zone. It is important to confirm the assumed value by pullout tests during preliminary stage of the execution, see Chapter 9 for pullout tests.

The force mobilised in the nail also depends on the angle of installation. An angle of 10 - 20 ° downwards is commonly used since this will permit grout to flow into the hole with gravity pressure and at the same time ensure that tension is developed as quickly as the active wedge starts to move.

In some literature different definitions of the factor of safety are suggested where the nail forces are considered to decrease the disturbing forces. However this document only considers the definition in Equation (7.2), since numerical analysis has shown that it gives the most realistic value when the results should be compared to a global factor of safety for a slope without nails.

The shape of the failure plane depends on the type of soil, installation angle for the nail, load, time, number of nails, groundwater and the angle of the slope. Results from investigation performed by *Gässler et al.* (1983) indicate that in clay the failure surface tends to be circular and in frictional soil the bi-linear failure surface is more accurate. If the slope angle is small if related to a vertical line (*i.e.* steep) the failure surface tends to be bi-linear and a circular failure surface is more likely for a flat slope. Consequently, for a more or less vertical wall in frictional soil with constant nail length, it might be adequate to use a single wedge analysis instead of the circular failure surface. If this simplified approach is used a force-polygon may be used to determine the necessary restoring force in the nails. However, if a wedge-analysis is used it is recommended that different wedge angles are analysed.

In the stability analysis it is important to consider the effect of pore-pressure, since it will have severe influence on the stability of the slope.

The stability analysis is performed for a unit slice of the soil and from this the horizontal distance between the nails may be determined. The horizontal distance is also related to the facing. Greater distance between the nails requires a more rigid facing that may distribute the force between the nails. As a rule of thumb the maximum distance should be minimised to two meters. For greater distances the soil nail will have more resemblance to a ground anchor than a soil nail.

3. VERIFICATION OF THE CHOSEN SOIL NAILING SYSTEM

Next step in the design is to verify that the nail capacity is sufficient. The internal failure could appear due to;

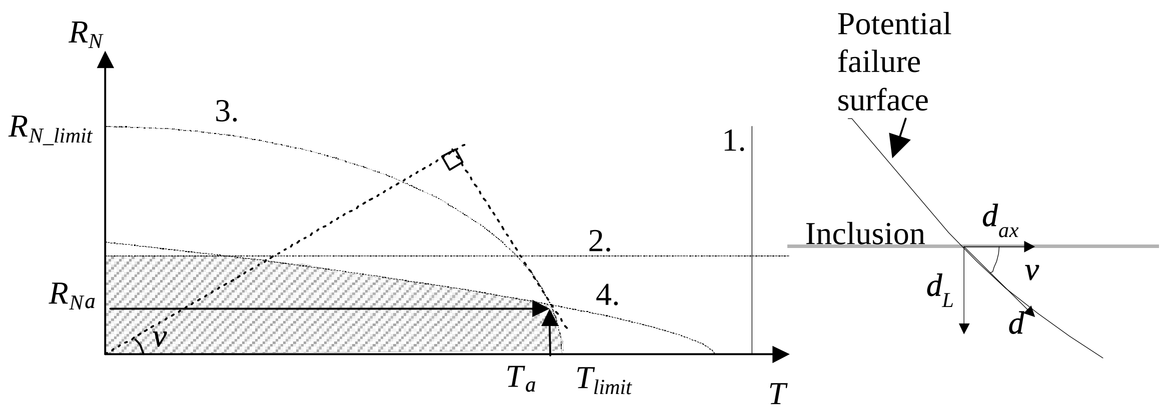
- Bearing failure in the soil because of movement of the nail
- Breakage of the nail due to tension
- Breakage of the nail due to shearing/bending in combination with tension

One possible way to analyse the internal failure of the nail is to use the French Multi-Criteria method (Clouterre, 1991).

This method is based on four failure criteria:

1. Pullout failure due to failure between the nail and the soil (tension)
2. Bearing failure in the soil below the nail
3. Failure of the steel in the nail due to tension
4. Failure of the steel in the nail due to bending/shearing

The four failure criteria are combined in a shear force vs. tension force graph, see Figure 7.3. The hatched area indicates the limiting yield envelope. To avoid failure the nail force should be inside the hatched area for all nails in the construction.



R_N - shear force, R_{N_limit}- limit shear force, T - tension force, T_{limit} - limit tension force	$R_{N\varpi}$- shear force nail angle ϖ T_{ϖ} - tension force for nail angle ϖ

Figure 7.3 Yield envelope according to the "Multi criteria method" (Clouterre - Schlosser et al., 1991)

4. EXTERNAL AND OVERALL STABILITY

After determining the appropriate nail layout to achieve stable gravity wall consisting of soil and nails, additional checks have to be made. The following external failure modes need to be considered.

- Sliding due to the active pressure from the soil behind the block acting on the reinforced block
- Bearing failure (the weight of the reinforced block and the lateral earth pressure acting on its back might cause a foundation bearing failure)
- Overturning of the reinforced block
- Overall failure (even though the nailed soil block itself is stable, an overall failure might still occur)

5. STABILITY ANALYSIS OF EACH EXCAVATION PHASE

After determining the final layout and the soil nailing system additional stability analysis is performed to verify that each excavation phase has sufficient stability. It might be necessary to have restriction on maximum height and length of each section that may be excavated.

7.1.3 Serviceability limit state design

The ultimate limit state approaches above only give a value of the total nail tension at Limit State. It does not give any advice on how the force is distributed between different nails and the actual movement of the slope. Consequently the movement of the slope has to be analysed with a different approach.

In urban areas it is of utter importance to analyse if the soil nailing may affect adjacent structures and installations. Is the sewer pipe installed 20 meters behind the wall influenced by the movement of the soil nailed wall? It should be remembered that the technique is based on that a small movement will occur to mobilise the force in the nails.

The method of construction of a soil nailed wall from the top and down, will lead to a greater movement of the soil in the top of the slope and consequently greater mobilised forces in the nails. As a consequence the mobilised force in the resisting zone may be greater in the top of the slope than in the bottom where the bond length is longer but the movement less.

The movement of the crust of the soil nailed wall depends on a number of factors. 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 such as the one found in Clouterre. For more sensitive structures a more thorough study of the deformation might be necessary which could be accomplished by application of Finite Element.

Table 7.4 gives an empirical estimate of the horizontal and vertical deformation of the facing for different types of soils. The final structure tends to tilt outward, due to the method of construction with greater movement at the top of the wall. To minimise the effect of this movement the wall may be tilted backwards a couple of degrees from the beginning.

The movement of the wall may lead to settlements behind the wall. The distance behind the facing that may be influenced can be estimated according to the following expression according to Clouterre,

$$l = H(1 - \tan y\ )k \qquad (7.3)$$

where
H is wall height,
ψ is initial inclination of the face relative to the vertical
κ is an empirical factor according to Table 7.4.

Table 7.4 Empirical estimate of deformation (Clouttere, 1991)

	Intermediate soils (rocks)	Sand	Clay
$\delta_v = \delta_h$	H/1000	2H/1000	4H/1000
κ	0.8	1.25	1.5

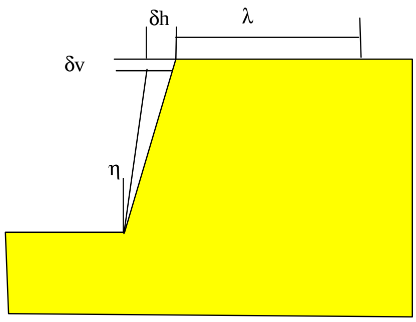


Figure 7.4 Deformations for a soil nailed wall

7.1.4 Drainage

Normally drainage should be incorporated in all soil nailing walls to avoid water pressure on the facing and to limit the detrimental effect that surface water and groundwater may have on the structure. The density and types of drains depend on the geometry of the wall, surface- and groundwater situation and type of soil. Commonly a minimum is to install weep holes though the facing (in case of shot-crete facing).

The drainage is designed so that its capacity is guaranteed throughout the entire design life of the structure.

SURFACE DRAINAGE is designed so that it have a sufficient capacity to control the water flow from the storm with the return period equivalent to the design life of the structure. The purpose of the surface drainage is to minimise the risk that surface water enters the soil behind the wall.

SUB SURFACE DRAINAGE should have a minimum internal diameter of 40 mm. The filter has to be compatible with the soilgrade curve. A thumb of rule is that the minimum density of the drains should be one for every 25 m^2, unless the design indicates a higher density. (Draft prEN 14490)

7.1.5 Facing

The purpose of the facing for a steep slope is to retain the material between the nails but also to distribute the force between different nails when the nail distance is increased.

For steep slopes hard facing is commonly used and the aspects that should be considered are;

- <u>The relation between facing thickness and nail distance</u>. Increasing the number of nails will result in less rigid facing (lower cost) but on the other hand the number of nails cost more. The design should aim for the most economical and technically best solution using an itterative procedure.
- <u>The connection between the nail and the facing</u>. The weak point might be the connection to the facing. If the nail head is assumed to take any force, it has to have a sufficient bearing capacity behind the bearing plate.
- <u>Durability</u> both for the facing itself and the connection between the nail and the facing.
- <u>Drainage.</u> To avoid water behind the facing that will give additional load on the facing.
- <u>Esthetical aspects</u> Adoption to the surroundings which is important especially in urban areas.

In the literature there are a number of different approaches for how the facing should be designed. Clouterre *e.g.* assumes a uniform pressure corresponding to the maximum tension that may be mobilised in the nail.

7.1.6 Durability

The requirements on corrosion protection system depend on the environment, type of nail and consequences of failure. In this section 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 specific 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 are evaluated and the final environmental class determined. In step 4 factors depending on the chosen soil nail system and consequences of failure are 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 U. Bergdahl (1986). Below is each step described in more detail.

7.1.6.1 Step 1 – preliminary estimate of the corrosion potential of the environment

The preliminary estimate is based on knowledge from traditional geotechnical investigations and geological maps. It is a system where the site gets a certain amount of points depending on a number of factors. According to Table 7.5 the site gets certain points depending on type of soil which is one of the major factors that influences the corrosion potential. In addition to these points the site will get additional plus or minus point depending on a number of factors according to
Based on the total amount of points the site is classified to have a low, medium or high potential for corrosion. If the total amount of points from this preliminary estimate is less than 5, no further information is needed to determine the necessary corrosion protection. Environmental class 1 is used in step 4 to determine the necessary corrosion protection.

Table 7.6.

Table 7.5 Classification of the environment corrosion potential depending on soil type

Corrosion potential	points	Type of soil
very high	10	Clay with salt content, organic soil (*e.g.* gyttja), fibrous peat, fill, industrial waste (cinders, ashes, coal)
high	6	Other clay and peat Construction waste (plasters, brick)
low	2	Silt, dry crust clay, moraine
very low	0	Rock, sand, gravel, sandy and gravely moraine

Based on the total amount of points the site is classified to have a low, medium or high potential for corrosion. If the total amount of points from this preliminary estimate is less than 5, no further information is needed to determine the necessary corrosion protection. Environmental class 1 is used in step 4 to determine the necessary corrosion protection.

Table 7.6 Classification of the environment corrosion potential additional points depending on a number of different factors

Factor	Additional points
The groundwater level is lower than 2.5 m below the ground surface	± 0
Groundwater[26] is periodically higher than 2.5 m below the ground surface	+3
Dry and well drained material	-2
Fill with both cohesion and frictional soil	+2
Organic clay (gyttja) or clay with sulphide	+3
Distance to road that is salted during the winter period is less than 25 m	+4
Meadows	± 0
Agriculture area where fertiliser is used	+2
Wood area - pine	+2
Wood area – mixed forest (spruce, leaf)	± 0
Wood area – deciduous forest (birch, alder)	-2
Waste water from industry, polluted soil	+2
Material that has been compacted and rearranged	+3
Varved soil where the soil nail crosses different types of soil	+2
Free ion from *e.g.* weathering (points depend on type of ion)	+(1 à 3)

7.1.6.2 Step 2 – Determination of environmental class based on more detailed soil investigation

It is not always enough to make a preliminary estimate of the environment but a more detailed investigation is sometimes necessary, including field investigation. Based on knowledge of the pH and resitvity of the soil, Table 7.7 can be used to determine a corrosion index, which gives the environmental class according to Table 7.8. The system is based on similar systems in Clouterre, 1991 and literature from AFNOR. The difference in soil between Sweden and France (the rock mass in Sweden is somewhat more acidic) is assumed to be taken into account by the pH-criteria.

[26] The groundwater level

Table 7.7 Corrosion index used to determine the environmental class

Criteria	Explanation	Index
Type of soil	Clay (impermeable, plastic)	2
	Clay, silt, moraine (normal)	1
	Sand, gravel, (porous, permeable)	0
	Gravelly /sandy moraine	0
	Peat	8
	Rock	0
Resistivety	p < 10 Ω m	5
	10 < p < 20 Ω m	3
	20 < p < 50 Ω m	2
	50 < p	0
Moisture - salt	Sample of soil with content of salt below the groundwater table (permanent or periodical)	8
	Sample of soil without content of salt below the groundwater table	4
	Moist sample of soil above groundwater table (w > 20 %)	2
	Dry sample of soil above groundwater table (w < 20 %)	0
pH	Very acid environment pH < 4	4
	Acid environment 4 < pH < 5	3
	Neutral environment 5< pH <6	2
	Basic environment pH > 6	0
Vertical layering	Soil profile with different layers	1
	Homogenous soil	0
	Rearranged soil - compacted	2
Other factors	Industrial waste; cinders, ashes, coal	8
	Construction waste; plasters, brick	4
	Waste water from industry	6
	Water with salt from road	8
	Σ	

Table 7.8 Determine the environmental class based on the sum of points from Table 7.7.

Environmental class	Explanation	Sum of points
I	Low potential for corrosion	0 - 4
II	Normal potential for corrosion	5 - 9
III	High potential for corrosion	10 -

7.1.6.3 Step 3 – Determination of environmental class considering other aspects

If one or more of the following statements are true, it might be advisable to choose a higher environmental class than the one obtained in step 2 *(e.g.* class III instead of II).

- The temperature is higher than normal
- Flowing water
- Stress level in the steel - high stress level or cyclic loads
- Contaminated soil – special investigation needs to be performed
- Leak current
- Chemical analysis of the soil and comparison with the values in Table 7.9 indicate that the soil has high aggressiveness or extremely high aggressiveness.

Table 7.9 Chemical analysis – limit values according to DIN 4030

	Degree of aggressiveness		
Parameter	Low	High	Extremely high
CO_2 (mg/l)	15 - 40	40 – 100	>100
Ammonium NH_4^+ (mg/l)	15 - 30	30 – 60	>60
Magnesium Mg^{2+} (mg/l)	300 – 1000	1000 – 3000	> 3000
Sulphate SO_4^{2-} (mg/l)	200 – 600	600 – 3000	> 3000

7.1.6.4 Step 4 – Choice of corrosion protection system

Two other parameters need to be considered in addition to the environmental class; type of soil nail and consequences of failure.

TYPE OF SOIL NAIL

All steel has a potential of corrosion and consequently it is always necessary to consider corrosion protection. No single parameter can be used to determine the potential of corrossion for a specific type of steel. Instead each steel quality needs to be tested for the proposed application. However, if any of these statements below are true, a higher requirement on the corrosion protection is needed. One way to account for this is to choose a higher environmental class than the one obtained in step 3, *(e.g.* class III instead of II).

- Steel with brittle failure is used
- Results from the FIP-test[27] give short time to failure
- The alloy content of the steel increases the corrosion potential of the steel or makes the steel failure more brittle

CONSEQUENCES OF FAILURE

A subjective opinion about the consequences of failure needs to be considered before the requirements of the corrosion protection are finally decided. In some cases it might be necessary with quite rigorous corrosion protection system in class I environment, since the consequenses of a failure are severe. In other cases the requirements could be quite low even in a class III environment since it is a temporary structure with minor consequences if a failure would occur.

CHOICE OF REQUIREMENTS FOR THE CORROSION PROTECTION

In Table 7.10 a suggestion for requirements for different design life and environmental class is given.

[27] Se FIP-report, Corrosion and corrosion protection of prestressed ground anchors, State of the art report, ISBN 0 7277 0265 3, 1986 for explanation of this test

Table 7.10 Suggested requirements for the corrosion protection depending on environmental class and design life

Environmental class	Design life Temporary	2-40 years	40-80 years	>80 years
I	no	low	normal	extremely high
II	no	normal	high	special investigation
III	low	high	extremely high	special investigation

No	no corrosion protection is necessary
Low	low degree of corrosion protection, *e.g.* 2 mm of sacrificial thickness or grout
Normal	normal degree of corrosion protection, *e.g.* 4 mm of sacrificial thickness or grout at least 20 mm thick combined with plastic barrier or sacrificial thickness.
High	high degree of corrosion protection, *e.g.* 8 mm of sacrificial thickness or grout at least 40 mm thick combined with plastic barrier or sacrificial thickness
Extremely high	plastic barrier is necessary

In Chapter 2.2.6 the different corrosion protection system is described further.

When determining the requirements for the corrosion protection of the soil nailed system it is important to not only look at the soil nail itself. Weak points such as the connection to the facing, couplings need to be considered, so that the whole construction has a sufficient protection.

7.2 NATURAL SLOPES

The main difference between a steep excavated soil nailing wall and a natural slope strengthening by soil nails is the loading of the nail. For the wall the nails are more or less loaded as soon as next excavation phase is performed. A soil nail installed in the natural slope will be non-tensioned as long as no further movement of the active wedge of the slope occurs. Additional load or changes in effective stress may result in movement of the soil, which in turn will mobilise friction along the nail. The mobilised friction in the resisting zone will prevent movement of the active wedge. The force induced by the movement of the unstable active wedge is redistributed through tension in the nails to the resisting zone. If instability increases and larger movements occur bending resistance might contribute with a small part to the resisting force. However, normally the movement, when the bending resistance is activated, is so large that the slope is more or less in Failure State.

7.2.1 Specific information needed for design

For a natural slope it is important to know the extent and form of the potential failure surface. An incorrect design of the soil nailing might actually destabilise the unstable slope further and consequently result in failure of the slope.

Geometry - It is important to determine the exact geometry of a natural slope before design of the soil nail system, both to be able to design a proper installation scheme (location of nails) and to determine how to access the slope for installation.

7.2.2 Ultimate limit state design

7.2.2.1 Failure modes

The failure modes for the slope reinforced with nails are similar to those for the soil nailed wall. The failure might occur, either as an internal failure of the nail or an external failure outside the reinforced block. For flatter slopes the resemblance with a gravity wall might be less obvious than for a vertical nailed slope. Nevertheless, a reinforced block is created due to the small movement of the active zone. This reinforced block will limit the movement of the soil behind it. Bearing capacity below the wall, tilting, sliding and overall stability are failure modes that might occur even for a nailed slope and should consequently be considered.

7.2.2.2 Design values and design loads

Design values for the soil and the soil nails are chosen in accordance with suggestions in Chapter 2 and 3.

7.2.2.3 Design step by step

The design of the natural slope is based on the same principal as the steep walL.
The following three steps are included in the design.

1. Stability analyses
2. Verification of the chosen soil nailing system
3. External failure modes and global stability

Theses will not be further discussed here since they previously have been discussed in Section 7.1.2.

For the preliminary estimate of nail layout the recommendations in Section 7.1.2.3 may be used but as a complement the critical failure surface of the unreinforced slope should be studied. The nail length behind the failure surface should be sufficiently long to make it reasonable to believe that the restoring moment due to the nails will be great enough to obtain a reasonable safety factor.

7.2.3 Serviceability limit state design

For the excavated slope it is evident that movement will occur as the excavation is performed from the top to the bottom. It might not be so obvious that the movement required to mobilise the nails in the natural slope is in the same order of magnitude. The outwards tilting might not occur but there will be a downward movement resulting in settlement. As for the excavated slope some movement has to occur to mobilise the nail force. Consideration to whether this will affect nearby houses or other structures has to be done.

7.2.4 Drainage

As for excavated soil nailed slopes it is important to control the water, since it might have severe consequences for the structure. As a general rule water should be avoided in a soil nailed structure. Surface drainage above the slope can be used to prevent runoff water to enter the construction. The facing system can be chosen so it minimises the effect of minor water flow. Additional drainage system could be installed to avoid the water in the structure. All drainage system installed should be robust and capable of maintenance during the design life of the structure.

7.2.5 Facing

There might be a number of different reasons for application of facing;
1. Redistributing the force between different nails
2. Work as a reaction frame so that tensile forces may be mobilised in the nail
3. Prevent local failure between the nails

Depending on the slope angle the suitability of different facings varies. For a steep slope a shot create facing might be the only alternative to achieve a local stability between the nails and work as a force redistributing beam. For slopes with a slope angle less than 30 ° it might not be necessary with any facing unless it is required due to erosion. For natural slopes it might be sufficient to use flexible facing such as geotextile.

The following factors should be considered:
- In case of a rather shallow slope and only vegetation is used as facing, it needs to be considered that it takes some time to establish the vegetation. Short term protection against erosion might be necessary
- In case of flexible facing it should be remembered that deformations are necessary to obtain forces in the geosynthetic. Consider if these deformations are acceptable for the facing (esthetical).
- In case of flexible facing the degradation of the geosynthetic needs to be considered. The long-term behaviour of the facing needs to be guaranteed.

7.2.6 Durability

The discussion about durability in Chapter 7.1.6 is valid for soil nailed slopes as well as soil nailed excavations.

7.3 EXECUTION, QUALITY CONTROL AND PROCUREMENT

These guidelines describe execution, quality control and procurement in the following chapters:
- Execution — Chapter 8
- Quality control: — Chapter 9
- Procurement: — Chapter 10

Based on the specific design it is recommended to ensure that any open question regarding project responsibilty, execution and quality control is clearly defined in the contract.

8 EXECUTION

This chapter 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). A paragraph that is indented marks text that is a quotation.

8.1 REINFORCED FILL

The construction procedure for a reinforced fill structure is comparable to any earthworks project, except that reinforced fill structures require additional considerations for supply, storage and installation of the prefabricated components. The construction method should not cause damage to the reinforcing materials, nor to the temporary or permanent supports and facing elements. Further it should not affect the free draining characteristics of the frictional fill material or of the drainage material placed in contact with the reinforced fill structure.

Construction of a reinforced fill structure may involve the following stages:

- Excavation, levelling
- Footings for facing elements or units
- Selection and haulage of materials:
    - fill material
    - drainage material
    - reinforcing elements
    - facing elements or units
- Erection of facing elements or units incl. placement of reinforcement and joint fillers
- Backfilling and compaction of fill and drainage material
- Construction of cladding or other surface structure on top of reinforced fill structure

8.1.1 Selection of Materials

8.1.1.1 Fill Material

Special attention has to be given to locating a source of suitable fill material. The following index parameters of the fill material shall be determined and checked against the demands given in the design for the work.

- Particle size distribution
- Liquid limit and plasticity index
- Moisture content
- pH-value
- Compaction values
- Shear strength
- Coefficient of friction between fill material and reinforcing elements

And if metallic reinforcing or facing elements are used

- Resistivity
- Redox potential
- Chloride-ion content
- Total sulphate content

- Sulphate ion content
- Total sulfide content

8.1.2 Reinforcing element material

8.1.2.1 Steel

Grade of steel elements and their quality are specified according to EN 10080 or EN 10025 (tensile strength, yield stress, strain at failure).

Galvanization is specified according to EN ISO 1461.

For woven steel wire meshes made of cold drawn steel EN 10218 and accordingly EN 10223/3 apply. Hot dip galvanised coatings on wires for woven meshes should comply with EN 10244 and EN 10245 for extruded organic coating.

Sacrificial steel thickness allowance has to be in accordance with the requirements of the design.

8.1.2.2 Geosynthetics

On site control of delivered reinforcing material may be done according to draft prEN 14475, which also allows a judgement for suitability of the material for the specific purpose.

8.1.3 Materials for Facings and Connections

All materials and products shall be in accordance with the relevant European standards. Where such are not available, the use of materials shall comply with national standards or guidelines and with local environmental regulations. The materials shall comply with the design specifications and appropriate test results shall be provided. Special instructions provided by the manufacturer concerning transport, handling, storage and placement of materials and products shall be observed.

8.1.4 Site Conditions and Site Investigations

8.1.4.1 General

Prior to the execution of the work all necessary information regarding the site conditions, any legal restrictions and conditions of adjacent infrastructure shall be given. This information shall cover access, the existing underground structures, any environmental restrictions etc.

8.1.4.2 Work Sites

Based on results from ground investigations of the work site the following information should be determined, the suitability of the site for a reinforced earth structure, the overall stability of the site for the execution of the work, the suitability of the material on the site for fill including both geotechnical and geochemical parameters.

- Index parameters: plasticity, soil classification, density, grain size distribution, water content, organic content
- Mechanical characteristics: shear strength, compressibility
- Hydraulic parameters: permeability
- Environmental data: contamination

Specific information is required concerning groundwater and surface water including the potential for inundation.

8.1.4.3 Sources of Fill Material

The geotechnical investigations of the borrow area shall provide the characteristic soil parameters specified in the design of the work. If geological mapping and visual inspection of the borrow area indicate high variation of the engineering properties of the fill material, ground investigations have to be adequately intensified.

8.1.5 Foundations

The reinforced fill block is generally founded on natural ground, excavated to a nominal level. The facing units are usually founded on a concrete strip footing, which in turn require additional excavation. The width of the strip footing has to allow for adjustment of the facing to the required alignment.

8.1.6 Drainage

Drainage is important in a reinforced fill structure, which must not be allowed to become water-logged. A possible increase of pore pressure and thus a reduction in shear strength may reduce its stability, and cause additional loads on the facings and increase tension in the reinforcing elements.

Where cohesive fill material is used a continuous drainage layer at least 0.3 m thick has to be placed at the rear of the facing and connected to the bottom drainage system. All reinforced fill structures shall be protected against infiltration of surface water.

On an impervious foundation layer provision shall be made for a horizontal drainage layer at foundation level. If necessary, suitable sealing and drainage measures should be taken on top and at the rear of the reinforced fill block to prevent aggressive substances from entering the reinforced structure.

Drainage layers in reinforced fill structures must be of sufficient thickness to cope with the anticipated water flow and in filter-relationship with the fill material.

The drainage material shall be spread in seperat layers along with the fill material and compacted, thus avoiding contamination of the drainage material. Their compaction may have to be done by using hand-held vibratory equipment.

A system of drainage pipes at the level of the strip footing shall be installed close to the rear of the facing. Provisions shall be taken to enable maintenance of the drainage pipe system, *e.g.* inspection manholes. Additional drainage holes may be required in the facing units above ground level may be required.

8.1.7 Facings

Facing units shall not be cracked or broken and have to be handled carefully using properly designed lifting devices connected to their upper edges.

Compaction of the fill or drainage material behind the facing units may change their alignment and require some realignment to meet the tolerances given in the design. Depending on the type of the facing unit special clamps or temporary support elements may be necessary to produce the desired finished face of the reinforced fill structure. *E.g.*

joint fillers may be necessary as a provisional measure to adjust the gaps between facing elements for possible differential settlements as well as for tolerances in the dimensions of the facing elements.

The joints of facing units shall be sealed, above the ground water level to prevent leaching out of the fill material. Joints below groundwater level should be left unsealed to allow for drainage and groundwater flow through the reinforced fill structure.

8.1.8 Selection, Placement and Construction of Fill Material

Reinforced fill structures have the advantage to allow a great variety of fill materials to be used. The surface on which the reinforcing material is placed must be compacted and levelled. All elements that might damage the reinforcement at any stage must be removed.

Correct placement and adequate compaction of the fill material are key issues to guarantee functioning of the structure as anticipated. The thickness placed or levelled must be such that it is possible to compact to the degree required by the design, *e.g.* 95 % of Modified Proctor density. Different compaction requirements or demands to follow a certain sequence in the compaction work for the areas close to facing or junctions have to be observed carefully.

The levelling requirements are unless otherwise specified, in the same order as for the individual layers of normal embankments, *e.g.* +/- 50 mm. A levelling thickness corresponding to the spacing of the reinforcement is preferable. Depending on the maximum stone size in the fill material the layer thickness has to be chosen such that optimum compaction efficiency can be achieved.

8.1.9 Installation of Reinforcing Elements and Connections

Due to the wide variety of possible processes only a few general rules are given. The reinforcing elements are placed on the compacted fill and connected to the facing units, or placed against a permanent or interim face (formwork support), folded back over the compacted layer and pretensioned. In all cases the instructions given in the design, guidelines of the material manufacturer or client have to be observed. Connections to the facings shall be in accordance with the demands and tolerances given in the design drawings. Special care shall be taken in checking the actual direction of the tensile elements, position and extent of possible overlaps and the right level of the reinforcement.

8.2 SOIL NAILING

The construction of a soil nailed excavated steep slope may include the following main processes.

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

The process is shown in Figure 8.1.

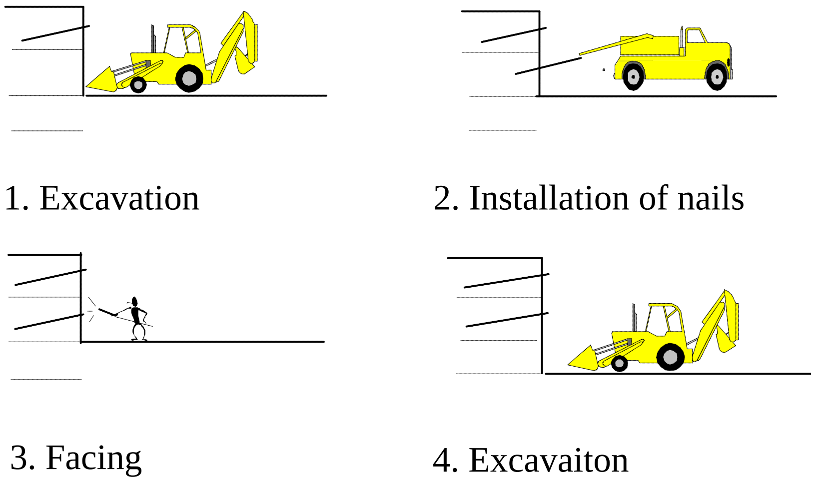


Figure 8.1 Construction phases of a soil nailed wall

In those cases when soil nailing is used to strengthen existing slopes, excavation and the facing may be excluded. In this section a brief description of each process is given in accordance with the draft of prEN 14490. For further details on the requirements related to execution the standard should be used.

Before the commencement of the execution of the work necessary information has to be provided to the contractor. This is further discussed in Chapter 10.

8.2.1 Preliminary work

The preliminary work may include all of or some of the following items;

- The position and geometri of the slope
- Access system for site where the access is limited (*e.g.* access road, excavated branches)
- Control of surface water and groundwater by installation of drainage. This is done to make it possible to execute the soil nailing work
- Pullout test on trial nails to confirm the assumed pullout strength of the nail
- Installation of monitoring system
- Trial pits to investigate the stand up time of the facing whitout support

The testing, control and monitoring is further described in Chapter 9.

8.2.2 Excavation and face preparation

Commonly the excavation comprises of an initial excavation (bulk excavation) followed by trimming of the face. Before commencement of the excavation the following should be agreed on;

- Who is responsible for the overall stability of the slope, and adjacent properties. It may be necessary with restrictions, *e.g.* such as excavation in section
- Excavation limits such as final slope, extension of the working area, temporary bench levels
- The tolerances of the excavation, such as how many degree divergences may be tolerated for the slope angle or the tolerance for the height of the temporary bench levels
- Monitoring system to ensure control of the tolerances

- What actions should be taken if the tolerances are exceeded
- How to handle unforeseen circumstances such as boulders and low face stability

Filling material is sometimes observed close to the surface and this material may be of poor quality and consequently have a short stand-up time. Special measures may be necessary to prevent their collapse.

The period of time between trimming of the face, installation of nail and facing construction should be limited. Trial pits during the preliminary work could achieve guidelines on maximum stand-up time. Based on this information the maximum length of the face that could be trimmed in advance is determined.

During the excavation the actual ground and water condition should be compared to the initial site investigation report. If differences are observed it should be reported according to the tender document. Based on the information changes in the design or action such as local back filling may be necessary.

8.2.3 Nail installation

The following should be agreed upon before the commencement of the execution of soil nailing

- Type of nail and installation method
- Tolerances for the nail layout according to Chapter 9

Two principal methods for installing soil nails exist; direct installation and drilled installation. The direct installation may be performed by percussive, vibratory or ballistic methods. Drilled installation methods with grouting may involve either gravity or pressure grouting.

The installation method should be chosen considering the specific site conditions. A different installation technique or relocation of the nail should be considered, if obstruction such as boulders prevents the installation of the nail to full length or with the correct alignment. Nails already fully or partially installed should not be removed.

During transportation and storage the nails should be handled with care to guarantee the quality of the nail. All nail installation should be carried out in a controlled manner so that the disturbance of the ground and the previously installed nail is limited. Before installation the nails should be controlled so that they are in a condition fulfilling the requirements in the design.

The control, which should be performed during the installation, is described in Chapter 9.

8.2.3.1 Direct installation methods

The direct installation methods install a driven nail utilising percussive, vibratory or ballistic methods. For direct installed nails the reinforcing element usually is in direct contact with the soil, without any grout. In those cases that the nail is not manufactured in one piece it is important that the joint is performed in a way that it does not influence the load transfer mechanism.

The nail needs to have sufficient stiffness to be driven into the soil to avoid buckling. The required stiffness needs to be determined considering the ground condition (degree of compaction) and the installation-driving tool.

During the installation the driving time and the depth should be recorded. The driving time gives an indication if layers of stiffer or weaker ground are found along the nail.

8.2.3.2 Drilled installation methods

Drilled installation methods include mainly two different installation techniques;

1. Regular nail - A hole is drilled and the reinforcing element is installed in the centre of the hole. The hole is then filled with grout from the bottom of the hole to the top. The reinforcing element may be installed after the hole has been filled with grout.
2. Simultaneously drilled and grouted - The nail itself is used as drill. During drilling the hole is simultaneously grouted. After installation the drillbit is left in the hole as the reinforcing element.

The drilling technique should be chosen so that the following is ensured;

- The nominal nail diameter is achieved along the entire length of the nail.
- The borehole should be drilled to a depth to ensure that the nail will have the required design length. It may be necessary to use over-boring with some drilling techniques.

To ensure that a minimum required cover of grout is achieved along the entire length of the nail spacers should be evenly spaced along the nail. It is recommended that the soil nail be grouted as soon as possible after it has been drilled, at least the same day.

There are mainly three different types of drilling according to draft prEN 14490;

- Open hole drilling
  This type of drilling with <u>augers</u> may be used in self-supporting soils. Excessive removal of soil during the drilling due to *e.g.* collapsing soil stratum should be avoided. If there is a risk of borehole collapse the use of hollow stem augers may be useful to allow for installation of the reinforcing element and grouting before withdrawal of the auger.
  <u>Flushing techniques</u> may be used in all types of soil as long as a suitable flushing material considering the soil material is used. In self-supporting soil, air may be used and denser fluid (*e.g.* grout) in less stable holes.
- Cased hole drilling
  This method is used in soil that will not stand open along it entire length until it has been grouted. In soil that is not self-supporting the grouting should be done before the removal of the casing.
- Drilling with reinforcing element
  A drillbit is applied to the reinforcing element. Grout is commonly used as flushing medium. In some cases the simultaneous drilling and grouting will result in an enlarged grouted body, consequently the rate of drilling, grout pressure and flow rate should be adjusted to suite the soil conditions.

The second part of the drilling installation techniques is the grouting, which could be performed either as gravity grouting or pressure grouting.

a) Gravity grouting
   A tremie tube is advanced to the bottom of the borehole and grouting is performed without interruption from the bottom of the borehole to the tube until a non-diluted,

non-contaminated mix emerges from the top of the borehole. The withdrawal rate of the tremie should guarantee that the end of the tremie pipe is below the grout surface during the entire procedure.

b) Pressure grouting
A grout pipe is connected to the reinforcing element and grouting is performed during driving or after completion of driving.
The system with rotary drilling and simultaneously flushing with grout is sometimes described as dynamic pressure grouting.

The grouting mixture that is used should be used immediately after mixing and the entire batch should fulfil the requirement in the design.

The appropriate grouting technique should be chosen to ensure that no features such as air voids, that could reduce the capacity and durability of the nail, is introduced. During the grouting volume of grout and grouting pressure for each nail should be recorded.

If hollow stem auger methods are used the auger rotation should not be reversed during the extraction since this may cause soil to mix with the grout and reduce grout strength.

8.2.4 Drainage installation

An effective system for drainage is an important part of the soil nailing construction. Surface water and groundwater may have detrimental influence on the wall. It is therefore important with drainage both during construction and design life. If unexpected groundwater conditions are observed during the execution it may be necessary to upgrade the drainage system.

Surface water may be controlled by *e.g.* cut-off trenches or channels. These should normally be installed before the execution of the soil nailing starts. Internal drainage or drainage blanket immediately behind the facing may control groundwater. Water from the drains should be collected at one point and discharged in accordance with the environmental regulations.

There are mainly three different types of drainage;
1. Surface drainage (*e.g.* sheeting, channel, trench)
2. Facing drainage (*e.g.* geotextile filter, weep holes)
3. Sub surface drainage (*e.g.* drainage pipe)

In Chapter 2 the three different types of drainage are shown.

On the surface above the soil nailed face, sheeting maybe applied to control the surface water. Special attention should be paid to the overlapping to the sheet to prevent water from entering between sheet and ground. It may be necessary to pin the sheeting to the ground to avoid that it will lift due to wind forces.

Drainage channels commonly constructed in concrete could be used to collect the surface water. If used it is important that they are constructed so that they have a continuos fall to a collection point and that all joints are watertight. The channel should be constructed with expansion joints to allow for differential settlement and thermal movement. The construction should also ensure that there is no ponding and prevent water to pass into the soil below the channel.

A trenched drain is another possibility for collecting the surface water. The excavation of the trench should be performed in a controlled manner, minimising the time the trench is left open. Before back filling the trench it maybe lined with a geotextile to prevent fines from clogging the drain in the long term. The drain (perforated pipe or well screen) should be inspected for damage, so it can be ensured that it will work as a continuoes drain. The drain should fall continuously to the collection point.

To control the water behind the facing geotextile drainage filters may be applied. Commonly the geotextile drainage filter is placed vertically at specified intervals. In some cases it may be suitable with additional horizontal strips at each shotcrete joint or in areas with much water. To avoid that the quality of the concrete facing is affected, it is recommended that not more than 15 percent of the facing area be covered with filter. The drain must be continuoes from the top of the wall to the bottom, and connected with sufficient overlap to ensure continuity of the hydraulic flow. The filter must be securely fixed to the ground to avoid voids behind the facing. Necessary actions should be taken to prevent damage to the filter during subsequent excavation and facing phases.

If shotcrete facing or other low permeability facing is applied weep holes should be used. The weep holes with a minimum diameter of 25 mm allow the free flow of water from the back of the facing. If it is possible the facing drainage system should be tested prior to application of facing.

In some cases it may be necessary to use sub-surface drainage and if applied the drainage should have a minimal fall of 5 percent towards the facing. A method of installation should be chosen that ensures that the pipe is not damaged and that soil is not smeared over the filter. The connection between the drain and the facing should be performed in such a way that the water passes through the drain and not erodes the soil around the connection point.

De-watering systems are not normally used since soil nailing usually is performed above the groundwater level.

8.2.5 Facing installation

The facing should be installed as soon as possible after excavation and installation of the nail to avoid local failure of the surface. In some cases it might be necessary to perform the installation of the nails through a protective berm in front of the slope/wall. Another possibility is to apply a thin layer of shotcrete on the slope directly after excavation.

The maximum height of each excavation stage should be determined based on calculation and experience from projects with similar soil conditions. It can be necessary to perform the excavation in slots to avoid failure of the slope before installation of the nails. Between two excavation stages a minimum time of 24 hours is recommended, to allow the grout to obtain certain strength.

It is important to consider the drainage of the slope during the execution, an unexpected waterbearing soil layer may have severe effect on the surface stability. If geosynthetics are used as facing for a natural slope it is important that the geosynthetic is firmly attached to the slope so that it will mobilise force at a small movement.

9 QUALITY CONTROL - SUPERVISION, TESTING, MONITORING

This chapter 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).

Problems relating to movements in the structure during construction are most common for reinforced fill structures, especially with facings composed of large single panels. When a vertical reinforced fill structure is built up in height, the lower layers of soil will consolidate, causing settlements and outward movements. It is therefore most essential to ensure the qua-lity of the work and of its material components. Supervision, monitoring and testing of such structures is undertaken by qualified and experienced specialists in compliance with the design and the contract documents, *c.f.* Chapter 10. The contract documents should define the level and amount of monitoring and testing to be performed and also define type and accuracy of monitoring required.

Problems relating to movements of reinforced fill structures incorporating wrapped around or semi-rigid facings during construction are usually not so important. Performance of the work is comparable to normal earthwork conditions. Supervision, testing and monitoring of this type of reinforced fill structures have some special features related to the quality of the rein-forcing materials, their proper storage, protection against damage, pre-tensioning and *e.g.* maintenance of vegetation.

For a Soil Nailed structure it is always important that the supervision, testing and monitoring should be performed to ensure the quality of the work and comply with the requirements in the tender document, *c.f* Chapter 10.

9.1 SUPERVISION

9.1.1 Reinforced fill

Supervision should be performed in accordance with the tender document. Supervision includes records, testing and monitoring and is required during all stages of work: storage and handling of facing units and reinforcing elements, excavation of foundations, compaction of fill material and of the drainage material. For reinforced fill structures incorporating steel and/or geosynthetic reinforcing materials supervision should be performed according to prEN 14475 (Chapter. 9.1).

The execution of the work needs to be supervised at the different construction stages in respect to the following details:

- site preparation (on site soil conditions for preparation of foundations, groundwater level, drainage conditions)
- Fill material (moisture content, density, grain size distribution, shear parameters, organic content)
- Reinforcing elements (identification, weight/ unit area, cross area, width, thickness, spacing of transverse members). For geosynthetic reinforcing material a standard document on quality control on site is in preparation and will be issued as a EN standard or CEN- Technical report (ref.: CEN TC 189 work item 70)

- Facing (storage conditions, overall dimensions, cracks or faults, connection devices)
- Drainage (pipes, filter requirements, gradation, organic content)

9.1.2 Soil Nailing

Supervision should be performed in accordance with the tender document. Supervision includes records, testing and monitoring during the construction.

A nail installation plan should be available at the site and contain the following information according to the draft of prEN14490

- Nail type
- Number of nails
- Location and orientation of each nail and tolerance in position to an agreed datum.
- Required load carrying capacity of the nail (pullout capacity)
- Installation technique
- Known obstructions and any other constraints on nail activities
- Date and time of installation of each nail
- Method of corrosion protection
- Nail testing undertaken.

As a complement to the installation plan each nail installation should be recorded, and the following information may be included;

- Nail type
- Installation date and time
- Nail type, diameter, length, and orientation.
- Drilling method
- Bore hole cased or not cased
- Flush method
- Underground condition (short description)
- Water condition
- Consumption of grout
- Remarks
- Special measures

The record is kept toghether with the other construction records.

9.2 TESTING

9.2.1 Reinforced fill

Testing of the material components for reinforced fill structures should be performed in accordance with ENV 1977 and the specification of the design. The QA-inspector should keep regular records covering information required in the project's monitoring plan. This usually includes both test reports and certifications supplied by the manufacturers of the reinforcing elements and reports on tests performed on site. The report gives test methods with corresponding test results, and conclusions on compliance with specified requirements.

9.2.1.1 Tests for fill material

Testing of the fill material should be performed for the materials used on the site in the form of:

- Field trials for checking dry density and compaction moisture content
- Laboratory tests or field tests for shear strength of the soil

The following tests may also be performed

- Laboratory tests to determine fill-reinforcement friction
- Laboratory tests on electro-chemical properties of the fill

9.2.1.2 Reinforcing elements

For the reinforcing elements field trials could be performed with the actual fill material and the compaction equipment used at the site to check the susceptibility of the reinforcement against installation damage, if there is limited knowledge about the material. For the reinforcing material delivered to the site the supplier should ensure that the materials fulfil the requirements of the design. Test methods as outlined in prEN 14475 and in Annex A should be applied.

9.2.2 Soil Nailing

9.2.2.1 Pullout test

The pullout capacity should always be verified by pullout tests at the site. Below is a short description of different tests, test equipment, test performances, suggested number of tests and interpretation of tests. This description is based on the draft of prEN 14490.

The tests might be performed either on a sacrificial nail, *i.e.* a nail that is loaded to failure and consequently it can not be included as a working nail in the final structure. A production nail may also be used. This nail is loaded to its design strength and will continue to be a working nail in the structure after the test.

DIFFERENT TYPES OF TESTS

Depending on the purpose of the test there exist different types of principal soil nail load tests.

- A design investigation test is performed during the design of the soil nailing at the actual location of the structure, to obtain a value of the pullout capacity for design. Sacrificial nails are used and the obtained value of the pullout capacity should be used with caution if a different nail or installation technique is used.
- A suitability test is performed either before the construction or during the initial construction stage to verify that assumed pullout capacity in the design is obtained in the field. If the required pullout capacity is not obtained the design might need to be changed. This test is commonly performed on a sacrificial nail.
- An acceptance test is performed to verify an acceptable load-deformation behavior of the nail at the working load. These tests are performed during the production. In this case it is important not to overstress the nail so that the bond between the nail and soil or the corrosion protection system is damaged.

TEST EQUIPMENT

The testing equipment consists of the following main items;

- Stressing device

The stressing device should be designed so that the load could be applied axially to the test nail. It is also preferable if the length of the stroke of the hydraulic jack is enough to avoid resetting during the test.

- Load measurement
  There are two different ways of measuring the load either indirectly by monitoring the hydraulic pressure in the stressing device or directly by a load cell. The measuring device should be calibrated to an accuracy of 1-2 percent of the maximum test load.
- Reaction system
  It is important to construct a reaction system that is stiff enough to provide a support to the maximum test load and at the same time makes sure that it does not influence on the measured pullout capacity.
- Displacement measurement
  Dial gauges with an accuracy of at least 0.1mm should be used with a accurracy of 0.02 mm. At least two dial gauges should be used to provide an average reading if the set-up is not perfectly centric. It is important that they are separated from the stressing device and attached to a free-standing frame that is rigid enough to ensure that it does not move due to other effects such as vibration, climatic conditions and so on.

TEST PERFORMANCE

For a design investigation and suitability test of the nail the following steps are included;

- Apply a small load (not exceeding 10 percent of the anticipated failure load of the nail) to align the test equipment.
- Apply the load in increments. Aim for at least 10 steps before failure and consequently apply one tenth of the anticipated failure load in each increment. For each step the displacement should be recorded at 1, 2, 4, 8 and 16 minutes. If the movement of the nail between 8 and 16 minutes is greater than 0.1 mm additional measurement at 32 minutes should be performed.
- The stepwise increase of the load is continued until failure.

For acceptance test the nail is only loaded to a load corresponding to the design load times a proof factor. The factor may be taken equal to the partial factor applied during the design. Since the only purpose of this test is to verify that the nail performance at the working load is acceptable, the number of increments may be reduced compared to test performed on sacrificial nail.

To ensure that the load transfer is only occurring in the bonded length of the nail a critrion for a minimum displacement has been established. The displacement of the nail head should be larger then the theoretical elongation of the unbounded length of the nail.

$$\mathrm{H}L \geq 0.8\, P\, \frac{UL\, 10^6}{AE}$$

P	is maximum applied load (kN)
UL	is unbounded length (m)
A	is cross-sectional area of the steel (m^2)
E	is young modulus of steel (200 GPa)

SUGGESTION OF NUMBER OF TESTS

Table 9.1 is taken from the draft of the European Execution Standard for Soil Nailing prEn14490 and gives a suggestion for the number of load test, which should be performed.

The number of tests depends on whether the soil nailed structure could be classified as a category 1, 2 or 3 structure.

- Category 1 structure – negligible risk to property or life
- Category 2 structure – no abnormal risk to property or life
- Category 3 structure – all other structures not included in category 1 or 2

Table 9.1 Suggested minimum frequency of load test according to draft of European Execution standard for soil nailing

	Suggested minimum Frequency of Load tests			
Test type	Investigation	Suitability	Acceptance	
			N[28]> 1 per 1.5 m^2	N< 1 per 1.5 m^2
Category 1	Optional	Optional	Optional	Optional
Category 2	Optional	S2	A2a	A2b
Category 3	Optional	S3	A3a	A3b

Below the abbreviations in Table 9.1 are explained;

S2	• If no experience of the specific soil type then at least 1 test nail per soil type (layer) should be performed and the total number of test nails should be at least 3. • If there is experience of the soil type at the site (tests or soil nailed structures have been performed in similar conditions) the suitability tests are optional.
S3	• As for S2 but the number of test nails should be a minimum of 2 test nails per soil type and the total minimum of test nails should be 6.
A2a	• If the slope that should be nailed covers an area less than 1000 m^2 the number of tests should be 5. • If the slope is greater then 1000 m^2 than at least 1 test per 400 m^2 slope should be performed.
A2b	• If the number of nails is less then 200 than 3 tests should be performed • If the number of nails is greater than 200 then 1.5% of the nails should be tested.
A3a	• If the slope that should be nailed covers an area less then 1000 m^2 the number of tests should be 5. • If the slope is greater than 1000 m^2 then at least 1 test per 200 m^2 slope should be performed.
A3b	• If the number of nails is less than 200 then 5 tests should be performed • If the number of nails is greater then 200 than 2.5% of the nails should be tested.
• For all acceptance tests the following criterion should also be fulfilled. At least one test per type of soil and excavation stage. • It is also important to distribute the test nails evenly throughout the structure.	

INTERPRETATION OF TEST RESULT

The results from the tests are plotted in a force vs. displacement graph. In those cases where a distinct peak value is obtained it is quite easy to evaluate the maximum pullout capacity. If no distinct peak is obtained another failure criterion has to be used. One suggestion is to use the force the movement continues without application of any more loads.

A distinct peak value indicates a brittle failure of the nail. However it is preferable to obtain a ductile failure and consequently it might be better to use the residual value in the

[28] Number of nails per m^2 of slope

design, *c.f.* discussion about which angle of internal friction that should be used in Section 2.2.3.

9.2.2.2 Material test

For material delivered to the site the supplier should ensure that the material fulfils the requirements in the design. Grout and sprayed concrete is mixed at the site and consequently it is necessary to perform tests to ensure that the material fulfil the requirements. Grout should be sampled and tested to make sure it has the desired characteristic strength. For the sprayed concrete tests should be made on both materials from preliminary test panels and the completed work.

9.2.2.3 Face stability

In some cases it may be recommendable to perform face stability tests to ensure the stability of the excavation of the soil nailed structure. This test is performed by excavating a trial pit to a batter and depth equal to the slope angle and bench height in the design. The width of the excavation should be at least twice the height and the time of observation should be equal to the anticipated time between the installations of two successive rows of soil nails.

If differences in soil variation is encountered during the execution of the soil nailing construction, that were not forseen in the original design, it might be recommendable to perform additional face stability tests during the execution.

9.2.2.4 Durability

The durability of the soil nail should be verified by ensuring that the characteristics of the soil nail in the design are fulfilled. The grout quality, grout cover over the entire length of the nail, quality of any other applied protection system, steel quality should be ensured. Test nails could be installed at the site, if a soil nailed structure is executed in a particularly severe environment where the failure of the soil nailed structure would result in major consequences. A test nail in this case is an identical nails to the one installed as a production nail but shorter (1-1.5 meters). If the production nail is grouted the test nail is installed without grout to simulate the influence of cracks in the grout. The test nails are excavated at regular intervals and the following tests performed;

- Visual examination
- Determine the comparative weight of the nail
- Mechanical tension tests.

Further information about the use of test nails for verification of the long time behaviour of the soil nail can be found in (Clouterre – Schlosser *et al.*, 1991).

9.3 MONITORING

9.3.1 General - Reinforced Fill and Soil Nailing

This general part is mainly based on information from FHWA’s field inspector’s manual for Soil Nailing, but is applicable for reinforced fill as well.

The purpose of the monitoring during the construction is to verify that the execution is performed according to the design and that the assumption made during the design is relevant. If differences are observed the result from the monitoring enables modification of the design to ensure a structure with high quality.

Below is a short summary of monitoring for different parts of the construction that could be relevant. Additional monitoring may be necessary dependant on the size and location of the site. Before the commencement of the execution the following should be agreed on:

- The responsibility for the monitoring. This person is referred to as QA-inspector in the following text. Additional information can be found in FHWA – field inspector's manual
- The frequency of the inspection
- Predicted threshold values and what measure to be taken if the threshold is exceeded

9.3.1.1 Excavation and site preparation

During the preliminary work the QA-inspector should check for any variances between the actual ground surface elevation and the one shown on the plans.

During the excavation the QA-inspector should control;

- That the construction is performed according to the design
- The tolerances are not exceeded
- Any tendency to stability problems should be noted at the daily inspection of the slope and adjacent area.
- After each excavation step, control that no over-excavation is performed.
- Ensure that sufficient time is allowed between successive excavation phases to ensure that grout has time to cure and achieve necessary grout strength (for Soil Nailing).
- Visual inspection of the excavated materials to verify that they comply with the assumed ground conditions for the design (for Soil Nailing).
- Identification of soil types, layers, fractured zones, seepage, sources of water and verification that they are in accordance with the design assumptions.

If unforeseen circumstances such as changes in ground or hydraulic conditions are encounter during the execution this should immediately be reported to the appropriate person according to the tender document.

9.3.1.2 Reinforcement or Soil nail installation

Before installation of the reinfrocement/nail the following should be controlled;

- That the reinforcing element and the grout fulfil the requirement of the design.
- The delivered steel elements have the required sacrificial thickness and correct steel quality.
- That the corrosion protection system has not obtained any damage during transportation or storage.

During the installation the following should be controlled;

- The nail is installed according to the tolerances specified in the design
- The connection to the face is in accordance with the design and is securely fastened.
- Ground conditions, if differences with the anticipated ground conditions are observed necessary action should be taken.

And for nails:

- For drilled nails it should be verified that the hole is drilled within acceptable tolerances of the specified alignment, length and diameter. It may also be relevant for some soil conditions to inspect the holes for caving.
- For grouted nails it should be verified that centralisers are installed with specified intervals and that they do not prevent the flow of the grout. Minimum grout cover

should be obtained. Verify that the reinforcing element is placed in the centre of the hole.
- Verify that the nails are installed to required length and with correct alignment. An allowable inclination tolerance of ± 3° is common.
- For longer nails couplers are commonly used to join sections of reinforcing elements. In this case it is important to control equal thread penetration into the coupler. To avoid uncoupling it is equally important to ensure that the thread is locked.

9.3.2 Reinforced fill

9.3.2.1 Facing

During the construction:
- Check that the material properties of the facing are according to the design
- Confirm that type and geometrical dimensions of the facing are correct

9.3.2.2 Drainage

- If facing drainage and weep holes are used it should be verified that these have been installed as specified and provide continuous drainage path
- If a geosynthetic sheet drainage is used at the rare of the facings it should, during construction, be regularly inspected against damage and repaired to maintain its serviceability

9.3.3 Soil Nailing

This chapter is mainly based on FHWA's field inspector's manual for Soil Nailing.

9.3.3.1 Grouting

During the grouting the QA-inspector should control the following;
- For gravity grouted nails verify that grouting is starting at the bottom of the hole and the tremie pipe always remains below the level of grout as it is extracted.
- Measure and record the volume of grout placed in the hole. Calculate the grout take as the actual volume of grout placed in the hole divided by the estimated hole volume.
- Verify that the auger is not reversed during withdrawal.
- The grout should be mixed according to the approved mix design.
- Confirm that the grout has the required strength. Tests on grout cubes.
- Verify the bonded and unbounded length of the test nails.

9.3.3.2 Drainage

- If sheeting drainage is used it should be regularly inspected and repaired to maintain its serviceability.
- If facing drainage and weep holes is used it should be verified that it has been installed in as specified and provide continuos drainage path.

9.3.3.3 Facing

For the facing it is important to verify during the execution that the facing is performed according to the design.
- Check that the material properties of the facing are according to the design
- Confirm that the layout of the facing is correct *e.g.*
    - number of welded wire mesh or geonet is correct and that they have the prescribed overlap,
    - control the thickness of the sprayed concrete,

  - the final grading of slope is according to the design
- For excavated slopes it is also important to make sure that the facing (usually shot-create) is applied in the specified time limit to avoid failure of the excavation step.

9.3.3.4 Corrosion protection system

During the installation of the soil nails it should be verified that the installation procedure do not damage the corrosion protection system; *e.g.*

- Control that the delivered steel core has the required sacrificial thickness and correct steel quality
- Control that centralisers are installed with the required distance along the nail to guarantee the prescribed grout cover
- Control that there is no damage (cracks) in the protective cover of the nail.

10 PROCUREMENT

10.1 THE OBJECTIVE OF THIS CHAPTER

There are, in principle, two distinct types of contract for civil engineering and building work, “construction contract” and “design and construct contract”, in Swedish (*utförande entreprenad, generalentreprenad*) and (*totalentreprenad*) respectively. However, these types are seldom used in their pure form; in reality much contract work contains a mixture of both in one or more parts. In the following sections two different types of contracts are described.

The objective of this chapter is to 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 are considered “design and construct” and “construction” contract. However, a number of other models of contract exist, such as “perform –operate – transfer”, etc, but they are not discussed here.

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 suggestions in this chapter should only be considered as suggestions. However, it is recommended that the responsibility of different activities be agreed upon before commencement of work.

10.1.1 Construction contract

The employer (beställaren) provides the design documents to the contractor (entreprenören) and the contractor executes the works in accordance with the employer’s design. The design concept also includes the functioning of the project, which subsequently falls under the employer’s responsibility.

The employer may contract the various trades companies (earthworks, building, installations etc) and manage the whole of the works himself, or contract a main contractor – general contractor – for carrying out the works under a single contract (”general contract”, Swedish ”generalentreprenad”). Trades that do not fall within the main contractor's area of business would be sub-contracted by him.

10.1.2 Design and construct contract

The employer transfers the design work upon the contractor, who will undertake the full responsibility for the whole of the execution of the project work, including its proper functioning.

The employer defines the principal layout, the function and other requirements in a programme, which will be the basis for the tendering and the contract with the successful contractor.

The contractor usually assigns the design part to a consulting company, which carries out the design calculations and prepares the drawings, specification etc, for the execution under a sub-consultancy contract. The contractor also sub-contracts works, if any do not fall under his field of business.

10.2 CONTENTS OF THE TENDER DOCUMENT

10.2.1 Information which needs to be considered

The tender documents will consequently depend on the type of contract applied. However, 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 is 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 Table 10.1 to Table 10.5.

The party who provides the information is responsible for its correctness.

The guideline presents the information as complete as possible, considering that the main purpose of the contents is to encompass information required.

The actual type of contract or mixture of contracts should be taken into consideration when reading the tables and therefore as said before the suggested responsibility should be considered as indicative only.

10.2.2 Tender document for Construction contract

10.2.2.1 Information provided by the Employer

In the case of a construction contract it is the employer’s responsibility to provide the contractor with documents with sufficient information to build the structure. This is commonly done in the tender document. Example of the information that is important to include in the documents, either in specifications and/or drawings, is suggested in Table 10.1 to Table 10.5. The structure should be shown both in plans, sections and details.

10.2.2.2 Information provided by the Contractor

The contractor’s responsibility is to make a programme for the execution based on the information from the client. The execution could not take place untill for employer has approved this programme.

10.2.3 Tender document for Design and construct contract

10.2.3.1 Information provided by the Employer

As mentioned previously the responsibility of the employer for this type of contract is to provide the contractor with a list of requirements. This list is essentially covered in Table 10.1.

10.2.3.2 Information provided by the Contractor

The responsibility of the contractor is to suggest a design and construction programme and provide documentation of the design to the employer, to prove that the structure will fulfil the requirements.

The programme of design includes the following information:
- Assumptions and the main calculation steps
- The failure modes and the earth pressure for reinforced walls
- The critical failure surface and the corresponding factor of safety for reinforced and unreinforced slope for soil nailing
- The critical failure surface and the corresponding factor of safety for reinforced and unreinforced embankment for embankments on weak ground
- The stability and deformation for embankments on improved soil
- List of computer codes used for design including version number
- If the design is performed as a hand calculation this should be shown in the documentation

The programme includes the information relevant to a design and construct contract according to Table 10.1 to Table 10.4. The construction could not take place untill the Employer has approved the programme.

The contractor also establishes a programme for execution. This document includes the information according to Table 10.5 and is to be approved by the employer before the commencement of the work..

To verify that the structure is performing according to the requirements by the employer the contractor establishes a programme for the control according to Table 10.5. The programme of control should also include the extent of records/documentation during the execution and is to be approved by the employer.

Table 10.1 General information (ex.)

	TYPE OF CONTRACT			
	C[29]		D AND C[30]	
GENERAL INFORMATION	Employer	Contractor	Employer	Contractor
REQUIREMENTS:				
On the final function of the structure	(x)[31]		x[32]	
Level of safety	(x)		x	
On the geometry of the structure	x		x	
Esthetical aspects of the construction	x		x	
Permissible deformations of the structure during service life	x		x	
Loading (permanent, temporary and dynamic)	(x)		x	
Service life (temporary, permanent)	(x)		x	
Requirements on measurement equipment, type and quantity	x		x	
Regulations and standards applied	x		x	
Type of structure if there are any requirements or wishes from the employer	x		x	
RESTRICTIONS:				
Restriction of available time for construction or other limiting factors such as for railways and roads on-going traffic during construction	x		x	
Restriction on the construction method due to environmental consideration, noise, vibration and pollution or other aspects	x		x	
Restriction due to tidal working or cold climate	x		x	
Restriction due to archaeological constraints	x		x	
Any legal restriction (*e.g.* lowering of groundwater not allowed)	x		x	
GENERAL INFORMATION ABOUT THE SITE:				
A Ground Investigation Report including the groundwater conditions	x		x	
Co-ordinate points for setting out	x		x	
Information about adjacent structures and roads; this includes underground structures such as services	x		x	

[29] Construction contract
[30] Design and construct contract
[31] (x) the employer has this information and may provide the contractor with it.
[32] x party marked with x should provide the opposite party with information

Table 10.2 Information about the structure (ex.)

	TYPE OF CONTRACT			
	C[33]		D AND C[34]	
INFORMATION ABOUT THE STRUCTURE	Employer	Contractor	Employer	Contractor
DESIGN PARAMETERS OF FILLING AND GROUND (*e.g.* friction angle, cohesion, density, water)	(x)[35]			x[36]
THE STRUCTURES GEOMETRY AND LOAD	(x)			x
Sections of the structures geometry including the soil profile and ground water level	x			x
Assumed load and load distribution of external loads and earth pressure	(x)			x
Design model both for the reinforcement and the facing if applicable	(x)			x
Definition of the factor of safety and chosen level of safety	(x)			x
THE LAYOUT OF THE STRUCTURE				
The reinforcement extension in the longitudinal and transverse direction of the structure including tolerances	x			x
For soil nails , their position, angle of installation, length and tolerances	x			x

[33] Construction contract
[34] Design and construct contract
[35] (x) the employer has this information and may provide the contractor with it.
[36] x party marked with x should provide the opposite party with information

Table 10.3 Information about material properties for Reinforced Fill (ex)

Information about properties of the materials	Type of contract: C[37] – Employer	C[37] – Contractor	D and C[38] – Employer	D and C[38] – Contractor
GEOSYNTHETIC				
Design life	x[39]		x	
Design temperature				x
Type of geosynthetic (geogrid, geotextile…) [40]	x			x
Tension strength in both direction	x			x
Requirements on tests that should be performed to verify the strength and the strain characteristic	x		x	
Type of joint and requirements on its strength	x			x
Distance between layers of reinforcement	x			x
Size of the mesh for geogrid related to surrounding soil	x			x
Type of polymer and its durability(mecahnical, chemical)	x			x
Durability to UV-radiation	x			x
FILLING MATERIAL				
Design life	(x)		x	
Grading, permeability	x			x
Thickness of layers, compaction	x			x
Durability	(x)			x
DRAINAGE				
Type of drainage, drainage capacity	x			x
Design life	(x)			x
Durability	(x)			x
FACING				
Design life	(x)		x	
Type of facing	x			x

[37] Construction contract
[38] Design and construct contract
[39] x party marked with x should provide the opposite party with information
[40] do not mentioned specific product

Table 10.4 Information about material properties for Soil Nailing (ex)

	TYPE OF CONTRACT			
	C[41]		D AND C[42]	
INFORMATION ABOUT PROPERTIES OF THE MATERIALS	Employer	Contractor	Employer	Contractor
SOIL NAIL				
Design life	(x)[43]		x[44]	
Strength of the soil nail (tension, shear and bending)	(x)			x
Pullout capacity	(x)			x
Type of soil nail (grouted, driven…) [45]	x			x
Durability	x			x
SOIL PROPERTIES				
Assumed design values	(x)			x
Assumed groundwater conditions	(x)			x
DRAINAGE				
Type of drainage	x			x
Design life	x		x	
Drainage capacity	(x)			x
Durability	(x)			x
FACING				
Design life	(x)		x	
Type of facing	x			x

[41] Construction contract
[42] Design and construct contract
[43] (x) the employer has this information and may provide the contractor with it
[44] x party marked with x should provide the opposite party with information
[45] type but not specific product

Table 10.5 Information related to execution and control (ex)

INFORMATION RELATED TO EXECUTION AND CONTROL	TYPE OF CONTRACT C[46] Employer	C[46] Contractor	D AND C[47] Employer	D AND C[47] Contractor
EXECUTION OF REINFORCED FILL				
Orientation of the geosynthetic	x[48]			x
Direction of the laying and compaction of filling	x			x
How to perform the joints	x			x
Whether or not the reinforcement should be pre-stretched	x			x
Restrictions such as: maximum time that the geosynthetic may be exposed to sun, thickness of layer above the geosynthetic before allowing traffic on it, temperature, equipment, environment	x			x
EXECUTION OF SOIL NAILING				
Information about sequences of excavation	x		x	
Any restriction for the execution	x		x	
RECORDS				
Specify the different records and their extent	x		x	
How and when the records should be provided to who	x		x	
Responsibility to file the records and for how long time	x		x	
CONTROL				
The required control (type of tests, number…)	x			x
Extent of the control	x			x
Interval for control	x			x
Limit value	x			x
Action programme if the limiting values are exceeded	x			x

10.3 RESPONSIBILITY FOR ACTIVITIES

In Table 10.6 to Table 10.8 a number of activities are listed. The contract document should clearly define who is responsible for each activity, employer or contractor. Either a consultant or a technical representative may perform the responsibilities of the employer. However, it is equally important to clearly define the responsibility of these two parties before the com-mencement of the work.

It is important to define the supplier or source of materials before the commencement of the work. After each table a list of activity is found that either the employer or the contractor could make an agreement with the supplier to be responsible for, before the commencement of the work. However, this does not change the responsibility between the employer and the contractor.

[46] Construction contract
[47] Design and construct contract
[48] x party marked with x should provide the opposite party with information

Table 10.6 to Table 10.8 should be used as a guideline for how the responsibility for different activities can be divided between the parties. However, the actual responsibility in any particular project should always be defined in the contract.

Table 10.6 Activities common for both application (ex.)

		TYPE OF CONTRACT			
		C[49]		D AND C[50]	
	WHO IS RESPONSIBLE FOR DIFFERENT ACTIVITIES	Employer	Contractor	Employer	Contractor
1	Provision of site investigation for execution	x		x	
2	Obtain all legal authorisations necessary for the execution from authorities and third parties	x		x	
3	Assessment of the site investigation data with respect to the design assumptions	x			x
4	Definition of the service life (permanent/temporary)	x		x	
5	Assessment of the construction feasibility of the design	x			x
6	Definition of the working sequence	x			x
7	Instruction to all parties involved of key items in the design criteria to which special attention should be directed	x			x
8	Definition of level of safety and geotechnical class	x		x	
9	Definition of tolerable limits of the effects of the execution (deformations, settlements, noise, grouting loss etc.) Especially considering the neighbouring structures	x		x	
10	Specification for monitoring the effects of structure on adjacent structures (type and accuracy of instrumentation, frequency of monitoring and measurements) and for interpreting the results	x		x	
11	Monitoring of the effects of work on adjacent structures and presenting the results.		x		x
12	Supervision of the works, including the definition of the quality requirements	x		x	
13	Definition of safety factors to be employed	x		x	
14	Responsibility of records during execution		x		x

Activities that the supplier could be responsible for after agreement with the contractor or employer.

- If required - instructions regarding the working sequence
- Definition of the working sequence
- Instruction to all parties involved of key items in the design criteria to which special attention should be directed

[49] Construction contract
[50] Design and construct contract

Table 10.7 Activities related to Reinforced soil (ex.)

		TYPE OF CONTRACT			
		C[51]		D AND C[52]	
	WHO IS RESPONSIBLE FOR DIFFERENT ACTIVITIES	Employer	Contractor	Employer	Contractor
1	Decision to use reinforced soil. *Preliminary trials and testing if required*	x		x	
2	Overall design of reinforced soil structure. Determination of necessary properties of the geosynthetic	x			x
3	Requirement for corrosion protection	x			
4	Consideration of the relevant temporary phases of execution	x			x
5	Execution of material tests	x			x
6	Execution of tests at the site if required		x		x
7	Evaluation of the results of the preliminary tests	x			x
8	Selection of the reinforced soil system	x			x
9	Detailing of the corrosion protecting system	x	x		x
10	Assessment of the reinforced soil system and definition of the working procedures	x			x
11	Definition of the dimensions, location and orientation of geosynthetic	x			x
12	Execution of works, including monitoring		x		x

Activities that the supplier could be responsible for after agreement with the contractor or employer:

- Overall design of reinforced soil structure. Determination of necessary properties of the geosynthetic
- Consideration of the relevant temporary phases of execution
- Execution of material tests
- Execution of tests if required
- Detailing of the corrosion protecting system
- Definition of the dimensions, location and orientation of geosynthetic
- Execution of works, including monitoring of the structure

[51] Construction contract
[52] Design and construct contract

Table 10.8 Activities related to Soil Nailing (ex)

	WHO IS RESPONSIBLE FOR DIFFERENT ACTIVITIES	TYPE OF CONTRACT C[53] Employer	C[53] Contractor	D AND C[54] Employer	D AND C[54] Contractor
1	Decision to use soil nailing	x		x	
2	Perform preliminary trials and testing if required, provision of specification	x			x
3	Overall design of soil nailing, calculation of the soil nailing forces required and the overall stability requirements	x			x
4	Requirements for corrosion protection, consideration of the relevant temporary phases of execution	x			x
5	Execution of trials if required	x			x
6	Selection of the soil nailing system, detailing of the corrosion protecting system, specification of nailing spacing and orientation and of nailing load	x			x
7	Assessment of the soil nailing system and definition of the working procedures	x			x
8	Execution of soil nailing works, including monitoring of the nailing parameters		x		x

10.4 DEFINITION OF REPORTING PROCEDURE

The following aspects should also be agreed upon before starting the work:

- Reporting procedure for how to deal with unforeseen circumstances
- Reporting procedure if the conditions at the site are not according to the assumptions in the design. Relevant actions to be taken under different circumstances
- Reporting procedure if observational method is applied

[53] Construction contract
[54] Design and construct contract

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NORDIC GUIDELINES

FOR

REINFORCED SOILS AND FILLS

ANNEX A TO F

Nordic **G**eosynthetic **G**roup

May 2003

Revision A – February 2004

Table of Content

A MATERIAL PROPERTIES

A.1 Reinforced fill

A.1.1 General

Test values of material properties are generally determined by index test methods, ISO-, CEN-, ASTM-standardised, which allow a direct comparison of different products. Many of these index methods have shortcomings especially if such determined material parameters are used for design purposes. The material properties relevant to reinforced fill applications involve the following categories:

- Short term mechanical properties
- Long term mechanical properties and creep
- Reinforcing material/ soil interaction
- Resistance to damage during installation
- Long term durability

For some of the material properties alternative standard test procedures will be listed, but in all cases priority has to be given to EN ISO, ISO, EN or the transformed identical national standards.

A.1.2 Short Term Mechanical Properties for Geosynthetic Reinforcing Materials

A.1.2.1 Tensile Strength and Dependent Strain

Tensile strength is measured according to different standards on different specimens. Strain rates have a remarkable influence for some polymers and for high strength grids of polyesteryarns or aramide yarns different clamping problems arise.

Table A.1 Standard Test Procedures to Determine Characteristic Values of Strength

Standard	Specimen size, mm[1]	Strain rate
EN ISO 10319 : 1996	200 x 100	20 %/min
ISO 5081	50 x 200	variable f (ε_u)
ASTM D 4595	200 x 100	10 %/min

The tensile test is the basis of characteristic values of strength and strain for reinforcement applications. Strain should be measured on the specimen by means of an extensometer.

A.1.2.2 Resistance Against Puncture

During installation mineral fill material is dropped on the geosynthetic reinforcement, then spread and compacted by vibration and heavy static loads. Both the static and the dynamic puncture test determine index parameters, which may be used as an indication on the resistance of the reinforcement against puncture.

STATIC PUNCTURE TEST (CBR-TEST) (EN ISO 12236 : 1996)

This test is using elements of a soil mechanical test apparatus (California bearing ratio = CBR) and also named CBR-test. In the test the geosynthetic material is fixed in rings of inner diameter 150 mm and a plunger of 50 mm diameter is moved with a speed of 50 ± 10

[1] width × free length

mm/ min onto and through the fixed specimen recording force and strain. The test is applicable to woven and nonwoven geotextiles , but it is not applicable to grids.

DYNAMIC PERFORATION TEST (ISO 13433, EN 918 : 1995) (CONE DROP TEST)

A steel cone of 1000 g mass with defined angle and sharpness is dropped from 500 mm above the specimen onto the geotextile reinforcing material which is fixed in rings of inner diameter 150 mm. The diameter of a created hole is measured by means of a measuring cone of 600 g weight and a smaller angle than the drop cone with a metering scale in mm.

FRICTION PROPERTIES (EN ISO 12957 : 1998)

Tensile load is tranferred to the reinforcing geosynthetics from the soil via friction. The friction ratio to normal stress is usually expressed as an angle of friction. Lower normal stresses may be tested by inclined plane test and higher normal stresses by direct shear ("shear box test") or by pulling the geosynthetic out of the soil.

DIRECT SHEAR (prEN ISO 12957-1)

The friction partners to be tested (geosynthetic reinforcing material/ soil) are placed separately, one in an upper box and the other in a lower box. The lower box is moved in strain control (for index testing: 1 mm/min) while recording force and strain. The results for the normal stresses of 50, 100, 150 kPa are plotted and the value of friction angle is calculated.

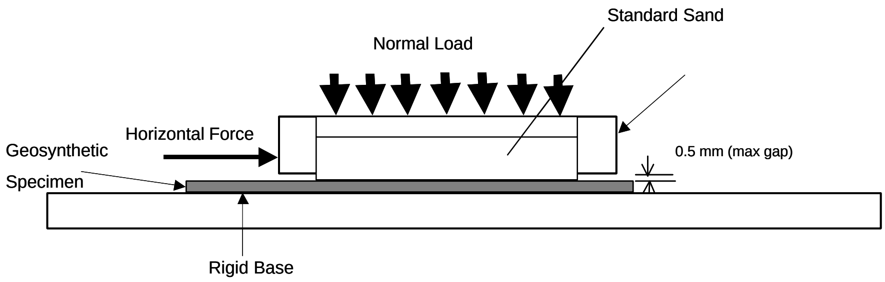


Figure A.1 Principal arrangement of direct shear test.

INCLINED PLANE TEST (prEN ISO 12957-2)

The friction partners to be tested (geosynthetic reinforcing material/ soil) are set up on a inclinable table. Slip of materials and inclination are measured while lifting the table by 3 degrees/ min. A movement of 50 mm stops the test and gives the angle of friction for the chosen material combination. The normal stress must be recalculated for the resulting angle.

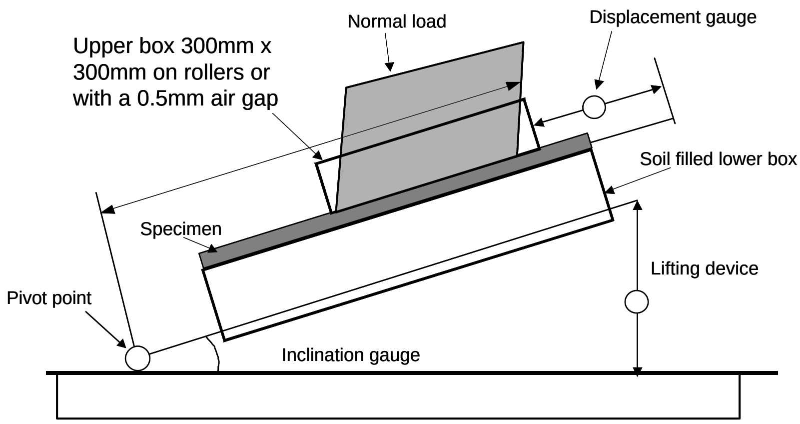


Figure A.2. Inclined plane test set up

PULL-OUT RESISTANCE (prEN 13738)

A strip of the material (width depending on box width) is pulled out of a soil filled box, where the soil is loaded normal to the geosynthetic reinforcement material. Force and strain are recorded for several points of the material inside the box. Results may be maximum force at rupture or slippage or plots of force versus strain. The corresponding standardised index test for determining pull-out resistance in soil is prEN 13738.

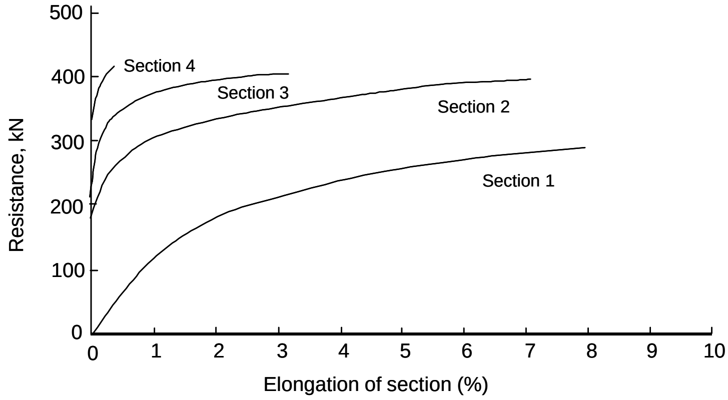


Figure A.3 Pull-out resistance versus percent strain (%) for different sections of the pull-out specimen

A.1.2.3 Long Term Mechanical Properties

TENSILE CREEP AND CREEP RUPTURE (EN ISO 13431 : 1996)

Tensile creep tests give information on time-dependent strain at a constant load. Loads for creep testing are most often dead weights. Creep rupture tests give time until failure at a constant load. A strain measurement is not necessary for creep rupture curves. The EN-ISO creep tests require 1000 hours testing, for creep rupture extrapolation to long-term (30, 60, 120 years) a test duration greater than 10 000 hours is necessary. Results are plotted for creep as linear deformation versus log-time, for creep rupture linear or log-stress level versus log-time. From creep curves at different stress levels isochronous stress strain curves may be derived for calculation of the structure's deformation at a given time. Typical curves are shown in Figure 2.5 The creep behaviour of geosynthetics depends mainly on the polymer used and how the base materials (yarns, tapes) are treated thermomechanically.

A.1.2.4 Damage During Installation (Index) (ENV ISO 10722-1 : 1997)

As the installation can be the most severe attack to geosynthetic reinforcing elements during their service life, an estimation of the resistance is to be tested. The EN-ISO standard applies a cyclic load on a platen (100 x 200) pressing via a layer of aggregates onto the geosynthetic to be tested. After 200 cycles between 5 kPa and 900 kPa maximum stress the specimen is exhumed and may be tested for residual strength. A performance test requires the soil and fill of the site and proper equipment to spread and compact the material.

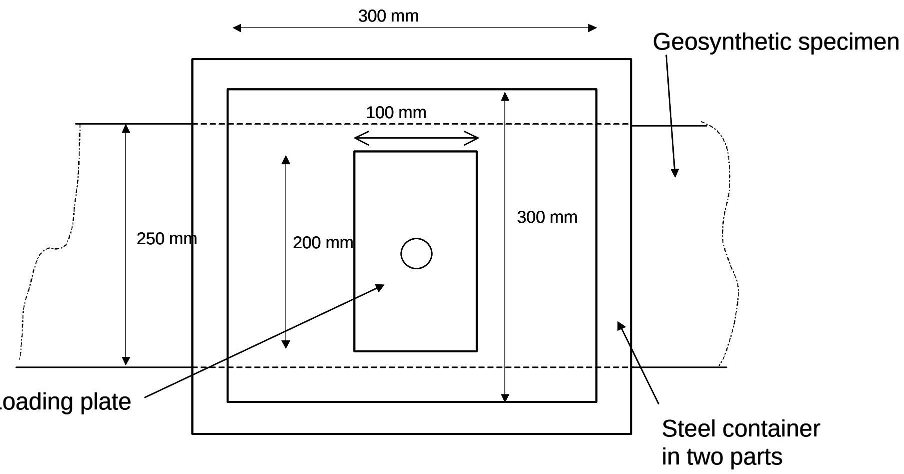


Figure A.4 Schematic plan of apparatus

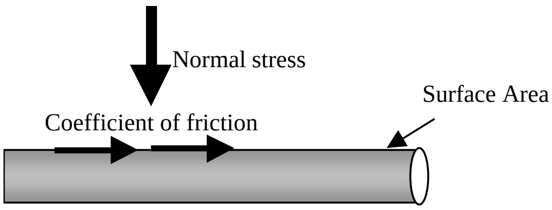
A.1.2.5 Durability of Geosynthetic Reinforcement Products

The procedure for judging long-term stability of geosynthetic reinforcement materials is outlined in the CEN report – “Guide to durability” (document CR ISO 13434). The requirements on the material properties of the geosynthetic reinforcement are covered by EN 13251 (the CEN harmonised standard applicable for CE-marking), and requirements on durability are given in the normative Annex B – “Durability aspects”.

A.2 Soil-nailing

A.2.1.1 Factors influencing the pull-out capacity

The total pull-out capacity that may be mobilised along the nail will depend on three main parameters; coefficient of friction at the nail interface, normal stress and the perimeter of the nail, see Figure A.5.



$$T_L = m\ s'_N q$$

m coefficient of friction

s'_N effective normal stress acting on the nail

q nail perimeter

Figure A.5 Factors influencing the pull-out capacity

The normal stress is influenced by factors such as:

- Type of soil (relative density, angle of internal friction, stiffness, cohesion)
- Water content, pore pressure
- Type of nail
- Installation method
- Displaced sand volume
- Grouting pressure, borehole geometry
- Installation depth, overburden pressure
- Time

The <u>coefficient of friction</u> is influenced by:

- Angle of internal friction (which in turn depends on the mineral contents, grain shape, grain roughness, stress level, density, and water content)
- Cohesion, clay content
- Water content
- Type of nail (material, nail texture andsurface roughness)

The obtained <u>nail surface area</u> will be influenced by the following factors:

- Type of nail
- For grouted nails: coefficient of uniformity (grain size distribution), fissures and week seems, water/cement ratio of grout, size of solids in grout, grouting pressure, shape of borehole, density
- For expansion bolts: degree of expansion, overburden pressure
- For driven nails: ribs

Consequently the pull-out capacity depends on a number of factors which interact with each other in a complex way. An attempt to systematise the influence of the different soil and nail parameters is made in Table A.2. The influence of time is not considered in the table. However, test results indicate that for driven nails without ribs the pull-out capacity tends to increase with time.

Table A.2 Effect of an increase in the magnitude of different parameters on the pull-out capacity a) soil parameters b) nail parameters

a) Increase of the following **soil parameters**:	Driven nail			Grouted nail		
	s'_N	tan d	q	s'_N	tan d	q
relative density	↑	↑	-	↑	↑	↓
angle of internal friction	↑	↑	-	↑	↑	-
cohesion - clay content	-	↓	-	-	↓	↓
water content - pore pressure	↓	↓[2]	-	↓	↓	-
coefficient of uniformity, C_U	-	↑	-	-	↑	↓
overburden pressure	↑↓	↑↓	-	↑↓	↑↓	-
soil modules	↑	↑	-	↑	↑	↓
b) Increase of the following **nail parameters**:	Driven nail			Grouted nail		
	s'_N	tan d	q	s'_N	tan d	q
soil displacement	↑	-	↑	↑	-	↑
surface roughness	↑	↑	-	↑	↑	-
surface texture including any irregularities (e.g. ribs, lumps of grout)	↑	↑	↑	↑	↑	↑
degree of expansion	↑	-	↑	n. a.	n. a.	n. a.
grouting pressure	n. a.	n. a.	n. a.	↑	-	↑
water/cement ratio grout	n. a.	n. a.	n. a.	-	-	↑
percentage of solids in grout	n. a.	n. a.	n. a.	-	-	↑

↑ increase and ↓ decrease in pull-out capacity, n. a. not applicable, - effect not determined

[2] for sand/steel interface marginal influence (Potyondy, 1961)

B PRINCIPLES OF DESIGN

B.1 Calibration of Partial Factors

When calibrating partial factors there are a few optional methods:

- Calibration with reliability analysis – *e.g.* FORM (First Order Reliability Method)
- "Design Value Method"
- Adjustment of traditional calculation models
- A combination of the above mentioned methods

A proper reliability analysis is to be preferred in many respects. It gives the opportunity to include a relevant description of the physical problem at hand and to incorporate the variable uncertainty in a systematic way. Furthermore, the solution of the problem can be regarded as an objective solution. However, the mathematics of the problem becomes easily troublesome and might not be worth the effort. In such cases it can be worthwhile to use a more simple method, here denoted Design Value Method. Below will be given a short description of one alternative of such a procedure

ILLUSTRATION OF A "DESIGN VALUE METHOD"

An important characteristic of reliability analysis, such as FORM, is the so called sensitivity factors, α_1, α_2,…α_n. The sensitivity factors are measurements of the influence of the different basic variables on the problem at hand and represent a very important result of a reliability analysis, for a thorough understanding of a complex problem. In the reliability analysis the values of the sensitivity factors are a result of the analysis. In the design value method the sensitivity factors are input parameters. For the sensitivity factors the following equation applies:

$$\sum_{i=1}^{n} a_i^2 = 1 \qquad \text{(B.1)}$$

This means that the sensitivity factor squared serves as a weight factor for the variable at hand (the sum of the squares is 100%). The procedure for the simplified Design Value Method then becomes:

1. Identify different uncertain variables, i.e. variables to which partial factors are applied
2. Appraise a weight factor of the different variables, i. e. to consider both physical influence of the variable and uncertainty of its value
3. Normalise the sum of the weight factors to 100%
4. Calculate sensitivity factors, α_i (= the root of the weight factor)
5. Appraise the uncertainty of each variable, e.g. by the coefficient of variation, V
6. Give the target safety level[1], in form of the value of the reliability index b
7. Calculate the partial factors as $\exp(a_i \cdot b \cdot V_i)^2$

---

1 In Sweden there are 3 classes of safety whereas in the Eurocode there is 1 class.
Class of safety 1: β=3,7
Class of safety 2: β=4,3
Class of safety 3: β=4,7
The class of safety is based on an evaluation of health and life.

EXAMPLE - REINFORCED SLOPE

Below is given an example of how to determine a set of the partial factors for a reinforced slope. The problem is based upon the formulation of the problem given by equation 3.11 in Chapter 3 Principles of design, with no respect to model uncertainty of the action effects, i.e. γ_{Sd}=1.0. The reliability index is set to *b*=4.7, which corresponds to a nominal probability of failure =10^{-5}

Table B.1 Partial factors in Example

Factor	Weight [%]	Standardised weight [%]	α	V [%]	$\gamma = e^{a \cdot b \cdot V}$
c'	20	13.8	0.37	30	1.68
ϕ'	15	10.3	0.32	10	1.16
R_N	40	27.6	0.53	10	1.28
γ	20	13.8	0.37	10	1.19
g	15	10.3	0.32	20	1.35
q	10	6.9	0.26	50	1.84
R_d	25	17.2	0.41	25	1.62
Sum	145	100			

Comments

1. *The example above is only meant as an illustration of how to determine partial factors. In this simplified Design value method, in comparison to FORM, the appraise of the weight of each parameter is made subjectively, while in FORM a calculation is made of "correct" values in respect of :*
   - *physical influence on the problem and*
   - *uncertainty – coefficient of variation*
2. *A number of different combinations of partial factors may give the same final solution. In problems where some of the partial factors are given, in e.g. Eurocodes, it is advisable to use these values and adjust others, which are not in the code, accordingly.*
3. *As mentioned before, in the partial factor 1.35 for permanent load in the Eurocode, the contribution of model error is included.*

B.2 Partial Factors from NAD and National Standards

B.2.1 Partial Factors of Actions

In the Eurocode ENV 1997-1 there is only one class of safety, but in for example Sweden there are three different classes. Different classes of safety are defined to consider risks to life and property, where class 1 is defined as little risk and class 3 as great risk to life and property. The partial factors to use in Sweden according to 2.2 (1)P Swedish NAD SS-ENV 1991-1 are as follows.

Class of safety 1: γ_d=0.83
Class of safety 2: γ_d=0.91
Class of safety 3: γ_d=1.0

2 $e^{a \cdot b \cdot V}$ is an approximate formula to determine partial factors based on log normal distributed parameters with a moderate uncertainty. Furthermore it is based upon the assumption that the characteristic value is chosen as the mean value. This should not be interpreted such that the log normal distribution is a comprehensive distribution, which can replace other distributions in detailed analyses. Only that it might work as a simple engineering tool for rapid assessments.

The partial factor is applied to the action. The safety level used in Eurocodes corresponds to class of safety 3 according to SS-ENV 1991-1.

B.2.2 Combinations of Actions

The main combination rule for Ultimate Limit States is in the Swedish NAD, replaced by two alternative rules. These mean that the dominant variable action is combined with either

- A reduced value of the permanent actions

or

- The permanent actions are combined with the combination values of all variable actions.

The reduction factor in the first case is 0.89. In the mentioned combination rules are also incorporated the factors for the class of safety, see B.2.1.

B.2.3 Partial Factors of Geotechnical Parameters

Table B.2 Partial material factors g_m[3] - ultimate Limit State in persistent and transient situations, according to Norwegian standard NS-ENV 1997-1 NAD:1997

Case	Ground Properties			
	tan ϕ	c'	c_u	q_u
Case A	1.10	1.10	1.20	1.20
Case B	1.00	1.00	1.00	1.00
Case C	1.20	1.20	1.30	1.40

Table B.3 Partial material factors g_m[3] - ultimate Limit State in persistent and transient situations, according to Swedish standard SS-ENV 1997-1 NAD

Case	Ground Properties			
	tan ϕ	c'	c_u	q_u
ABC[3]	1.20	1.60	1.50	1.60

3 In Sweden there is only one case.

C REINFORCED STEEP WALL

C.1 Example

INPUT DATA

A 3 m high reinforced steep wall is to be built with an inclination of 90°. The following data is available with tolerances included in the geometrical parameters. Partial factors according to prENV 1997-1.

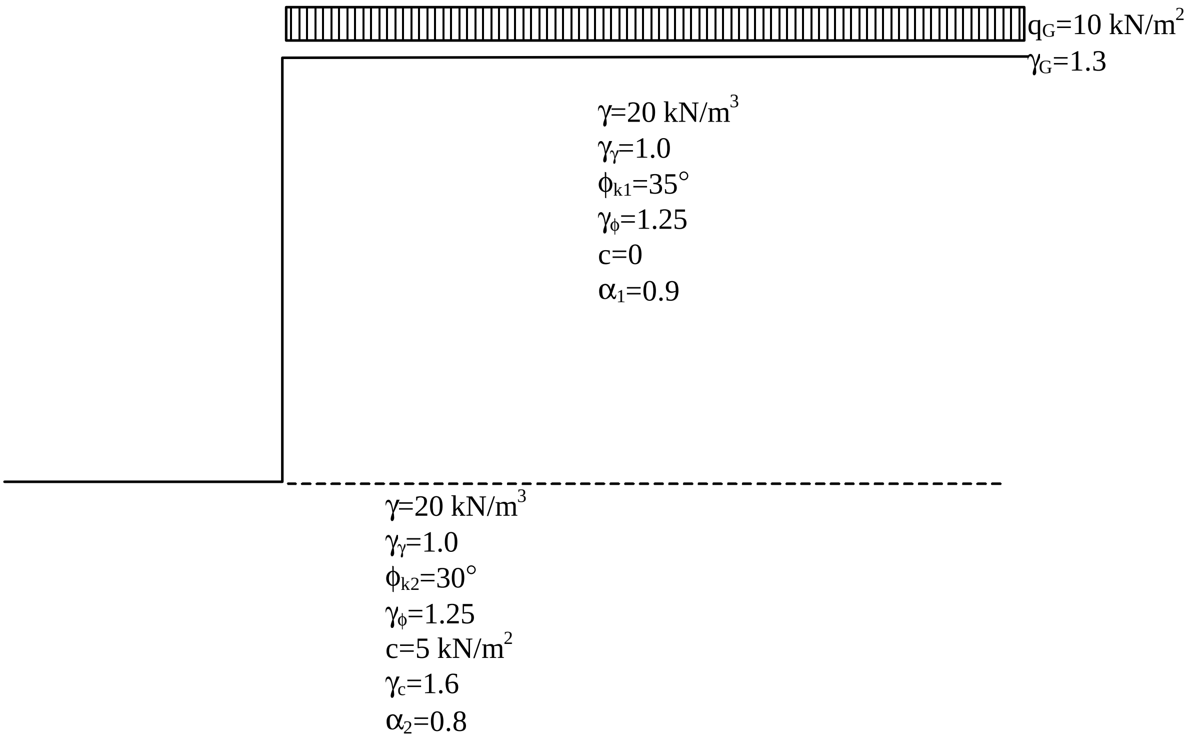


Figure C.1 Example Steep Wall

The partial factor for pull-out resistance and sliding is assumed to be γ_P=1.3 and γ_S=1.3 respectively.

Ground water table is assumed below foundation level.

The friction angle and active earth pressure is

$$\tan \phi_d = \frac{\tan \phi_k}{\gamma_f} = \frac{\tan 35°}{1.25} = 0.56, \textit{ i.e. } \phi_d \approx 29° \qquad (\text{C.1})$$

$$K_{ad} = \tan^2(45° - \frac{\phi_d}{2}) = 0.35 \qquad (\text{C.2})$$

Vertical stress at bottom of fill:

$$s_v d = g \cdot g_g \cdot H + q_G \cdot g_G = 20 \cdot 1 \cdot 3 + 10 \cdot 1.3 = 73 \ \ kN/m^2 \tag{C.3}$$

$$u = 0$$

$$s'_{vd} = s_{vd} - u = 73 \ \ kN/m^2$$

Horizontal stress at bottom level (= maximum horizontal stress):

$$p_{ad,\max} = K_{ad} \cdot s'_{vd} + u = 0.35 \cdot 73 - 0 = 25.6 \ \ kN/m^2 \tag{C.4}$$

REINFORCEMENT SPACING (LAYER THICKNESS):

A layer thickness of 0.5 m is chosen, which requires the following design tensile strength for the reinforcement at the bottom level.

$$S_{vd} = \frac{T_d}{p_{ad,\max}} \Rightarrow T_d = S_{vd} \cdot p_{ad,\max} = 0.5m \cdot 25.6 = 12.8 \ \ kN/m^2 \tag{C.5}$$

LATERAL SLIDING:

$$L_e \geq \frac{0.5 K_{ad} H \left(g_? g \ \ H + 2\left(q_{Q_d} + q_{G_d} \right) \right) g_s}{g_g g \ \ h \ \ \dfrac{a'_1 \tan f'_k}{g_f}} \tag{C.6}$$

<u>To find minimum length of reinforcement at the base, two cases is considered:</u>

SLIDING ABOVE THE REINFORCEMENT:

$$L_e \geq \frac{0.5 K_a H \left(g_g g \ \ H + 2\left(g_Q q_Q + g_G q_G \right) \right) g_s}{g \ \ h \ \ \dfrac{a'_1 \tan f_{k1}}{g_f}} = \frac{0.5 \cdot 0.35 \cdot 3 \left(1 \cdot 20 \ \ \cdot 3 + 2\left(0 + 1.3 \cdot 10 \right) \right) 1.3}{20 \cdot 3 \dfrac{0.9 \cdot \tan 35^\circ}{1.25}} = 1.94 m$$

SLIDING BELOW REINFORCEMENT:

$$L_e \geq \frac{0.5 K_a H \left(g_g g \ \ H + 2\left(g_Q q_Q + g_G q_G \right) \right) g_s}{g \ \ h \ \ \dfrac{a'_2 \tan f_{k2}}{g_f}} = \frac{0.5 \cdot 0.35 \cdot 3 \left(1 \cdot 20 \ \ \cdot 3 + 2\left(0 + 1.3 \cdot 10 \right) \right) 1.3}{20 \cdot 3 \dfrac{0.8 \cdot \tan 30^\circ}{1.25}} = 2.64 \ \ m$$

Minimum length of reinforcement at the base (between fill material and foundation soil): L=2.64 m

TO FIND MINIMUM LENGTH OF REINFORCEMENT AT TOP:

$$L = L_R + L_E \tag{C.7}$$

The active zone at the top level (0.5 m below surface):

$$L_R = (H - h_i)\tan(45° - \frac{f_d}{2}) = (3 - 0.5)\tan(45° - \frac{29°}{2}) = 1.47 \ \ m \qquad (C.8)$$

Anchoring in passive zone at level 0.5 m from the top:

$$s'_{vd} = g \cdot g_g \cdot h_{toplevel} + q_G \cdot g_G - u = 20 \cdot 1 \cdot 0.5 + 10 \cdot 1.3 - 0 = 23 \ \ kN/m^2 \qquad (C.9)$$

$$p_{ad} = K_{ad} \cdot s'_{vd} + u = 0.35 \cdot 23 - 0 = 8.1 \ \ kN/m^2 \qquad (C.10)$$

$$L_E = \frac{p_{ad} S_{Vd}}{\frac{2a_1}{g_p}(\frac{c'}{g_{c'}} + g\, h \tan f'_d)} = \frac{8.1 \cdot 0.5}{\frac{2 \cdot 0.9}{1.3}(\frac{0}{g_{c'}} + 20 \cdot 0.5 \cdot \tan 29°)} = 0.53 \ \ m \qquad (C.11)$$

Anchoring at top L_E = min 1.0 m

Minimum length at top:
$L = L_R + L_E = 1.76 + 1.0 = 2.47\ m$

CONTROL OF ANCHORING AT BOTTOM LEVEL:

$$L_E = \frac{p_{ad,\max} S_{Vd}}{\frac{2a_1}{g_p}(\frac{c'}{g_{c'}} + g \cdot h \cdot \tan f'_d)} = \frac{25.6 \cdot 0.5}{\frac{2 \cdot 0.9}{1.3}(\frac{0}{g_{c'}} + 20 \cdot 3 \cdot \tan 29°)} = 0.28 \ \ m \qquad (C.12)$$

i.e. L_E = min. 1.0 m

CONCLUSION:

Required length of reinforcement:

- Top level: 2.47 m
- Bottom level: 2.64 m

Note: The reinforcement length is often chosen equal for all layers. For this example the difference between the top layer and the bottom layer is small and we use equal length for all layers.

L = 2.7 m for all layers

Tensile Strength:

- Minimum long term design strength: 12.8 kN/m (which may correspond to a polyester reinforcement with short term characteristic strength about 40 – 70 kN/m depending on the conversion factors which are specific for each product, see Table C.1.

Table C.1. Conversion factors related to geosynthetic reinforcement

Conversion factors – material	Factor
Factor of creep - depending on lifetime and only relevant when using the short term tensile strength.	?$_1$
Installation damage factor	?$_2$
Chemical and biological degradation	?$_3$

EXAMPLE POLYESTER GRID:

$$T_d = \frac{T_{sts}}{g_m} \cdot h_1 \cdot h_2 \cdot h_3 \Rightarrow T_{sts} = \frac{T_d}{(h_1 \cdot h_2 \cdot h_3)} \cdot g_m = \frac{12.8}{(0.5 \cdot 0.7 \cdot 0.9)} \cdot 1.3 = 52.8 \ \ kN/m$$

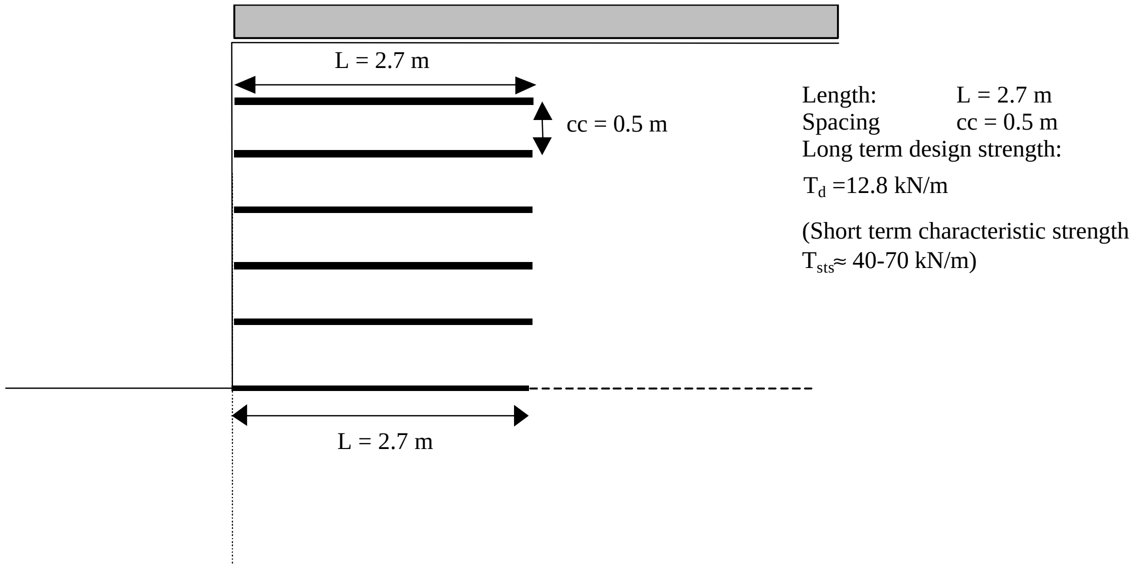


Figure C.2 Final layout of the example

C.2 Potential Failure Surfaces – Stress Ratio with Depth

When using inextensible reinforcement in the structure the potential failure surface is as illustrated in Figure C.3. For design of such reinforcement calculations are not further described here but could be done according to Publication No FHWA-NHI-00-043.

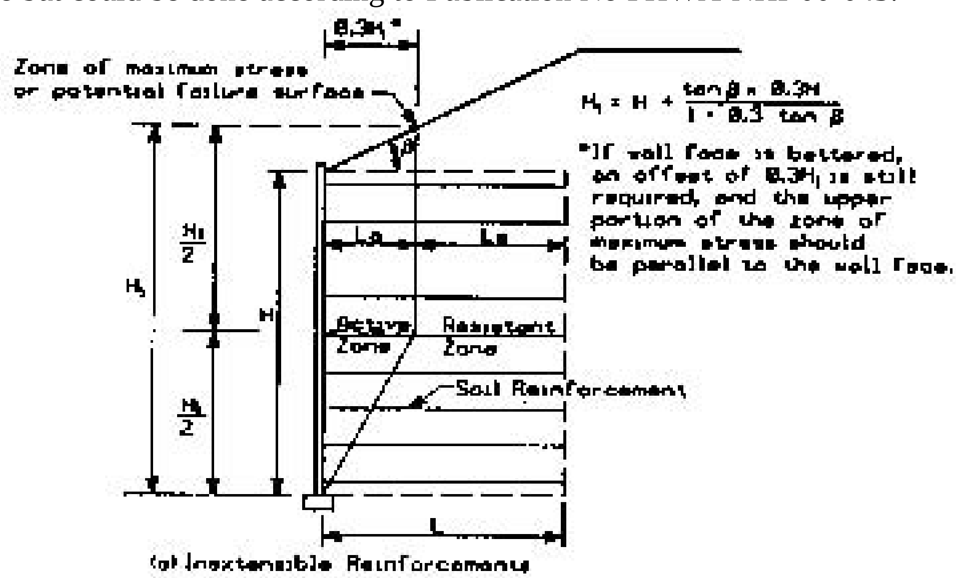


Figure C.3 Location of potential failure surface from internal stability design of MSE walls[6]. Inextensible Reinforcement

[6] Figure is from Publ. No.: FHWA-NHI-00-043 (Federal Highway Administration (USA))

Figure C.4 shows variation of stress ratio with depth. *I.e.* factor to multiply the active earth pressure coefficient when using inextensible reinforcements.

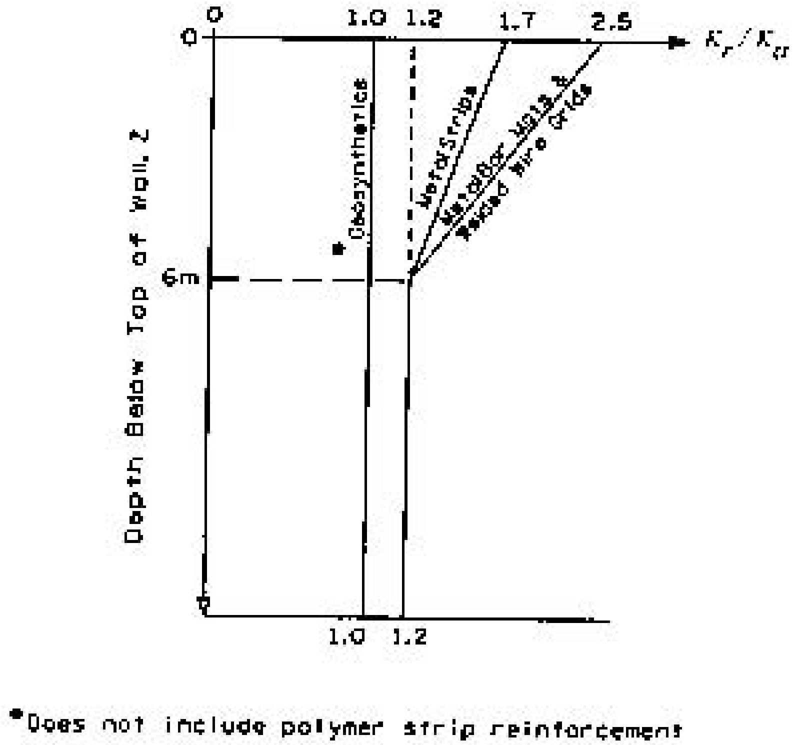


Figure C.4 Variation of stress ratio with depth in a MSE wall. Figure is from Publ. No.: FHWA-NHI-00-043 (Federal Highway Administration (USA)

D EMBANKMENT ON SOFT SOIL

D.1 Example

A 2.5 m high embankment with basal reinforcement is built on soft clay with constant undrained shear strength with depth, c_u=10 kN/m^2. The soft layer thickness is 2.5 m. Partial factors according to Eurocode 7 (prENV 1997-1) are applied. No surcharge load is applied. A geotextile is used as reinforcement.

D.1.1 Ultimate Limit State, Input Data:

Geometry and material data:	
Embankment height:	H = 2.5 m
Unit weight, fill material	g = 20 kN/m^3
Angle of friction	f = 38°
Undrained shear strength at top of soft soil layer:	C_{u0} = 10 kN/m^2
Factor for increased shear strength with depth:	x = 0
Soft soil layer thickness:	t = 2.5 m
Permanent load (dead load):	q_G = 0
Variable load (live load):	q_Q = 0

PARTIAL FACTORS ACCORDING TO EUROCODE 7 (ENV 1997-1) (TABLE 3.4, CASE C):

Load factors:	
Applied to soil unit mass:	$\gamma_\gamma = 1$
Applied to permanent load:	$\gamma_G = 1$
Applied to variable load:	$\gamma_Q = 1.3$
Soil material factors:	
Applied to tan ϕ:	$\gamma_\phi = 1.25$
Applied to cohesion intercept:	$\gamma_{c'} = 1.6$
Applied to undrained shear strength:	$\gamma_{Cu} = 1.4$
Reinforcement safety factors:	
Reinforcement material factor:	$\gamma_{re} = 1.3$
Applied to sliding on reinforcement:	$\gamma_s = 1.1$
Applied to pull-out resistance:	$\gamma_p = 1.3$

PARTIAL FACTORS GIVEN FROM TESTS ON ACTUAL REINFORCEMENT USED IN COMBINATION WITH ACTUAL FILL MATERIAL AND SOFT SUBSOIL (OR GIVEN BY MANUFACTURER/EMPIRICAL VALUES):

Interaction coefficient geotextile/fill material:	$\alpha_1 = 0.7$
Interaction coefficient geotextile/soft soil:	$\alpha_2 = 0.7$
Installation damage effect:	$\eta_2 = 0.77$ ($\gamma_i = 1.3$)
Environmental impact on durability:	$\eta_3 = 0.91$ ($\gamma_d = 1.1$)
Creep factor:	$\eta_1 = 0.5$ ($\gamma_{cr} = 2.0$)
(or the strength could be given as long term strength)	

As a start the slope inclination is assumed to be 1:2, *i.e.* slope length, L_s = *2*H = 2*2.5 =5 m*.

LOCAL STABILITY:

The local stability of the embankment sideslope should be checked, see Equation 5.1:

$$\left(\frac{1}{n}=\right) \quad \frac{H}{L_s} \leq \frac{\tan f'_k}{g_f} \qquad \text{(D.1)}$$

$$\frac{\tan j'_k}{g_f} = \frac{\tan 38}{1.25} = 0.62 \qquad \frac{1}{n} = \frac{1}{2} = 0.5 \qquad \text{i.e. ok.}$$

LATERAL SLIDING STABILITY:

The reinforcement tensile load T_{ds} needed to resist the outward thrust of the embankment is calculated, see equation 5.2.

To calculate the horizontal force in the embankment we need first to calculate the active earth pressure coefficient, K_a , as given in equation 5.3:

$$K_{ad} = \tan^2\left(45° - \frac{f''_d}{2}\right) \qquad \text{(D.2)}$$

where

$$f_d = \arctan\left(\frac{\tan f_k}{g_f}\right) \longrightarrow K_a = 0.31$$

The tensile load generated from lateral sliding, T_{ds}:

$$T_{ds} = 0.5K_{ad}\left(g_{?}g \;\; H + 2(q_{Q_d} + q_{G_d})\right)H \qquad \text{(D.3)}$$

T_{ds} = 0.5*0.31*(1.0*20*2.5+2(1.3*0+1.0*0))*2.5=19.4 kN (per metre 'run')

The minimum reinforcement bond length, L_e , to prevent horizontal sliding, see Equation 5.4

$$L_e \geq \frac{0.5K_{ad}H\left(g_{?}g \;\; H + 2\left(q_{Q_d} + q_{G_d}\right)\right)g_s}{g_g g \;\; h \;\; \dfrac{a'_1 \tan f'_{k1}}{g_f}} \qquad \text{(D.4)}$$

where h is average fill height over the reinforcement bond. Iteration on h is necessary to find the minimum length required, but a conservative assumption is to assume *h = H/2* (i.e. assume L_e = L_s). If calculated L_e £ L_s the slope inclination is OK in regard to lateral sliding: *h = H/2 = 2.5/2 = 1.25 m*

$$L_e \geq \frac{0.5 \cdot 0.31 \cdot 2.5 \cdot \left(1.0 \cdot 20 \cdot 2.5 + 2\left(1.3 \cdot 0 + 1.0 \cdot 0\right)\right) \cdot 1.1 \cdot 1.0}{20 \cdot 1.25 \cdot \dfrac{0.7 \cdot \tan 38°}{1.25}} = 1.95 \;\; m \qquad \text{(D.5)}$$

i.e. OK (L_e < L_s).

FOUNDATION EXTRUSION STABILITY:

First the minimum side slope length and reinforcement length to prevent foundation extrusion is calculated

To find minimum $L_s = L_{ext}$ equation 5.6 is used:

$$L_{\text{ext}} \geq \frac{\left(g_{?} g_{1} H + q_{\text{G}d} + q_{\text{Q}d} - \frac{\left(4C_{\text{u0}} + 2x\ z_{\text{i}}\right)}{g_{\text{Cu}}} \right) z_{\text{i}}}{\frac{(1 + a_{2}) c_{\text{u0}} + x\ z_{\text{i}}}{g_{\text{Cu}}}} \geq 0 \qquad \text{(D.6)}$$

Iteration on z_i is necessary for the case with increasing undrained shear strength with depth. The larger L_{ext} is to be used. Here the undrained shear strength is constant with depth in the soft soil layer, and the most unfavourable case is for $z_i = t$ (total layer thickness) < $1.5H$.

$$L_{\text{ext}} \geq \frac{\left(1.0 * 20 * 2.5 + 1.3 * 0 + 1.0 * 0 - \frac{(4 * 10 + 2 * 0 * 2.5)}{1.4} \right) * 2.5}{\frac{(1 + 0.7) * 10 + 0 * 2.5}{1.4}} = 4.41\ m \qquad \textit{(D.7)}$$

i.e. OK

Tensile load generated in the reinforcement due to extrusion is calculated for the case $L_s = L_{ext}$, see Equation 5.7. I.e. we assume that the tensile load will be reduced if Ls is increased, and that our calculation therefore is conservative:

$$T_{\text{rf}} = \frac{a_{2} c_{\text{u0}} L_{\text{ext}}}{g_{\text{Cu}}} = \frac{0.7 * 10 * 4.41}{1.4} = 22.0\ kN\ \ (per\ metre\ \ 'run') \qquad \text{(D.8)}$$

GLOBAL STABILITY (ROTATIONAL STABILITY)

A computer program would be convenient to use for this calculation because a lot of calculations/iteration have to be done. The embankment is here calculated using REmbank, which uses the rotational stability calculation procedure shown in Chapter 5. The result window is shown in Figure D.1.

Maximum tensile force is T_{Rc}=*24.42 kN*. The calculation also showed that the required reinforcement bond length outside the shoulder L_b was shorter than the one calculated in the foundation extrusion analysis (the result is not shown here).

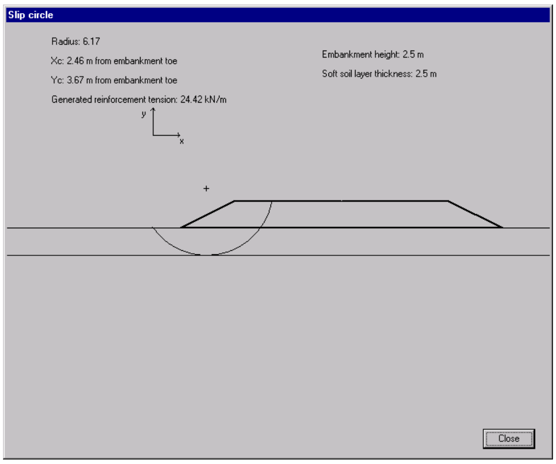


Figure D.1 Result window from REmbank; slip circle analyses.

REINFORCEMENT BOND LENGTH

Necessary bond length outside the embankment shoulder, L_b, is the greater of L_b due to rotational stability, L_e due to lateral sliding and L_{ext} due to foundation extrusion.

This gives a minimum required bond length $\underline{L_b = L_{ext} = 4.41\ m}$
However, a good practice could be to install the reinforcement all the way out to the toe of the embankment.

THE MAXIMUM ULTIMATE LIMIT STATE TENSILE FORCE Tr :

The maximum ultimate Limit State tensile force T_r to be resisted by the basal reinforcement is the greater of

a) the maximum tensile force, T_{ro}, needed to resist the Rotational Limit State per metre 'run'; or
b) the sum of the maximal tensile force, T_{ds}, needed to resist lateral sliding per metre 'run' and the maximum tensile force, T_{rf}, needed to resist foundation extrusion per metre 'run'. (*i.e.* $T_{ds}+T_{rf}$)

T_{ro} = 24.4 kN

$T_{ds}+T_{rf}$=19.4+22.0=41.4 kN

I.e. T_r =41.4 kN per metre 'run'

REQUIRED STRENGTH FOR THE REINFORCEMENT:

To ensure the ultimate Limit State governing reinforcement rupture is not attained over the design life for the reinforcement the following condition should be used:

$$T_d \geq T_r \qquad \text{(D.9)}$$

Where

T_d is the design strength of the actual reinforcement based on data given for the reinforcement and the design life over which the reinforcement is needed

T_r is the maximum tensile load in the reinforcement (from calculations above)

Minimum required long term design strength T_d = 41.4 kN/m (which may correspond to a polyester reinforcement with short term characteristic strength about 120 – 200 kN/m depending on the conversion factors which are specific for each product, see Table D.1).

Table D.1 Conversion factors related to geosynthetic reinforcement.

Conversion factors – material	Factor
Factor of creep – depending on lifetime and only relevant when using the short term tensile strength	$?_1$
Installation damage factor	$?_2$
Chemical and biological degradation	$?_3$

Example polyester grid:

$$T_\mathrm{d} = \frac{T_\mathrm{sts}}{g_m} \cdot h_1 \cdot h_2 \cdot h_3 \Rightarrow T_{sts} = \frac{T_d}{(h_1 \cdot h_2 \cdot h_3)} \cdot g_m = \frac{41.4}{(0.5 \cdot 0.77 \cdot 0.91)} \cdot 1.3 = 153.6 \ \ kN/m$$

D.1.2 Serviceability Limit State

Settlement should be calculated using conventional analyses.
Using reinforcement having the calculated design strength (the required long term design strength) normally ensures that excessive strain in the reinforcement will not occur.

E REINFORCED PILED EMBANKMENT

E.1 Example

A 2.5 m high reinforced piled embankment, see Figure E.1 is designed according to Chapter 6 for all parts except local stability that is designed according to Chapter 5.

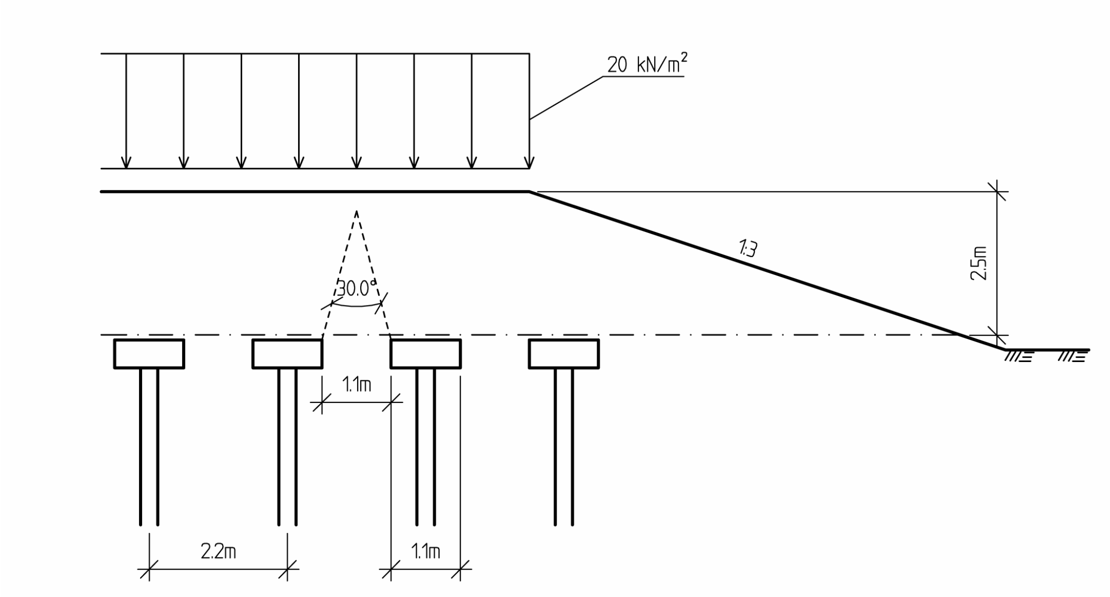


Figure E.1. Reinforced piled embankment - design example with vertical piles or inclined piles below the slope

E.1.1 Ultimate Limit State, Input Data:

Geometry and material data:	
Embankment height:	$H = 2.5$ m
Centre distance between piles (including tolerances)	c = 2.2 m
Pile cap width	b = 1.1 m
Distance between pile caps (including tolerances)	c-b = 1.1 m
Unit weight, fill material	$g = 20$ kN/m^3
Angle of friction	$f = 38°$

According to national regulations:	
Permanent load (dead load):	$q_G = 0$
Variable load (live load):	$q_Q = 20$ kN/m^2

PARTIAL FACTORS ACCORDING TO EUROCODE 7 (ENV 1997-1) (TABLE 2.1, CASE C):

Load factors:	
Applied to soil unit mass:	$\gamma_\gamma = 1$
Applied to permanent load:	$\gamma_G = 1$
Applied to variable load:	$\gamma_Q = 1.3$

Soil material factors:	
Applied to tan ϕ:	$\gamma_\phi = 1.25$

Reinforcement safety factors:

Applied to sliding of reinforcement:	$\gamma_s = 1.1$
Applied to pull-out resistance:	$\gamma_p = 1.3$

PARTIAL FACTORS GIVEN FROM TESTS ON ACTUAL REINFORCEMENT USED IN COMBINATION WITH ACTUAL FILL MATERIAL AND SUBSOIL (OR GIVEN BY MANUFACTURER/EMPIRICAL VALUES):

Interaction coefficient geotextile/fill material (crushed gravel):	$\alpha_1 = 1.0$
Interaction coefficient geotextile/fill material or soil:	$\alpha_2 = 1.0$ or 0.8

Installation damage effect:	$\eta_2 = 0.72$ ($\gamma_i = 1.4$)
Environmental impact on durability:	$\eta_3 = 0.91$ ($\gamma_d = 1.1$)
Creep factor:	$\eta_1 = 1.0$ ($\gamma_{cr} = 1.0$)

(in this case the strength should be given as long term strength)

The slope inclination is assumed to be 1:3,
i.e. slope length, $L_s = 3*H = 3*2.5 = 7.5\ m$.

E.1.1.1 Design of Horizontal Force

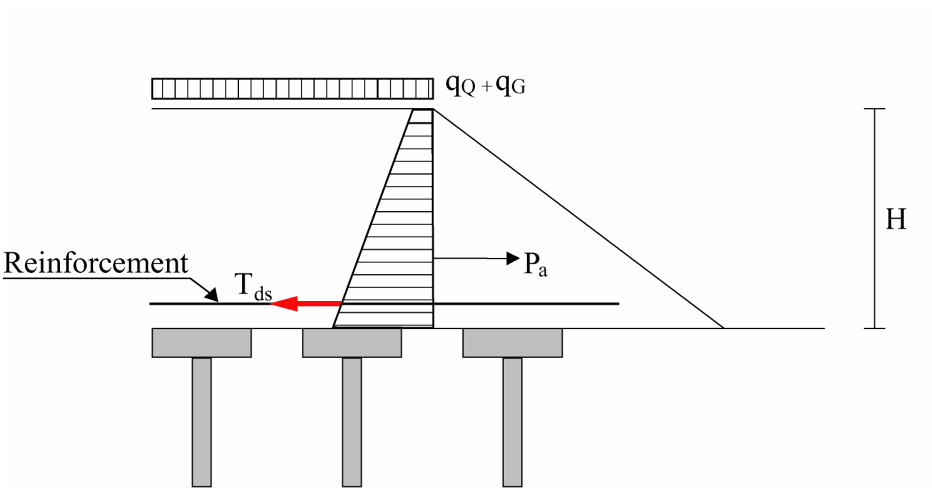


Figure E.2 Horizontal force in reinforcement using vertical piles beneath embankment slope.

If vertical piles are used beneath embankment slope, the tensile force in the reinforcement can be calculated as:

$$T_{ds} = P_{ad} = 0.5K_{ad}\left(g_d H + 2(g_Q q_Q + g_G q_G)\right)H \qquad (E.1)$$

$$f_d = \arctan\left(\frac{\tan f_k}{g_f}\right) = \arctan\left(\frac{\tan 38°}{1.25}\right) = 32° \qquad (E.2)$$

$$K_{ad} = \tan^2\left(45 - \frac{f_d}{2}\right) = \tan^2\left(45 - \frac{32°}{2}\right) = 0.31 \qquad (E.3)$$

$$T_{ds} = 0.5 \cdot 0.31\left(20 \cdot 2.5 + 2(20 \cdot 1.3 + 0)\right)2.5 = 39.5\ kN/m \qquad (E.4)$$

E.1.1.2 Design of Vertical Load Transfer

The initial strain in the reinforcement is in this case chosen to 6 %, maximal allowed strain level to ensure creep strain < 2 %. The surcharge load will not have any influence on the calculation model in this case for the restrictions of the height of the embankment given in the model.

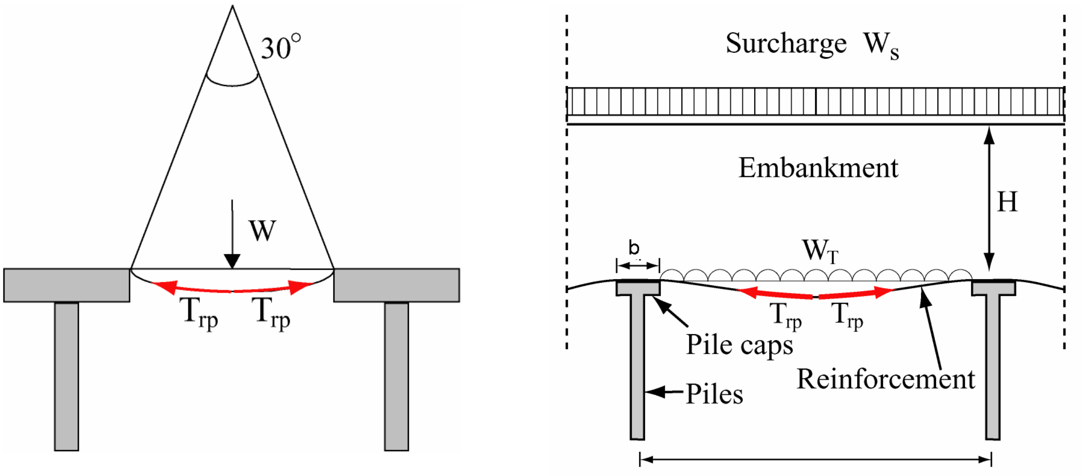


Figure E.3 The soil wedge which is carried by the reinforcement.

The weight of the soil wedge, W, according to Figure E.3 is:

$$W_{2Dd} = \frac{(c-b)^2}{4 \cdot \tan 15°} \cdot g_d = 0{,}93(c-b)^2 \cdot g_d \quad \text{kN/m} \qquad \text{(E.5)}$$

$$W_{2Dd} = \frac{(2.2-1.1)^2}{4 \cdot \tan 15°} \cdot 20 = 22.6 \quad \text{kN/m} \qquad \text{(E.6)}$$

The weight of the soil in three dimensions, W_{3D}, is calculated as follows:

$$W_{3Dd} = \frac{1+\frac{c}{b}}{2} \cdot W_{2Dd} = \frac{1+\frac{2.2}{1.1}}{2} \cdot 22.6 = 34 \ \text{kN/m} \qquad \text{(E.7)}$$

The displacement, d, is:

$$d = (c-b)\sqrt{\frac{3}{8}e} = 1.1\sqrt{\frac{3}{8} \cdot 0.06} = 0.16m \qquad \text{(E.8)}$$

The force in the reinforcement due to the vertical load in three dimensions, $T_{rp\ 3D}$ is calculated as:

$$T_{rp3D} = \frac{W_{3Dd}}{2} \cdot \sqrt{1+\frac{1}{6e}} = \frac{34}{2}\sqrt{1+\frac{1}{6 \cdot 0.06}} = 33 \ \text{kN/m} \qquad \text{(E.9)}$$

E.1.1.3 Design of Total Force

The total force, T_{tot}, in the reinforcement if vertical piles are chosen is:

$$T_{tot} = T_{ds} + T_{rp\,3D} = 39.5 + 33 = 72.5 \,\text{kN/m} \qquad \text{(E.10)}$$

The total force if inclined piles are chosen is 33 kN/m.
The strength of the seam has to be considered.

E.1.1.4 Design of the Bond Length of the Reinforcement

The reinforcement should achieve an adequate bond with the fill at the outer edge of the piled area and all vertical sections should be verified. For the necessary reinforcement length, illustrated in Figure E.5, on account of transverse sliding and pull-out force across the bank, the bond length$_b$, of the reinforcement can be determined according to the following calculations

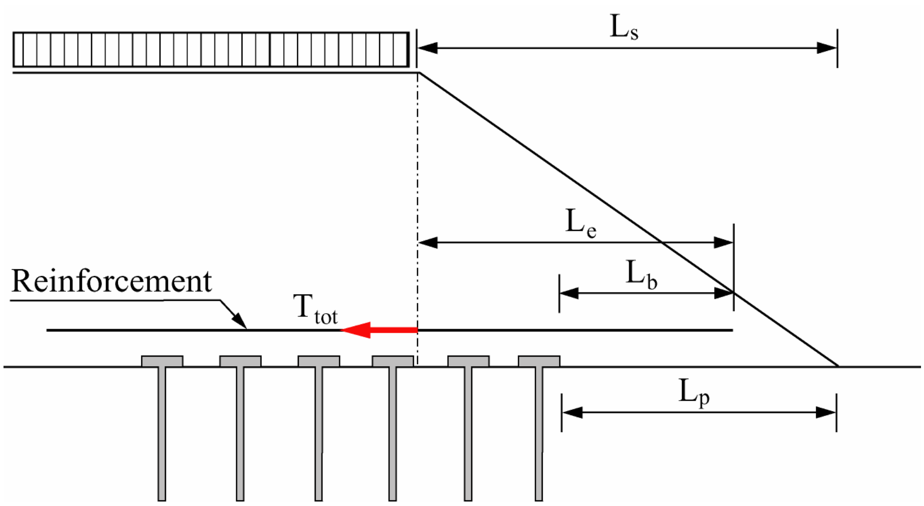


Figure E.4 The bond length according to transverse sliding across the bank and the pull-out length of the reinforcement

TRANSVERSE SLIDING

If vertical piles are chosen the bond length across the bank due to transverse sliding according to Figure E.4 can be calculated as:

$$L_e \geq \frac{T_{ds} \cdot \gamma_s}{\gamma_d h \left(\frac{\alpha \tan \phi}{\gamma_f} \right)} \qquad (\text{E.11})$$

$$L_e \geq \frac{0.5 K_{ad} H \left(\gamma_d H + 2\left(q_{Q_d} + q_{G\,d} \right) \right) \gamma_s}{\gamma_d h \alpha \tan \varphi_d}$$

$$L_e \geq \frac{0.5 \cdot 0.31 \cdot 2.5\left(20 \cdot 2.5 + 2\left(1.3 \cdot 20 + 0\right)\right) \cdot 1.1}{20 \cdot 1.25 \cdot \frac{1.0 \cdot \tan 38°}{1.25}} = 2.1 \ m$$

i.e. OK ($L_e < L_s$).

PULL-OUT FORCE

If vertical piles are chosen the bond length due to pull-out force across the embankment is calculated as:

$$L_b \geq \frac{(T_{rp3D} + T_{ds})g_{\ p}}{g_{\ d} h \left(\frac{a_1 \tan f_1}{g_{\ f}} + \frac{a_2 \tan f_2}{g_{\ f}} \right)} \qquad \text{(E.12)}$$

$$L_b \geq \frac{(33 + 39.5)1.3}{20 \cdot 1.25 \left(\frac{1.0 \tan 38}{1.25} + 0 \right)} = 6.0 \ \ m$$

In this case the friction below the reinforcement has not been taken into account and this will give a bond length on the safe side. If inclined piles are chosen L_b = 2.8 m.

Adjacent bond length along the length of the embankment could be calculated by the same equation where T_{ds}=0. Then the anchor length will be 2.8 m.

LOCAL STABILITY:

The local stability of the embankment side slope should be checked according to Chapter 5, (Equation 5.1):

$$\left(\frac{1}{n} = \right) \quad \frac{H}{L_s} \leq \frac{\tan f'_k}{g_{\ f}} \qquad \text{(E.13)}$$

$$\frac{1}{n} = \frac{1}{3} = 0.33 \ = \frac{\tan j\,'_k}{g_{\ f}} = \frac{\tan 38}{1.25} = 0.62 \qquad \text{i.e. ok.}$$

E.1.1.5 Design of Reinforcement

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

$$T_d = T_{cr} \cdot h_1 \cdot h_2 \cdot h_3 \qquad \text{(E.14)}$$

or

$$T_d = T_{cs} \cdot h_1 \cdot h_2 \cdot h_3 \qquad \text{(E.15)}$$

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

In Figure E.5 an example of short-term tensile test for a polyester type of product is shown. The short term breaking load is 226 kN/m. This value is not possible to use in the design. In Figure E.6 and Figure E.7 results from tensile creep test are given for the same type of product. T_{cr} the peak tensile creep rupture strength is the tensile failure of a specimen subject to tensile load, which is less than the tensile strength. In this case this has not occurred for the loads and times measured. T_{cs} the average tensile strength based on creep strain considerations is 113 kN/m (50 percent of load at failure) for an allowable strain of 6

% according to Figure E.6. This value can be compared with the load at failure of 226 kN/m. The creep strain after construction will be less than 2 % according to Figure E.7.

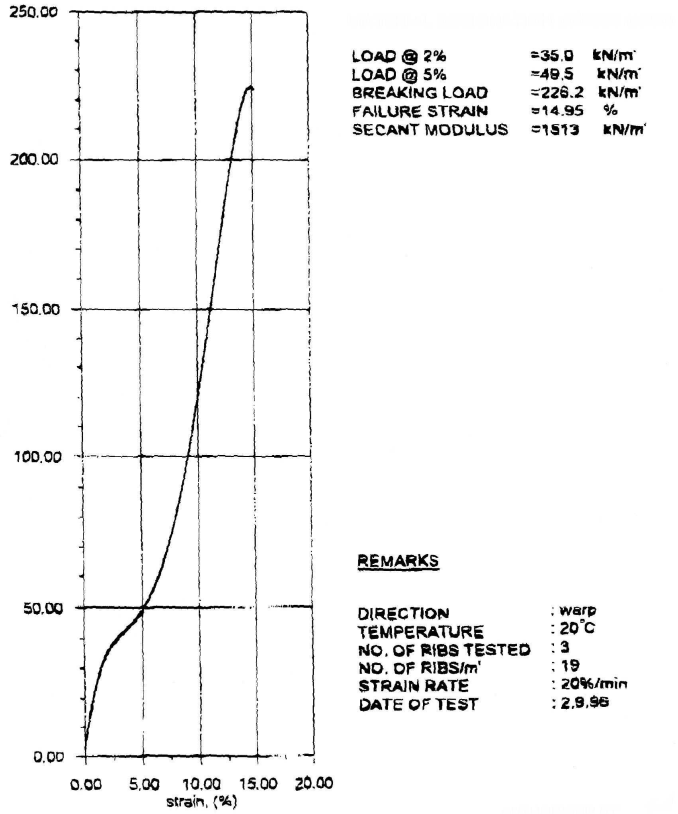


Figure E.5 Example of short term tensile strength for a polyester type product

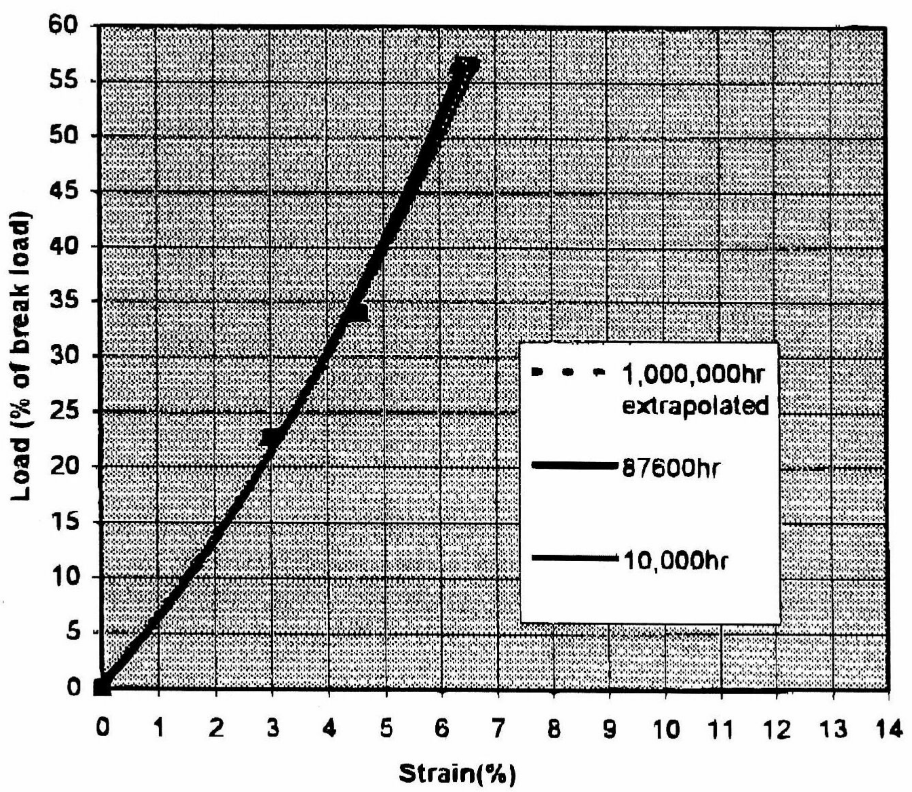


Figure E.6 Isochronous curve of tensile creep test for a polyester type product

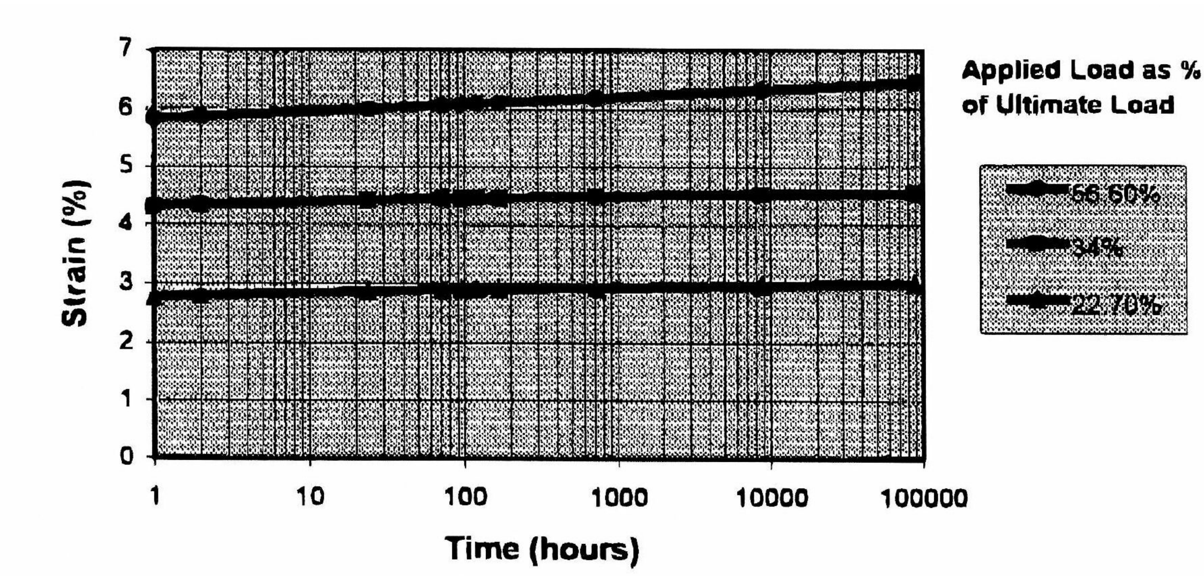


Figure E.7 Creep performance, tensile creep strain, for a polyester type product. ***Long term design strength***

The design strength of the reinforcement should be greater than total needed strength according to the calculations, $T_d > T_{tot}$.

For vertical piles, T_{cs}, needs to be:

$$T_d = T_{cs} \cdot h_1 \cdot h_2 \cdot h_3 \Rightarrow T_{cs} = \frac{T_d}{(h_1 \cdot h_2 \cdot h_3)} = \frac{72.5}{(1.0 \cdot 0.72 \cdot 0.91)} = 110 \;\; kN/m$$

For inclined piles , T_{cs}, needs to be:

$$T_{cs} = \frac{T_d}{(h_1 \cdot h_2 \cdot h_3)} = \frac{33}{(1.0 \cdot 0.72 \cdot 0.91)} = 50 \;\; kN/m \qquad \text{(E.16)}$$

For the case with vertical piles it is enough with one layer of this geogrid if the seam fulfills the same requirements or if it is placed with an overlap. In case of inclined piles another product will give a more economical solution.

The pile group capacity, pile group extent and overall stability has to be checked according to national regulations

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F SOIL NAILING

F.1 Example - Design of soil nailing for steep slopes and excavations

This chapter gives an example of how a steep slope/excavation reinforced with soil nails may be designed. All projects are different and therefore this example should only be used as a guideline. Other aspects than the one mentioned below might need to be considered for other projects.

F.1.1 Background

In this specific project a road needs to be broader to allow for increased traffic. Due to neighbouring houses the space is limited, a steep slope or a retaining structure needs to be constructed. It is decided that soil nailing should be used instead of a traditional retaining wall. The site is located in one of the Nordic countries and the road is one of the main roads in to the close by town.

The soil ranges from silty sand to sandy gravel. The groundwater is located below the preliminary wall. The wall is 6 meters high and about 75 meters long.

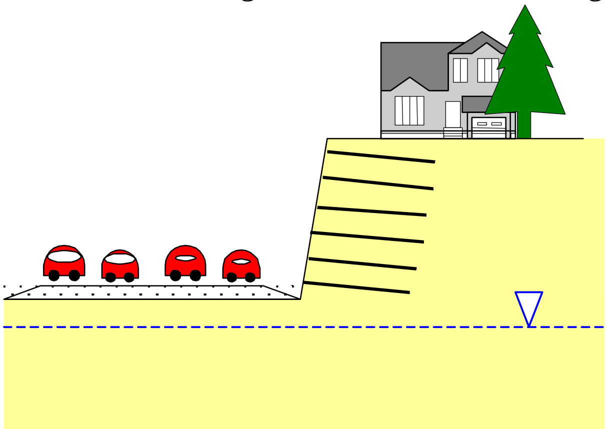


Figure F.1 Principal sketch of the site

F.1.2 Preliminary layout

F.1.2.1 Definition of the type of system

Before design of the soil nailing system it is important to define the purpose of the soil nailing. Identify specific problems that need to be considered.

This results in the following requirements for the soil nailing structure above.

- It should be a permanent structure where the aesthetic aspects shall be considered so that the structure becomes a natural part of the landscape.
- The structure shall be designed for the Nordic climate.
- The structure shall have a design life of 100 years.
- The groundwater level and the type of soil do not indicate any special problems for the execution of the project.

F.1.2.2 Empirical correlation

Before the actual design of the Soil Nailing wall, it is advisable to do a preliminary estimate of the layout of the final structure. Empirical correlation based on an article published by Bruce *et al*. (1986) is useful.

Assume that grouted nails should be used and use the empirical correlation in chapter 7.

NAIL LENGTH

According to the article the nail is about 0.5 to 0.8 times the height of the slope. If the height of the slope is 6 meter then the nail length is about 3 to 5 meters.

MOBILISED FRICTION

The size of the area there friction may be mobilised (nail length times the perimeter) should be about 0,3 to 0.6 times the surface area it reinforce (nail distance in horizontal direction times nails distance in vertical direction).

If a 4 meter long grouted nail with a diameter of 0.1 meter is used the distance between the nail should then be between $\sqrt{0.3\times 0.1 p \times 4} = 1.1$ to $\sqrt{0.6\times 0.1 p \times 4} = 0.87$ meter.

NAIL STRENGTH

The strength of the nail (the cross-sectional area of the nail that resists tension) should be about 0.0004 to 0.0008 times the surface area it reinforces.

Assume that the steel core have a diameter of 0.025 meter. Then the nail distance should be between $\sqrt{\frac{0.025^2}{0.0004}} = 1.25$ and $\sqrt{\frac{0.025^2}{0.0008}} = 0.88$.

ASSUMED NAIL LAYOUT

A preliminary assumption is that 4 meter long grouted nails should be used. The steel core should have a diameter of 0.025 meters and the nails distance less then 1.2 meters.

F.1.3 Stability analyses

To determine the actual amount of nails that is necessary to obtain a structure with satisfactory safety level a traditional stability analyses is performed.

F.1.3.1 Input values

GEOMETRY

The geometry used in the calculations should include the tolerances that are allowed during the execution. E.g. if the nail distance is 1.0 ± 0.1 meter then 1.1 meter should be used in the calculation since this is a more critical case than nail distance 0.9 m.

The following initial estimates are assumed;

- The nail distance is 1.2 meter, *i.e*. the actual nail distance is 1.1 meters, but there is a tolerance of ± 0.1 meters
- The nail length is 4 meters
- For the grouted nail the nail diameter is assumed to be 0.1 meters

MATERIAL PROPERTIES OF SOIL AND PARTIAL FACTORS

According to the soil investigation results the soil ranges from a silty sand to a sandy gravel, with fairly similar layering for the whole site. The following material properties are assumed;

The angle of shearing should according to section 3.5 be based on a cautious estimate from field and laboratory tests. An angle of shearing, ϕ, equal to 33° is therefore used. To obtain the design value the partial factor $\gamma_{m\phi}$ 1.25 is applied according to EC7 and case C.

$$\phi_d = \frac{\tan \phi_k}{\gamma_{m\phi}} = \frac{\tan 33}{1.25} \rightarrow \phi_d = 27.5 \qquad (\text{F.1})$$

The cohesion intercept is assumed to be 0.

The unit weight is assumed to be 20 kN/m^3 and the partial factor 1.0 is used in this case.

GROUNDWATER CONDITIONS

According to the field investigation results the groundwater level is below the toe of the slope. In the calculation a dry slope is therefore assumed. However, drainage system needs to be included in the final design to take care of percolating surface water.

PULLOUT CAPACITY AND PARTIAL FACTORS

Based on the charts in Clouterre (*c.f.* chapter 2) the pullout-resistance, q_s, for a grouted nail in sand varies between 0.05 to 0.1 MPa. The pullout force mobilised per meter nail is then;

$$\begin{aligned} \theta &= \pi D = \pi\ 0.1 = 0.31\ m^2/m \\ T &= \theta\ q_s \rightarrow 15.7\ kN/m\ to\ 31.4\ kN/m \end{aligned} \qquad (\text{F.2})$$

Assume that the characteristic average value based on field results is T_k = 35 kN/m. This has to be verified in the initial part of the execution. To obtain the design value a partial factor should be applied to account for the natural variation in the soil, γ_T and the nail. According to chapter 2 this values may be chosen as;

$$\gamma_T = \gamma_f \times \gamma_m = 1.25 \times 1.4 = 1.75 \qquad (\text{F.3})$$

To account for the number of test that have been performed a conversion factor is chosen according to Chapter 2. In this case 4 tests were performed and the average value used, consequently η 1/1.3 is applied.

$$T_d = \eta \frac{T_k}{\gamma_{mqs}} = \frac{1}{1.3}\frac{35}{1.75} = 15.4\ kN/m \qquad (\text{F.4})$$

LOADS[7]

For this specific case no variable or seismic loading is applied. A permanent action, q_G, 10 kPa from the close by houses should however be included. The load factor in this case γ_G is taken as 1.0 since it is a permanent action (*c.f.* chapter 3).

[7] *C.f.* appendix B.2 for applications in Sweden.

F.1.3.2 Stability analyses

The stability analyses may be performed with any program that can handle the extra force introduced by the soil nail. However, before using the program for an actual case, the program should be verified for a simple case that can be calculated by hand. Most programs may be used but have their limitations that the user needs to be aware of.

In chapter 7 the equations for the stability analysis is shown. The design value of each parameter is used and the aim is to find a layout of the nails that gives F=1.0. In a traditional stability analysis there are requirements on both the drained, undrained and combined safety. In this specific case only the drained parameters, the shearing angle describe the soil frictional resistance, consequently a drained analyses is performed.

The following values are used as an initial estimate:

Table F.1 Input values for the stability analysis

f_d	27.5°
g	20 kN/m^3
Nail distance	1.2 meter
Nail length	4 meter
T_d	15.4 kN/m divided by nail distance 1.2 gives 12.8 kN/m per meters of the slope.

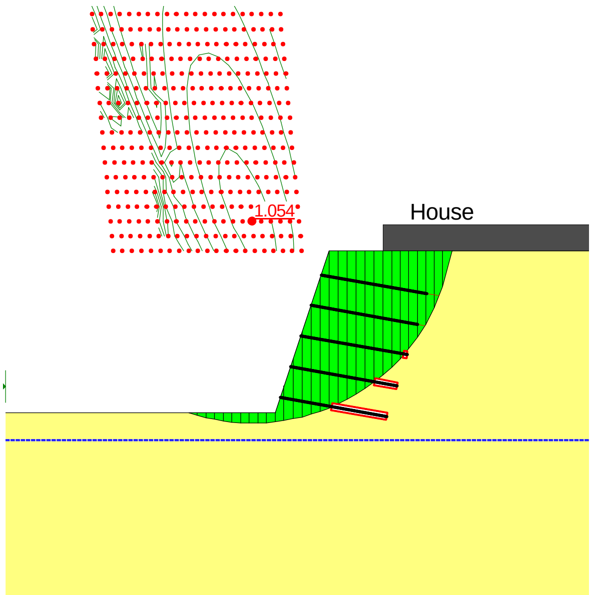


Figure F.2 Initial calculation

The initial estimate results in a factor of safety equal to 1.05. In this case it is not considered necessary to do further calculations to optimise the layout.

F.1.4 Verification of chosen soil nailing system

Before the layout is finally chosen additional failure modes need to be considered: failure of a single nail and long-term failure due to durability.

F.1.4.1 Failure of the nail

In the stability analysis only the frictional resistance that may be mobilised along the nail limits the pullout capacity. However, additional failure modes need to be considered to ensure that the nail will mobilise the necessary force.

The failure modes that need to be considered are;
1. Pullout failure due to failure between the nail and the soil (tension) in active and passive zone.
2. Bearing failure in the soil below the nail
3. Failure of the steel in the nail due to tension
4. Failure of the steel in the nail due to bending/shearing

The French multi-criteria method handles this in a systematic way.

For each nail and possible failure surface the above criteria should be checked. In this example the use of the multi-criteria method is shown only for the critical failure surface.

CRITERION 1 – PULLOUT

For each nail determine the maximum design value of the pullout capacity that can be mobilised in the active and resisting zone. In the specific case it is assumed that the nail head do not contribute, consequently failure will occur when the smaller of the two values are exceeded. The pullout capacity is determined as;

$$P = T_d \times L \tag{F.5}$$

The design value for the pullout capacity has previously been determined to 15.4 kN/m. (The calculation is performed for each nail and consequently the pullout force should not be reduced with the nail distance, as in the slope stability calculation.). The results in Table F.2 indicate that for nail 1, it is not possible to mobilise the full pullout capacity in the active zone unless the nail plate is designed to take some force.

Table F.2 Maximum allowable pullout force in the nails for the critical failure surface

	length (m)		pullout force kN/m	
Nail	active	resisting	active	resisting
5	4.0	0.0	61.6	<u>0.0</u>
4	4.0	0.0	61.6	<u>0.0</u>
3	3.7	0.3	57	<u>4.6</u>
2	3.0	1.0	46.2	<u>15.4</u>
1	1.8	2.2	<u>27.7</u>	33.8

___ = indicates the limiting values to be used in the calculation

CRITERION 2 – BEARING FAILURE IN THE SOIL BELOW THE NAIL

The ultimate lateral pressure of the soil limits the pressure from the nail on the soil. The maximum bearing capacity is obtained either in a single point (point of maximum moment) or along a distance defined by the two points of maximum shearing (the transfer length, l_0). The following criterion should be fulfilled;

$$R_N \leq \frac{D}{2} l_0 p_u$$
$$l_0 = \sqrt[4]{\frac{4EI}{k_h D}} = 0.24\ m \qquad \text{(F.6)}$$

k_h is the horizontal modulus of subgrade reaction, in this case, 180-300 MNm^3.
EI is the nail stiffness that for a circular steelcore with diamter 0.025 m is 4025 kNm^2. (only calculated for the steel)
pu is the ultimate lateral pressure in the soil, in this case, 500 to 800 kPa, depending on depth below the surface.
D nail diameter including grout

This gives that the maximum shearing force in the nail is less than;

$$R_N \leq \frac{0.025}{2} 0.24 \times 650 kPa = 11\ kN \qquad \text{(F.7)}$$

CRITERION 3 – FAILURE IN THE STEEL DUE TO TENSION

The combined effect of shearing and tension in the nail give the following criterion.

$$\left(\frac{T}{T_{\text{limit}}}\right)^2 + \left(\frac{R_N}{R_{N\_\text{limit}}}\right)^2 \geq 1 \qquad \text{(F.8)}$$

CRITERIA 4 – FAILURE IN THE STEEL DUE TO BENDING/SHEARING

The following criteria should be fulfilled

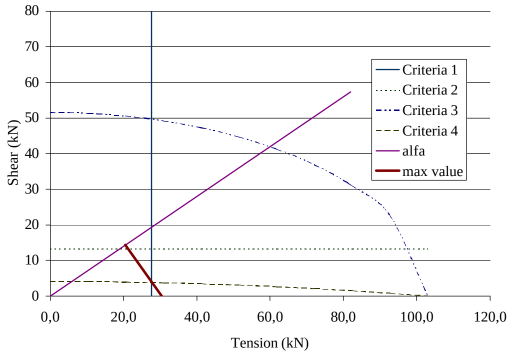
$$R_N \leq b\left(\frac{M_0}{l_0}\right) \times \left(1 - \left(\frac{T}{T_{\text{limit}}}\right)^2\right) + cDl_0 p_u \qquad \text{(F.9)}$$

b and c is constant with the values 1.62 and 0.24 respectively.
M_0 is the plastic moment of the steel.

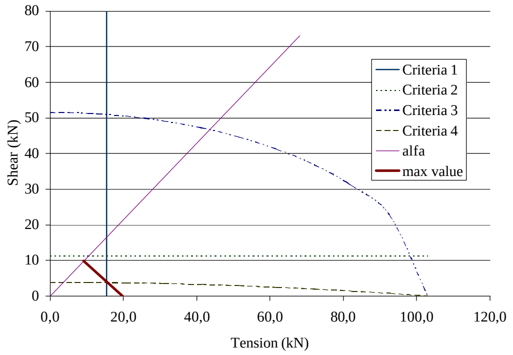
ALLOWABLE WORKING LIMITS FOR THE NAIL

The above criteria can be shown in a shear vs. tension graph for each nail. The allowable nail force is then determined with respect to the angle between the nail and the failure surface. In Figure F.3 the results from the analyses are presented. For all nails the tension force is limited by the pullout capacity either in the active or passive zone. For the nails in the bottom line the pullout capacity is smaller than the one used in the above stability analyses (the pullout capacity in the passive zone is limiting). A re-analysis of the problem allowing for a smaller value results in a factor of safety close to1.0.

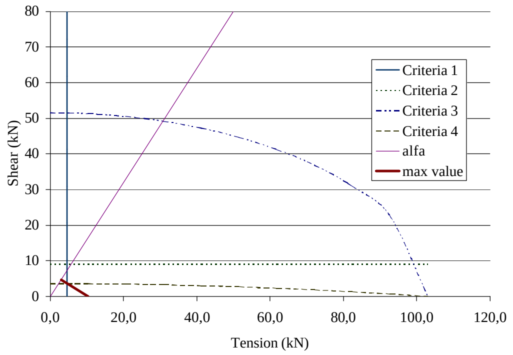
The analysis indicate that no alteration of the layout is necessary.



Nail 1, α 35°
Tension force limited by pullout to 27.7 kN that gives 23 kN per length meters of the slope.



Nail 2, α 47°
Tension force limited by pullout to 15.4 kN that gives 13 kN per length meters of the slope



Nail3, α 58°
Tension force limited by pullout to 4.6 kN that gives 3.8 kN per length meter of the slope

It is not necessary to make multi-criteria analyses for nail 4 and 5, since these nail do not mobilise any force for the critical failure surface.

For anoher failure surface it might be necessary to look also at nail 4 and 5. If analyses should be performed for other failure surfaces than the critical is determined for each caase.

Figure F.3 Results from Multi Criteria, nails numbered from bottom up.

F.1.4.2 Durability

The long term performance of the nail needs to be considered. In chapter 7 a system for how to choose the necessary corrosion protection system is given.

STEP 1- PRELIMINARY ESTIMATE OF THE CORROSION POTENTIAL OF THE ENVIRONMENT

An initial estimate of the corrosion potential is made based on table 7.5 and 7.6. The soil is a silty sand to a sandy gravel, and according to table 7.5 this gives a low to very low corrosion potential. Assume 2 points. From Table 7.6 the following factors applies to the site.

Factor	Additional points
The groundwater level is lower than 2.5 meters below the ground surface	± 0
Dry and well drained material	-2
Distance to road that is salted during the winter period is less then 25 m	+4
Agriculture area where fertiliser is used	+2
Σ	4

A total of 6 points is obtained for the site. A more detailed classification of the soil is necessary according to Chapter 7.

STEP 2- DETERMINATION OF ENVIRONMENTAL CLASS BASED ON MORE DETAILED SOIL INVESTIGATION

Table 7.7 in chapter 7 is used to make a more detailed classification

Criterion	Explanation	Index
	Clay, silt, moraine (normal)	1
Soil	Sand, gravel, (porous, permeable)	0
Resistivety	50 < p	0
Moisture - salt	Moist sample of soil above groundwater table (w > 20 %)	2
pH	Basic environment pH > 6	0
Vertical layering	Homogenous soil	0
Other factors	Water with salt from road	8
	Σ	11

According to Chapter 7 that due to the contact with a road that during the wintertime is salted the site need to be considered as environmental class III, with high potential of corrosion.

STEP 3 - DETERMINATION OF ENVIRONMENTAL CLASS CONSIDERING OTHER ASPECTS

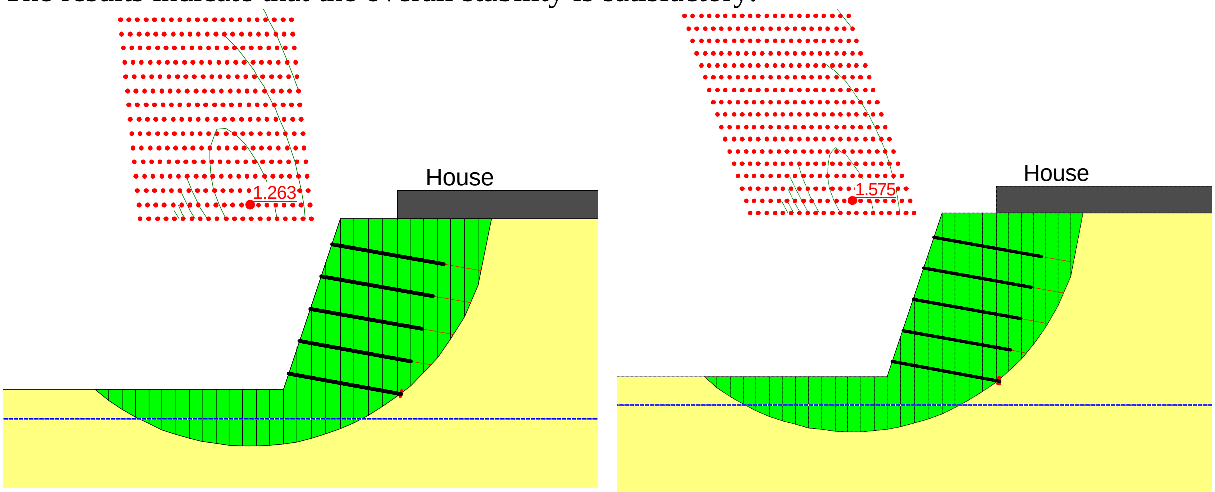
None of the aspects listed in step 3 is applicable for the site.

STEP 4 - CHOICE OF CORROSION PROTECTION SYSTEM

Table 7.10 indicates that a special investigation should be performed. In these case it seems reasonable to determine if the influence of the road-salt is that severe as the preliminary estimate indicate. However, as a minimum a nail with grout combined with either sacrificial barrier or plastic barrier is chosen.

F.1.5 Overall stability analyses

The overall stability of the soil nailed structure need to be analysed. For comparison both a traditional analysis using characteristic values and a partial factor analysis is performed. The results indicate that the overall stability is satisfactory.



Partial factor, F = 1.26

Global factor of safety, F = 1.58

Figure F.4 Overall stability of the soil nailed structure

In Chapter 7 three other failure mode is mentioned;

- Sliding due to the active pressure from the soil behind the block acting on the reinforced block
- Bearing failure (the weight of the reinforced block and the lateral earth pressure acting on its back might cause a foundation bearing failure)
- Overturning of the reinforced block

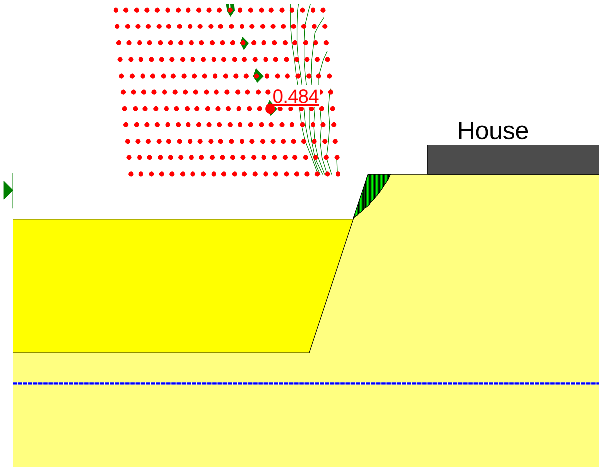
In the specific case it was determined that these failure modes were not limiting for the above structure.

F.1.6 Stability analyses of each excavation stage

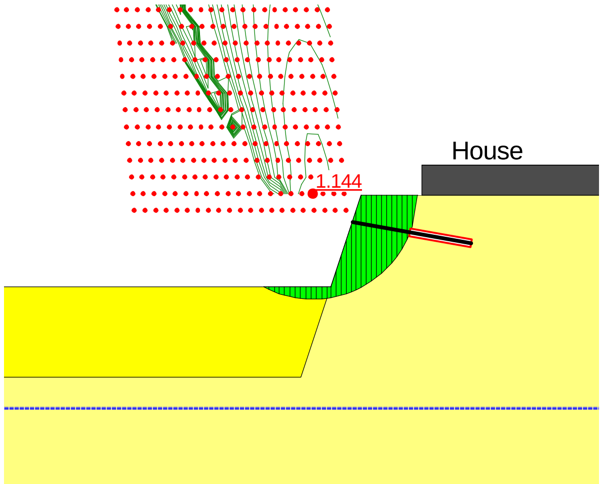
Each excavation step during the execution needs to be checked to make sure that a satisfactory safety is obtained.

Four excavation steps are performed, each with a height of 1.5 meters.

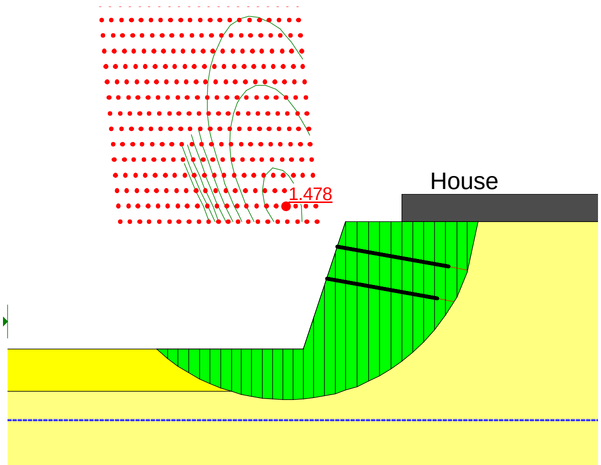
Stage 1 – before installation of first nail.
F<1.0



Stage 2 – before installation of second nail.
F>1.0



Stage 3 – before installation of third nail
F > 1.0



Stage 4 – before installation of last row of nails. F < 1.0

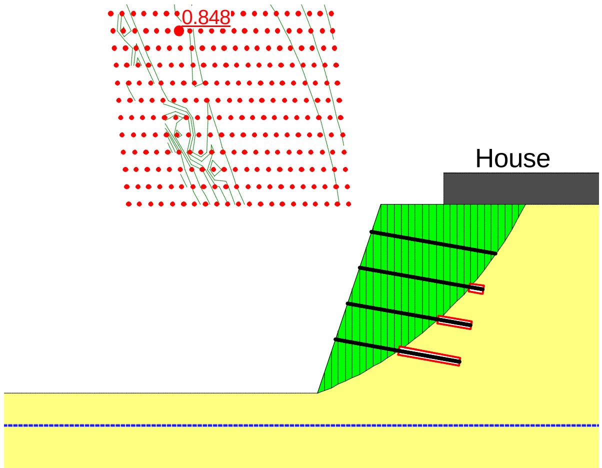


Figure F.5 Stability analysies for each excavation stage

The safety of the unreinforced slope in stage 1 is low, and therefore the protective action needs to be taken for the execution of the excavation. The protective actions could either be by excavation in sections and applying the facing immediately after excavation or by installation of the nail through a protective beam. Stage 2 and 3 have satisfactory safety, but for step 4 it could also be necessary to take protective action.

For each stage a multi-criteria analysis shall be performed to verify that the nails have sufficient capacity. For the stages presented in Figure F.5 the multi-critera analysis gives that the pullout capacity in the active zone is limiting for the maximum tension force that the nails may mobilise for stage 1, 3 and 4. For stage 2 the pullout capacity in the resisting zone is limiting.

F.1.7 Facing

F.1.7.1 Constructional aspects

For a steep slope facing is necessary. The purpose of the facing is to stabilise the soil between the nails, to avoid local failure. In this case shotcrete facing has been chosen. The facing should be designed in accordance with regular concrete design guidelines considering the following aspects:

- The facing should withstand the bending moment caused by the earth pressure between the nails. For long-term facing both ultimate limit and serviceability limit state should be considered. For serviceability limit state the cracking should be taken into account.
- In the design it should also be accounted for punching mode at the nail head

The nail head needs to be designed to take the additional load without punching into the soil. if the design of the nail length has been made in such a way that the mobilised pullout capacity in the active zone is greater then the force that could be mobilised in the passive zone. In this case the pullout force has been chosen as the smaller of the two values.

The climate influences the choice of facing. The soil nailing site is located in an area there you could expect frost. Consequently it is necessary to consider the possibility of frost-thaw which could result in extra forces on the nail.

In this specific case a combined frost and drainage plate made of XPS is placed against the soil and covered with spry concrete.

F.1.7.2 Esthetical aspects

A shotcrete structure is not always the most esthetical facing. For this specific case a traditional stone wall was built in front of the soil nailing facing to make sure that the structure adapt to the environment.

F.1.8 Drainage

The designed facing is fairly thin and consequently it is very important that no water pressure is built up behind the wall. The XPS plate with drainage channels is placed next to the soil and to make sure that the water is transferred from the back to the front, sub-surface drainage through the facing is installed.

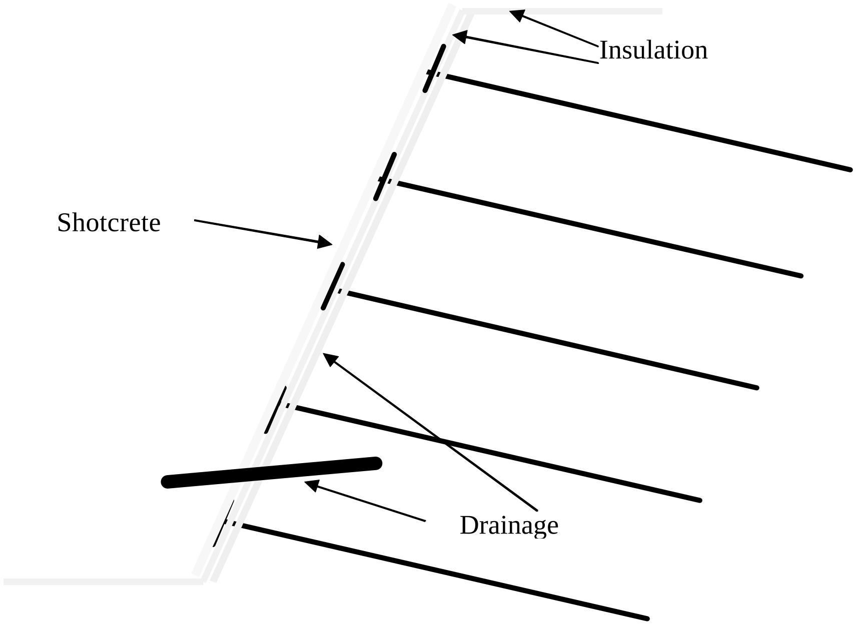


Figure F.6 Facing and drainage

F.1.9 Serviceability state

An estimate of the deformation that could be expected during the design life is made based on the empirical correlation given in Chapter 7. The movement of the top of the slope is in sand about 2H/1000, which in this case is 12 mm. The movement will influence an area about 5 meters behind the wall according to the same empirical correlation.

$$l = H(1 - \tan h)k = 6(1 - \tan(18.4))1.25 = 5 \qquad \text{(F.10)}$$

F.1.10 Acknowledgement

The input values for this example are partly based on an soil nailing project located in Lillehammer, Norway. The project is further described in Statens Vegvesen, laboratorieserien, rapport 56

F.2 Example - Design of soil nailing for natural slope

Soil Nailing is often used to increase the factor of safety for a natural slope. In the example in this chapter the main differences between the design of an excavated soil nailing structure and a natural slope reinforced with soil nails are shown.

F.2.1 Background

In this project a road is located at the edge of a steep slope, about 17 meters high, above a major river. The slope is about 1:1.25. Cracks have been observed in the road and it has been decided that the safety of the road needs to be increased. Soil Nailing is considered as one alternative.

The area is fluvio-glacial-deposit with silty sand at the top over a gravelly sand/morain. The depth to rock varies between 8 to 14 meters. The groundwater level is assumed to be located deep below the ground surface.

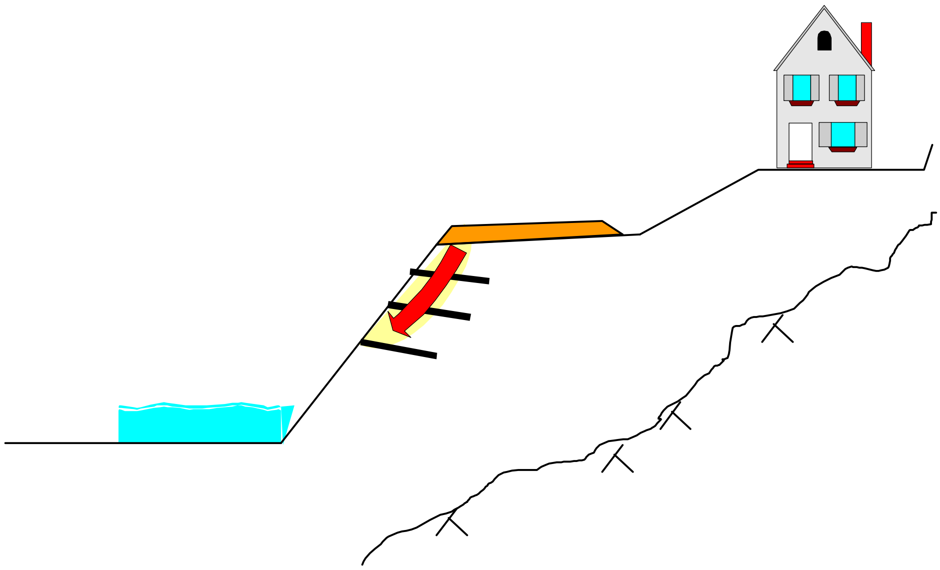


Figure F.7 Principal sketch of the site

F.2.2 Preliminary layout

F.2.2.1 Definition of the type of system

In this case the main purpose of the soil nailing is to increase the safety for the local failure surface.

This results in the following requirements for the soil nailing structure above.

- It should be a permanent structure with a design life of 40 years.
- The facing should if possible adapt to the nature.
- The structure shall be designed for the Nordic climate.
- Special consideration needs to be taken regarding the working procedure. Traffic on the road and limited accessibility to the site.

F.2.2.2 Empirical correlation

The empirical correlation presented by Bruce *et al* (1986) may be used for a preliminary estimate of the layout slope, even though the correlation is mainly based on results from excavations. Assume that grouted nails should be used and use the empirical correlation in chapter 7.

NAIL LENGTH

The height of the slope is about. 17 meters and according to the correlation the length of the nails should be about 0.5 to 0.8 *H*, 8.5 to 13.6 m.

MOBILISED FRICTION

The size of the area where friction may be mobilised (nail length times the perimeter) should be about 0.3 to 0.6 times the surface area it reinforce (nail distance in horizontal direction times nails distance in vertical direction).

If an 8-meter long grouted nail with a diameter of 0.1 meter is used the distance between the nail should then be between $\sqrt{0.3 \times 0.1 p \times 8} = 0.87$ to $\sqrt{0.6 \times 0.1 p \times 8} = 1.22$ meter.

NAIL STRENGTH

The strength of the nail (the cross-sectional area of the nail that resist tension) should be about 0.0004 to 0.0008 times the surface area it reinforces.

Assume that the steel core has a diameter of 0.025 meter. Then the nail distance should be between $\sqrt{\frac{0.025^2}{0.0004}} = 1.25$ and $\sqrt{\frac{0.025^2}{0.0008}} = 0.88$.

ASSUMED NAIL LAYOUT

The above empirical correlation indicates that four nails with a length of 8 meter and a centre distance of 1.1 meter would be sufficient for reinforcing the road. However, considering the site and the difficulties with installation of the nails, an attempt is made to minimise the number of nails. The nail distance is increased to 1.5 meters. An alternative could be to increase the length of the nails in the top of the slope but it might result in local failure at the bottom of the slope.

F.2.3 Stability analysis

To determine the actual amount of nails that is necessary to obtain a structure with satisfactory safety level a traditional stability analysis is performed.

F.2.3.1 Input values

GEOMETRY

The geometry used in the calculations should include the tolerances that are allowed during the execution. E.g. if the nail distance is 1.0 ± 0.1 meter then 1.1 meter should be used in the calculation since this is a more critical case then nail distance 0.9 m.

The following initial estimates are assumed;

- The nail distance 1.5 meters, *i.e.* the actual nail distance is 1.4 meter, but there is a tolerance of ± 0.1 meters
- The nail length is 8 meters
- For the grouted nail the nail diameter is assumed to be 0.1 meters

MATERIAL PROPERTIES OF SOIL AND PARTIAL FACTORS

According to the soil investigation results three different layers may be identified at the site; silty sand, sand and moraine. The characteristic value for the different layers and the corresponding design values is found in Table F.3. To obtain the design value the partial factor $\gamma_{m\phi}$ 1.25 is applied according to EC7 and case C.

$$f_d = \frac{\tan f_k}{g_{mf}} \rightarrow f_d \qquad (\text{F.11})$$

Table F.3 Input values for the soil

	ϕ_k	ϕ_d
Silty sand	36	30.2
Dense sand	38	32
Moraine	39	33

The cohesion intercept is assumed to be 0.

The unit weight is assumed to be 18 kN/m^3 and the partial factor 1.0 is used in this case.

GROUNDWATER CONDITIONS

The ground water level is assumed to be located fairly deep and will not influence the stability analyses of the local failure surface.

PULLOUT CAPACITY AND PARTIAL FACTORS

Based on the charts in Clouterre (*c.f.* chapter 2) the pullout-resistance, q_s, for a grouted nail in sand varies between 0.05 to 0.1 MPa. The pullout force mobilised per meter nail is then;

$$q = pD = p\ 0.1 = 0.31\, m^2 / m$$
$$T = q\ q_s \rightarrow 15.7\ kN/m\ to\ 31.4\ kN/m \qquad (\text{F.12})$$

Assume that the characteristic average value based on field results is T_k = 35 kN/m. This has to be verified in the initial part of the execution. To obtain the design value a partial factor should be applied to account for the natural variation in the soil, γ_T and the nail. According to chapter 2 these values may be chosen as;

$$g_T = g_f \times g_m = 1.25 \times 1.4 = 1.75 \qquad (\text{F.13})$$

To account for the number of tests that have been performed a conversion factor is chosen according to chapter 2. In this case 4 tests were performed and the average value used, consequently η 1/1.3 is applied.

$$T_d = h \frac{T_k}{g_{mqs}} = \frac{1}{1.3} \frac{35}{1.75} = 15.4\ kN/m \qquad (\text{F.14})$$

LOADS

In this case there is both a permanent action from the house and a variable action from the traffic. The partial factors are taken as 1.0 and 1.3 respectively. The following loads are used in the calculation.

Permanent loading $q_G \times g_G = 10 \times 1.0 = 10\ kPa$

Variable loading $q_Q \times g_Q = 20 \times 1.3 = 26 kPa \qquad (\text{F.15})$

F.2.3.2 Stability analyses

Table F.4 Input values for the stability analysis

f_d	30.2° / 32° / 33°
g	18 kN/m^3
Nail distance	1.6 meter
Nail length	8 meter
T_d	15.4 kN/m divided by nail distance 1.5 gives 10.3 kN/m for each meters in the length direction.

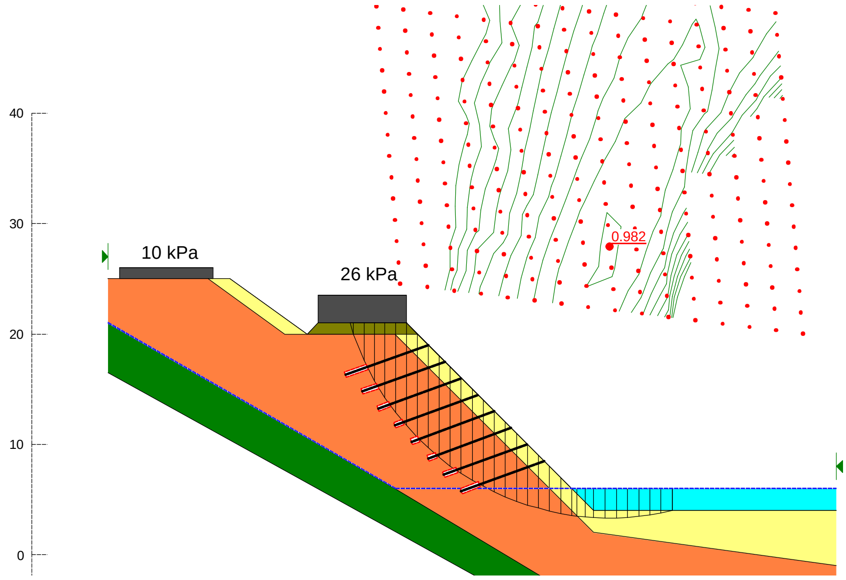


Figure F.8 Initial calculation, F = 0.98

It is difficult to install the nails in the lower part of the slope. An attempt is therefore made to reduce the number of rows by increasing the nail length. Four rows with 12-meter long nails are considered.

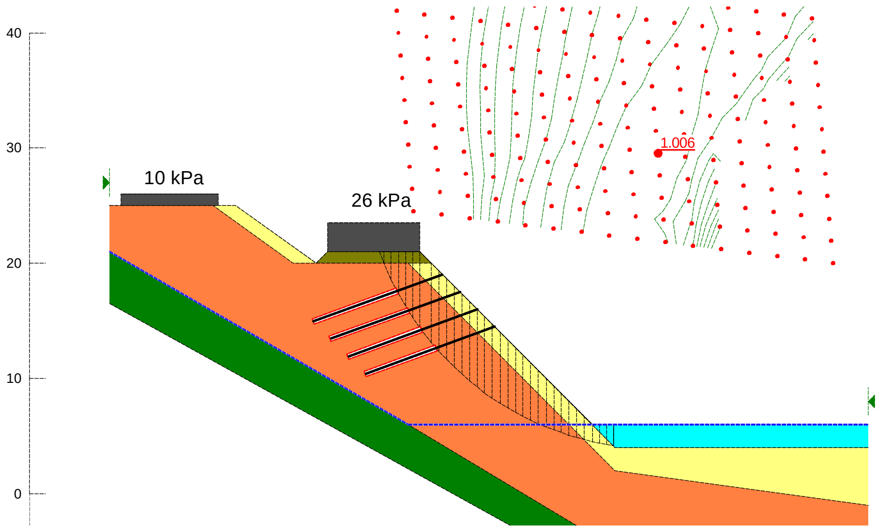


Figure F.9 Modified layout of the nails.

This layout is due to practical reasons preferable, and will be used.

F.2.4 Verification of chosen soil nailing system

As for the example 1 (the soil nailed excavated slope) additional failure modes for the nail need to be considered: failure of a single nail and long-term failure due to durability, before the final lay-out is chosen.

F.2.4.1 Failure of the nail

In the stability analyses only the frictional resistance that may be mobilised along the nail limits the pullout capacity. However, additional failure modes need to be considered to ensure that the nail will mobilise the necessary force.

The failure modes that need to be considered are;

1. Pullout failure due to failure between the nail and the soil (tension) in active and passive zone.
2. Bearing failure in the soil below the nail
3. Failure of the steel in the nail due to tension
4. Failure of the steel in the nail due to bending/shearing

The French multi-criteria method handles this in a systematic way. For each nail the above criteria should be checked.

CRITERION 1 – PULLOUT

For each nail determine the maximum design value of the pullout capacity that can be mobilised in the active and resisting zone. In the specific case it is assumed that the nail head do not contribute, consequently failure will occur then the smaller of the two values are exceeded. The pullout capacity is determined as;

$$P = T_d \times L \qquad (F.16)$$

The design value for the pullout capacity has previous been determined to 15.4 kN/m. (The calculation is performed for each nail and consequently the pullout force should not be reduced with the nail distance, as in the slope stability calculation.). The results in Table F.5 indicate that none of the nails is capable of mobilising the full pullout capacity in the active zone unless the nail plate is designed to take some force.

Table F.5 Maximum allowable pullout force in the nails for the cirtical failure surface. Case with four nails.

	length (m)		pullout force kN/m	
Nail	resisting	active	resisting	active
1	7.8	4.2	120,1	64,7
2	7.2	4.8	110,9	73,9
3	6.8	5.2	104,7	80,1
4	6.6	5.4	101,6	83,2

___ = Indicates the limiting values to be used in the calculation

CRITERION 2 – BEARING FAILURE IN THE SOIL BELOW THE NAIL

The same equations as in the previous example for soil nailed wall is used.

$$R_N \le \frac{D}{2} l_0 p_u$$
$$l_0 = \sqrt[4]{\frac{4EI}{k_h D}} = 0.2\ m \qquad (F.17)$$

k_h is the horizontal modulus of subgrade reaction, in this case, 180-300 MNm^3.
EI is the nail stiffness that for a circular steelcore with diamter 0.025 m is 4025 kNm^2. (only calculated for the steel)
pu is the ultimate lateral pressure in the soil, in this case, 500 to 800 kPa, depending on depth below the surface.
D nail diameter including grout

This gives that the maximum shearing force in the nail should be less than;

$$R_N \leq \frac{0.025}{2} 0.2 \times 800kPa = 12\ kN \qquad (\text{F.18})$$

CRITERION 3 – FAILURE IN THE STEEL DUE TO TENSION

The combined effect of shearing and tension in the nail should fulfil the following criteria.

$$\left(\frac{T}{T_{\text{limit}}}\right)^2 + \left(\frac{R_N}{R_{N\_\text{limit}}}\right)^2 \geq 1 \qquad (\text{F.19})$$

CRITERION 4 – FAILURE IN THE STEEL DUE TO BENDING/SHEARING

The following criteria should be fulfilled

$$R_N \leq b\left(\frac{M_0}{l_0}\right) \times \left(1 - (\frac{T}{T_{\text{limit}}})^2\right) + cDl_0\, p_u \qquad (\text{F.20})$$

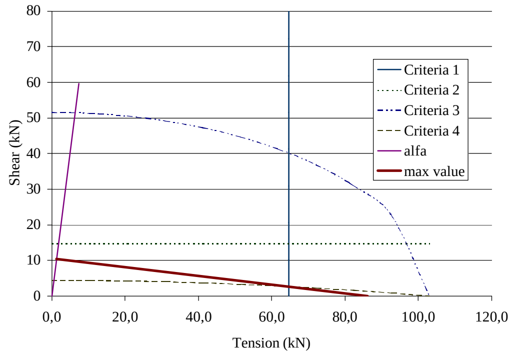
b and c is constant with the values 1.62 and 0.24 respectively.
M_0 is the plastic moment of the steel.

ALLOWABLE WORKING LIMITS FOR THE NAIL

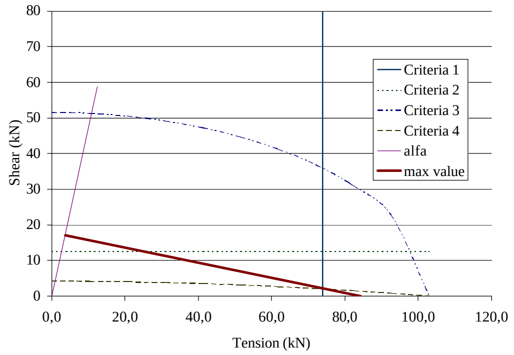
The above criteria can be shown in a shear vs. tension graph for each nail. The allowable nail force is then determined with respect to the angle between the nail and the failure surface. In Figure F.10 the results from the analysis is presented.

For all nails the tension force is limited by the pullout capacity in the active zone. The pullout capacity is smaller than the one used in the above stability analyses and consequently a reanalyse is necessary. The calculation results in a factor of safety about 0.93 and is not considered satisfactory.

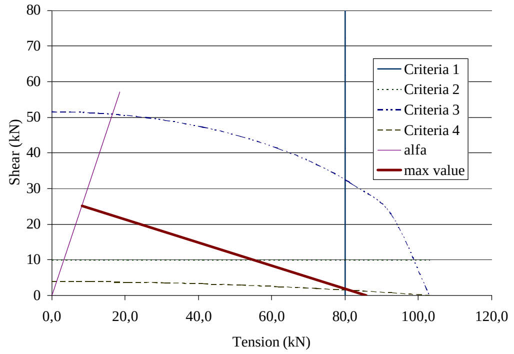
The layout either has to be changed or the plates need to be designed to contribute with some force. In this case it is decided that the plates should be designed to take the necessary force so that the pullout capacity in the resisting zone may be mobilised. The negative consequence of this decision is mainly esthetical, but since the alternative with additional nail rows will result in difficulties with the execution it is preferable.



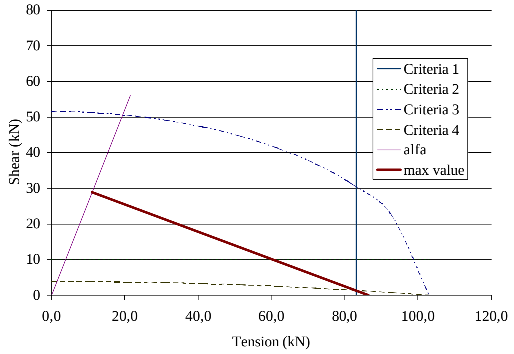
Nail 1, α 83°
Tension force limited by pullout to 64.7 kN that gives 43 kN per length meter of the slope.



Nail 2, α 78°
Tension force limited by pullout to 73.9 kN that gives 49 kN per length meter of the slope.



Nail 3, α 72°
Tension force limited by pullout to 80.1 kN that gives 53 kN per length meter of the slope.



Nail 4, α 69°
Tension force limited by pullout to 83.,2 kN that gives 55 kN per length meter of the slope.

Figure F.10 Results from Multi Criteria, nails numbered from top down.

F.2.4.2 Durability

The corrosion potential at the site is determined according to the suggested methodology in chapter 7.

STEP 1- PRELIMINARY ESTIMATE OF THE CORROSION POTENTIAL OF THE ENVIRONMENT

An initial estimate of the corrosion potential is made based on table 7.5 and 7.6. The soil range from a sand to a moraine, and according to table 7.5 this gives a low to very low corrosion potential. Assume 2 points. From Table 7.6 the following factor applies to the site;

Factor		Additional points
The groundwater level is lower than 2.5 meter below the ground surface		± 0
Distance to road that is salted during the winter period is less thaan 25 m		+4
	Σ	4

A total of 6 points is obtained for the site. A more detailed classification of the soil is necessary according to chapter 7.

STEP 2- DETERMINATION OF ENVIRONMENTAL CLASS BASED ON MORE DETAILED SOIL

Table 7.7 in chapter 7 is used to make a more detailed classification

Table F.6 Corrosion potential for the site, step 2

Criterion	Explanation	Index
Type of soil	- Clay, silt, moraine (normal)	1
	- Sand, gravel, (porous, permeable)	0
Resistivitet	50 < p	0
Moisture - salt	Moist sample of soil above groundwater table (w > 20 %)	2
pH	Basic environment pH > 6	0
Vertical layering	Soil profile with different layers	1
other factors	Water with salt from road	8
	Σ	12

Based on the calculated index the site is classified as an environment with high potential of corrosion. Environment class III.

STEP 3 - DETERMINATION OF ENVIRONMENTAL CLASS CONSIDERING OTHER ASPECTS

None of the mentioned aspects are relevant for the site.

STEP 4 - CHOICE OF CORROSION PROTECTION SYSTEM

According to the suggestion in step 4, a special investigation should be considered before the choice of a relevant corrosion protection system. The question is if the salt from the road will have an impact on the soil nails or not. On the other hand the consequence of a failure is in this case severe and therefore the protection of the nail should be extensive. A double corrosion protection system was chosen.

F.2.5 Overall stability analysis

The overall stability analyses has been performed as one part of the design of the soil nails.

F.2.6 Stability analyses of each excavation stage

For a natural slope no excavation will be performed. However, each installation stage should be considered to make sure that a reasonable safety level is obtained for each execution stage.

F.2.7 Facing

F.2.7.1 Constructional aspects

A geonet in combination with load transferring plates are chosen as facing for the slope. The geonet is designed so that it will have the required strength to resist the active earth pressure of the soil between the nails at a reasonable deformation

The required bearing capacity should be mobilised below the plates, so that the nails can mobilise full pullout capacity in the resisting zone. In the design the additional force of frost needs to be considered.

F.2.7.2 Esthetical aspects

As mentioned above the plates need to be designed to take some of the load and will consequently have larger size than if they were not contributing. To obtain a facing that adapts to the environment a grass seeding is used together with a geotextile. This results in a green slope.

F.2.8 Drainage

One important part of the design is to consider the drainage of the slope. A cut-off trench above the slope in connection to the road is necessary to take care of the water from the road, that in other case could have negative effects on the facing of the slope.

F.2.9 Serviceability state

In this case the purpose of the soil nailing is to reduce the movement that has resulted in cracking of the road. However, since soil nailing is a passive reinforcement it will not mobilise force until a small movement has occurred between the nail and the soil. An estimate of the movement that could be expected to occur before the nail will start working and stop the movement is made. According to the empirical correlation in chapter 7 the movement of the reinforced block should be about 2H/1000, *i.e.* 2.8 cm. If movement greater than this occurs it could be an indication of that the nails do not work properly.

F.2.10 Acknowledgement

This example is based on a project that was designed by SGI. However, the slope geometry has been slightly changed for the purpose of this example.