BS 8006-2:2011

Incorporating Corrigenda Nos. 1 and 2

BSI Standards Publication

Code of practice for strengthened/reinforced soils

Part 2: Soil nail design

Publishing and copyright information

Published by BSI Standards Limited 2013

ISBN 978 0 580 84676 2

ICS 93.020

The following BSI references relate to the work on this standard:
Committee reference B/526/4
Draft for comment 11/30161659 DC

Publication history

First published (as BS 8006), November 1995
Revised as BS 8006-1, October 2010 and BS 8006-2 (this standard), December 2011

Amendments issued since publication

C2 Date	Text affected
November 2013	C2. Missing symbols added (pp. 5–6)
	C1. The word "nail" inserted in Table 5 (p. 43)
	C1. Correction to Figure 18 (p. 47)
	C1. Missing equation added to **4.3.5.2** (p. 58)
	C1. Corrections to Figure 32 (p. 82)
	C1. Corrections to Figure 33 (p. 84)
	C1. BS EN reference number corrected in **5.1,** Note 1 (p. 89)
	C1+C2. See Foreword C2

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Summary of pages

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Foreword

Publishing information

This part of BS 8006 is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 31 December 2011. It was prepared by Subcommittee B/526/4, *Strengthened/reinforced soils and other fills*, under the authority of Technical Committee B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

Together with BS 8006-1, this part of BS 8006 supersedes BS 8006:1995, which is withdrawn.

Relationship with other publications

This standard is published in two parts:

- *Code of practice for strengthened/reinforced soils and other fills*
- *Code of practice for strengthened/reinforced soils – Part 2: Soil nail design*

This part has been drafted following the principles of BS EN 1997-1:2004.

Information about this document

This part of BS 8006 was drafted to meet the specific needs of designers and installers of soil nails for strengthening and/or reinforcing soil slopes.

Text introduced by or altered by Corrigenda Nos. 1 and 2 is indicated in the text by tags [C1⟩ ⟨C1] and [C2⟩ ⟨C2]. Minor editorial corrections are not tagged.

Use of this document

As a code of practice, this part of BS 8006 takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Any user claiming compliance with this part of BS 8006 is expected to be able to justify any course of action that deviates from its recommendations.

It has been assumed in the preparation of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people, for whose use it has been produced.

The recommendations in this British Standard are based on typical UK practice and therefore might not be wholly valid in other territorial or regional environments. Design checks in accordance with other British or international Standards might be necessary.

This standard is likely to be used under a variety of contractual arrangements and forms of contract. In many cases multiple designers might be involved. Therefore, irrespective of the contract form it is essential that the design of the soil nailing element of a project is properly integrated into whole scheme and contractual interfaces are clearly and appropriately specified within contract documents.

Presentational conventions

The provisions in this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is “should”.

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

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Section 1 : General

1.1 Scope

This part of BS 8006 gives recommendations and guidance for stabilizing soil slopes and faces using soil nails. Other methods of stabilization using reinforced soil methods are given in BS 8006-1:2010 and both parts might be needed for complex structures.

Additional considerations might be required for unusually loaded or high soil nailed slopes, or where they interface with other structures.

Whilst BS EN 1997-1:2004 specifically excludes soil nailing, this standard is intended to harmonize the design approach of soil nailing with other geotechnical structures designed using BS EN 1997-1:2004.

The principal purpose of this standard is to provide design guidance, however, where knowledge of construction methodology is required for design purposes then appropriate paragraphs have been included. Construction guidance is given in execution standard BS EN 14490:2010. At the time of preparation of this standard, CEN Technical Committee TC341 is drafting a standard covering the testing of soil nails.

Structures and processes that are similar to soil nailing but not addressed in the standard are described in **2.3.6**.

1.2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 8006-1:2010, *Code of practice for strengthened/reinforced soils and other fills*

BS 8081, *Code of practice for ground anchorages*

BS EN 196 (all parts), *Methods of testing concrete*

BS EN 197-1:2000, *Cement – Part 1: Composition, specifications and conformity criteria for common cements*

BS EN 206-1, *Concrete – Part 1: Specification, performance, production and conformity*

BS EN 1537, *Execution of special geotechnical work– Ground anchors*

BS EN 1990, *Eurocode – Basis of structural design*

BS EN 1992-1-1, *Eurocode 2 – Design of concrete structures – Part 1-1: General rules and rules for buildings*

BS EN 1997-1:2004, *Eurocode 7 – Geotechnical design – Part 1: General rules*

BS EN 1997-2, *Eurocode 7 – Geotechnical design – Part 2: Ground investigation and testing*

BS EN 10080, *Steel for the reinforcement of concrete – Weldable reinforcing steel – General*

BS EN 14487 (both parts), *Sprayed concrete*

BS EN 14490:2010, *Execution of special geotechnical works – Soil nailing*

BS EN ISO 14688 (both parts), *Geotechnical investigation and testing – Identification and classification of soil*

BS EN ISO 14689-1, *Geotechnical investigation and testing – Identification and classification of rock – Part 1: Identification and description*

BS EN ISO 22475-1, *Geotechnical investigation and testing – Sampling methods and groundwater measurements – Part 1: Technical principles for execution*

BS EN ISO 22476 (all parts), *Geotechnical investigation and testing – Field testing*

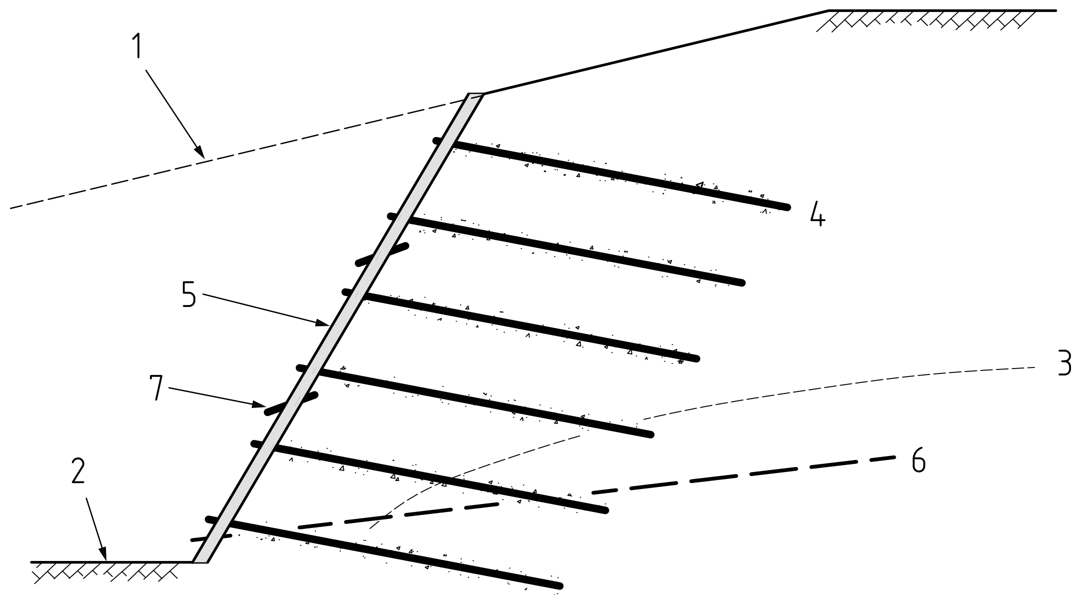
1.3 Terms, definitions and symbols

1.3.1 Terms and definitions

The following terms and definitions apply.

NOTE Some additional terms are illustrated in Figure 1.

Figure 1 **Terms used in this standard**



NOTE Other systems might not use grout/duct/couplers/facing/spacers.

Key

1 Original ground surface
2 Base of new cut
3 Groundwater (drained)
4 Soil nail
5 Facing
6 Deep drain
7 Weep hole

1.3.1.1 bearing plate

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

[BS EN 14490:2010]

1.3.1.2 comparable experience

documented or other clearly established information related to the ground being considered in design, involving the same types of soil and rock and for which similar geotechnical behaviour is expected, and involving similar structures; information gained locally is considered to be particularly relevant

[BS EN 1997-1:2004]

1.3.1.3 derived value

value of a geotechnical parameter obtained by theory, correlation or empiricism from test results

[BS EN 1997-1:2004]

1.3.1.4 design life

service life in years required by the design

[BS EN 14490:2010]

1.3.1.5 drainage system

series of drains, drainage layers or other means to control surface and ground water

[BS EN 14490:2010]

1.3.1.6 facing

covering to the exposed face of the reinforced ground that may provide a stabilizing function to retain the ground between soil nails, provide erosion protection and have an aesthetic function

NOTE See Figure 1.

[BS EN 14490:2010]

1.3.1.7 facing drainage

system of drains used to control water behind the facing

[BS EN 14490:2010]

1.3.1.8 facing system

assemblage of facing units used to produce a finished facing of reinforced ground

[BS EN 14490:2010]

1.3.1.9 facing unit

discrete element used to construct the facing

[BS EN 14490:2010]

1.3.1.10 flexible facing

flexible covering which assists in containing soil between the nails

[BS EN 14490:2010]

1.3.1.11 geotechnical action

action transmitted to the structure by the ground, fill, standing water or ground-water

[BS EN 1997-1:2004]

1.3.1.12 geotechnical category

category assigned to a structure in order to establish minimum requirements for the extent and content of geotechnical investigations, calculations and construction control checks of the design in relation to the associated risks

[BS EN 1997-1:2004]

1.3.1.13 ground

soil, rock and fill existing in place prior to the execution of the construction works

[BS EN 14490:2010]

1.3.1.14 hard facing
stiff covering, for example sprayed concrete, precast concrete section or cast in-situ concrete

[BS EN 14490:2010]

1.3.1.15 production nail
soil nail which forms part of the completed soil nail structure

[BS EN 14490:2010]

1.3.1.16 proof load
maximum load applied during testing

[BS EN 14490:2010]

1.3.1.17 raking drain
drain, normally drilled into the slope through the front face, raked above horizontal

1.3.1.18 reinforcing element
generic term for reinforcing inclusions inserted into ground

[BS EN 14490:2010]

1.3.1.19 reinforced ground
ground that is reinforced by the insertion of reinforcing elements

[BS EN 14490:2010]

1.3.1.20 resistance
capacity of a component, or cross-section of a component, of a structure to withstand actions without mechanical failure e.g. resistance of the ground, bending resistance, buckling resistance, tensile resistance

[BS EN 1997-1:2004]

1.3.1.21 sacrificial nail
soil nail installed in the same way as the production nails, solely to establish the pullout capacity and not forming part of the soil nail structure

[BS EN 14490:2010]

1.3.1.22 soft facing
soft facing has only a short-term function to provide topsoil stability while vegetation becomes established

[BS EN 14490:2010]

1.3.1.23 soil nail
reinforcing element installed into the ground, usually at a sub-horizontal angle, that mobilizes resistance with the soil along its entire length

NOTE See Figure 1.

[BS EN 14490:2010]

1.3.1.24 soil nail construction
any works that incorporates soil nails, and can have a facing and/or a drainage system

[BS EN 14490:2010]

1.3.1.25 soil nail system
consists of a reinforcing tendon and may include joints and couplings, centralisers, spacers, grouts and corrosion protection

[BS EN 14490:2010]

1.3.1.26 stiffness
material resistance against deformation

[BS EN 1997-1:2004]

1.3.1.27 structure
organized combination of connected parts, including fill placed during execution of the construction works, designed to carry loads and provide adequate rigidity

[BS EN 1997-1:2004]

1.3.1.28 tendon
structural component of a soil nail, often in the form of a solid or hollow bar and running the full length of the soil nail

1.3.1.29 test nail
nail installed by the same method as the production nails for the purpose of verifying the pullout capacity and durability, and which could form part of the structure

[BS EN 14490:2010]

1.3.2 Symbols

The following variables are used in this standard.

Symbol	Definition	Units
a	Nail plate size	m
A_s	Cross-sectional area of nail tendon for structural design	m²
$A_{s,nom}$	Nominal cross-sectional area at end of design life	m²
B	Slice width	m
c'_d	Design value of the cohesion intercept in terms of effective stress ($c'_k/\gamma_{c'}$)	kN/m²
c_u	Undrained shear strength	kN/m²
$C_{1,2}$	Cohesion force acting on base of wedges 1 or 2	kN/m
d_{hole}	Nail hole diameter	m
d_{bar}	Diameter of reinforcing element, tendon or bar	m
[C2⟩ d_m	Diameter of the front face mesh element	m ⟨C2]
E_d	Design value of the effect of actions	kN
[C2⟩ E_m	Young's modulus for the mesh material	kN/m² ⟨C2]
f_y	Yield strength of nail tendon	kN/m²
f_{yk}	Characteristic yield strength of nail tendon	kN/m²
H	Slope height	m
K	Earth pressure coefficient in terms of total stress and factored soil strength (*not* equivalent to the active Rankine co-efficient, k_a)	—
k_a	Rankine active earth pressure co-efficient	—

k_L	Ratio of the in plane horizontal effective stress and the vertical effective stress	—
k_r	Ratio of the average radial effective stress σ'_r on a nail to the vertical effective stress σ'_v	—
L_n	Total length of nail ($L_n = L_a + L_e$)	m
L_a	Length of nail in active soil zone (front of nail)	m
L_b	Bonded length of test nail	m
L_e	Embedded length of nail in stable soil (back of nail)	m
$M_{driving}$	Driving moment	kNm/m
$M_{resisting}$	Resisting moment	kNm/m
$N_{12,21}$	Normal total force acting on inter-wedge boundary	kN/m
p_h	Average pressure acting behind hard facings (Figure 31)	kN/m²
p_m	Average pressure acting on flexible facing (Figure 32)	kN/m²
p_{np}	Average pressure acting on nail plate (Figure 34)	kN/m²
P_{ult}	Ultimate pullout force achieved in test	kN
$Q_{1,2}$	Characteristic surcharge force acting on the surface of wedges 1 or 2	kN/m
q_p	Permanent unfactored surcharge pressure acting on the surface of a slice	kN/m²
q_v	Variable unfactored surcharge pressure acting on the surface of a slice	kN/m²
r	Radius of slip circle	m
$R_{1,2}$	Frictional force acting on base of wedges 1 or 2	kN/m
R_d	Design resistance	kN
R_{td}	Design nail tendon strength	kN
R_{tk}	Characteristic nail tendon strength	kN
S_h	Horizontal nail spacing	m
C2 S_m	Horizontal spacing between elements within the mesh	m C2
S_v	Vertical nail spacing	m
$\tan \varphi'_d$	Design value of the tangent of the angle of shearing resistance ($\tan \varphi'_k/\gamma_{\tan\varphi'}$)	—
T_d	Design nail force per nail, which is the lesser of the nail's factored pull-out capacity ($\tau_{bk}\pi d_{hole}L_e/\gamma_{\tau b}$) and its factored tendon strength ($f_{yk}A_{s,nom}/\gamma_s$) (see **4.3** and **4.5** and Figure 21)	kN
T_f	Axial force in nail head at facing	kN
T	Axial force in nail	kN
$U_{1,2}$	Pore water force acting on base of wedges 1 or 2	kN/m
u	Pore pressure acting on base of slice	kN/m²
W	Self-weight of slice	kN/m
z	Depth of mid-point between layer of nails in question and the next layer below	m

α	Angle to the vertical from centre of rotation to base of slice	degrees
β_1	Principal slope angle	degrees
β_2	Upper slope angle	degrees
γ	Unit weight [or weight density in BS EN 1997 (both parts)]	kN/m^3
$\gamma_{c'}$	Partial factor on effective cohesion	—
γ_{cu}	Partial factor on undrained shear strength	—
γ_g	Partial factor on weight of soil	—
γ_k	Partial factor used to derive a characteristic parameter	—
γ_{qp}	Partial factor on permanent surcharge	—
γ_{qv}	Partial factor on variable surcharge	—
γ_s	Partial factor on strength (including strength of nail tendon)	—
γ_{Sd}	Model factor	—
γ_m	Partial factor on material (structural) strength	—
$\gamma_{\tan\varphi'}$	Partial factor on tangent of soil friction angle	—
γ_u	Partial factor on pore pressure	—
γ_γ	Partial factor on unit weight (weight density)	—
$\gamma_{\tau b}$	Partial factor on bond stress	—
ε	Nail declination angle and/or strain	degrees
η	Angle defined as 90° – α – ε (Figure 17 and Figure 18)	degrees
η_d	Design nail plate constant (Figure 30)	—
$\theta_{1,2}$	Base angle of wedge 1 or 2	degrees
λ_f	Interface factor relating mobilized friction angle to characteristic friction angle	—
ξ	Correlation factors on the minimum and mean pullout resistances or bond resistances measured in a soil nail pullout test	—
σ'_r	Radial effective stress acting normal to the surface of a nail	kN/m^2
σ'_v	Vertical effective stress	kN/m^2
τ_b	Bond stress	kN/m^2
φ'	Friction angle of soil	degrees

Subscript

i	Relates to slice number *i* (from 1 to *n*)
j	Relates to nail row number *j* (from 1 to *m*)
d	Design value of parameter
k	Characteristic value of parameter
t	Test value of parameter
u	Ultimate value of parameter (except as defined elsewhere, e.g. c_u and γ_u)

Section 2 : Soil nailing applications and construction considerations

2.1 General

*COMMENTARY ON **2.1***

Soil nailing is a ground stabilization method used to enhance the stability of slopes and faces. It is employed in "in-situ" ground, which may be natural or deposited by man, by the insertion of soil nails. Correctly orientated soil nails can improve the shear strength of soil, which is naturally weak in tension. Frictional forces are mobilized when surrounding soil shears against relatively inextensible soil nails. This results in:

- *an improvement of the slope's mass shear resistance; and*
- *development of tensile forces in the soil nails.*

Depending on the soil nailing application, the soil nails can rely on other elements such as bearing pads, facing systems and drainage to be most effective. The effectiveness and efficiency of a soil nailing application is highly dependent upon the ground conditions.

Soil nailing applications are described in **2.3**.

2.2 Description of typical soil nail element components

A typical soil nail element comprises a number of components, some of which are required for structural load carrying capacity and others to ensure durability over the design life. The material requirements of the components are detailed in BS EN 14490:2010, but a brief description is given below, with reference to Figure 2.

The key components can be described as:

a) *Tendon*: This is the main component for transferring axial load along the length of the soil nail. It is typically a steel bar or rod, with diameters in the range 10 mm to 32 mm. Tendons with other diameters or made from other materials may be used provided they exhibit the necessary mechanical properties. Tendons may be constructed from several sections joined by couplers. The structural capacity of the couplers, or reduced section area at the coupler, may dictate the design strength of the tendon.

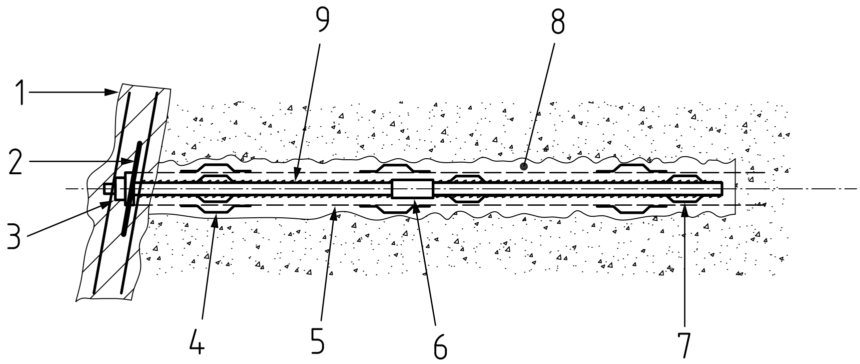
b) *Head plate and locking nut*: A head plate and locking nut are used to transfer load between the tendon and facing. A tapered washer, or similar arrangement, is often inserted between the head plate and locking nut to ensure even load distribution; particularly where the tendon and facing are not arranged orthogonally. The head plate may be embedded within the facing or bedded onto it or a concrete pad. Concrete pads can be recessed into the slope so they are less visible.

c) *Protective ducts, sheaths and coatings*: Protective ducts, sheaths and coatings may be used to improve durability of the soil nail. Where required they should be detailed to ensure continuity along the entire length of the nail element. Coatings may be applied directly to the reinforcing element.

d) *Grout annulus*: The grout annulus is designed to provide intimate contact between the soil nail and the ground. It should also be designed to protect the tendon from chemical attack in aggressive soil. Spacers around the tendon may be used to ensure adequate grout cover. Separate inner and outer annuli may be formed if ducts or sheathes are included in the design.

Inner and outer spacers would normally be required in such cases and different grout specifications may be applied to each annulus.

Figure 2 **Possible components of soil nail system, pre-bored and grouted, shown with rigid facing**



NOTE Other systems might not use grout/duct/couplers/facing/spacers.

Key

1 Facing
2 Head plate
3 Locking nut
4 Outer spacer
5 Duct
6 Coupler
7 Inner spacer
8 Grout annulus
9 Reinforcing element

2.3 Typical applications

2.3.1 Stabilizing new cut slopes

Where ground is excavated at an angle steeper than that at which it can stand safely then soil nails may be used to improve the stability. Soil nailing may be used to stabilize slope angles up to the vertical.

NOTE 1 Early examples of new cut slopes were cited by Bruce and Jewell [16] and more recently, a number of UK examples were presented in CIRIA C637 [1].

New cut slopes should normally be constructed incrementally. Each increment should consist of a phase of excavation to a stable level, followed by a phase in which soil nails and facing are placed in the newly cut face. This sequence should continue until the full depth of the excavation is achieved [Figure 3a)].

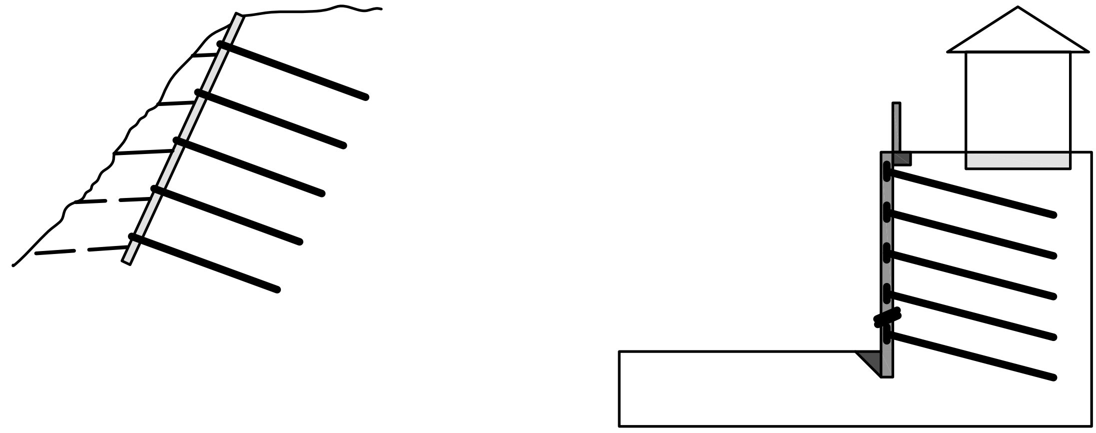
NOTE 2 In this way, the soil nails become progressively loaded as excavation induced movements occur.

New cuttings may be formed in natural ground or in existing earthworks; the latter occur typically when an existing cutting is widened without increasing land take at the crest or when the toe of an embankment is cut away to facilitate later construction without narrowing the crest.

Recommendations for the design for the soil nails and facing become more onerous as the slope angle increases. The case of vertical or near vertical soil nailed structures should be considered further as often the crest of such slopes are close to existing structures, foundations, infrastructure or services where ground movement might be critical [Figure 3b)]. Furthermore, as the near surface ground is often poor and sensitive to surcharging the control of groundwater, excavation, stability of the cut face and nail installation should be critical design considerations.

Reference to BS 8002 may be considered appropriate for some elements of the design of steep soil nailed slopes that are relevant to retaining structures.

Figure 3 **Typical soil nailing applications (new cut and vertical cutting)**



a) New cut
b) Vertical cutting

2.3.2 Existing slopes

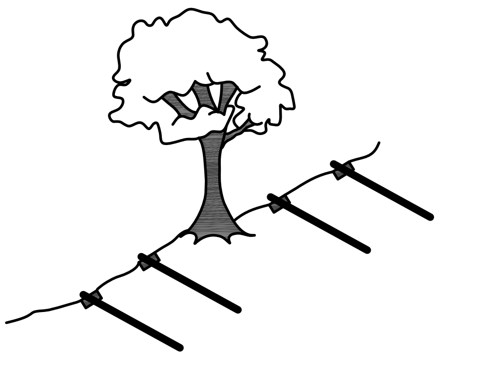
Soil nailing may be used to improve the stability of both natural and man-made slopes with a range of slope angles. It should normally be used in such applications where the slope is deemed to have an unacceptably low factor of safety and presents a hazard to adjacent land usage or places a restriction on its development. There are many examples of where it may be used on coastal slopes (above the tidal zone) where its purpose would also include reducing the effects of weathering.

NOTE Soil nailing is likely to be more effective in reducing the risk of a first time failure than the remediation of existing failed slopes. Unlike the case for new cut slopes, it might be necessary to back analyse the existing slope on the assumption that a factor of safety of unity exists and demonstrate an improvement in stability (against all likely failure modes) following nailing to achieve an economic design. Unless or until further movement of the slope occurs the soil nails will remain unloaded.

The arrangement of soil nailing used in a remedial situation may be very different to that in a preventative one as a specific failure mode will most likely need to be addressed.

For natural slopes (Figure 4) the design should take cognisance and be sensitive to existing vegetation and wildlife and often soft or flexible facing systems may be preferred, in association with landscaping.

Figure 4 **Soil nail placement to preserve existing vegetation**



2.3.3 Existing retaining structures

NOTE Some of the first uses of soil nailing in the UK were for the upgrading and remediation of existing retaining walls (see Barley [15]).

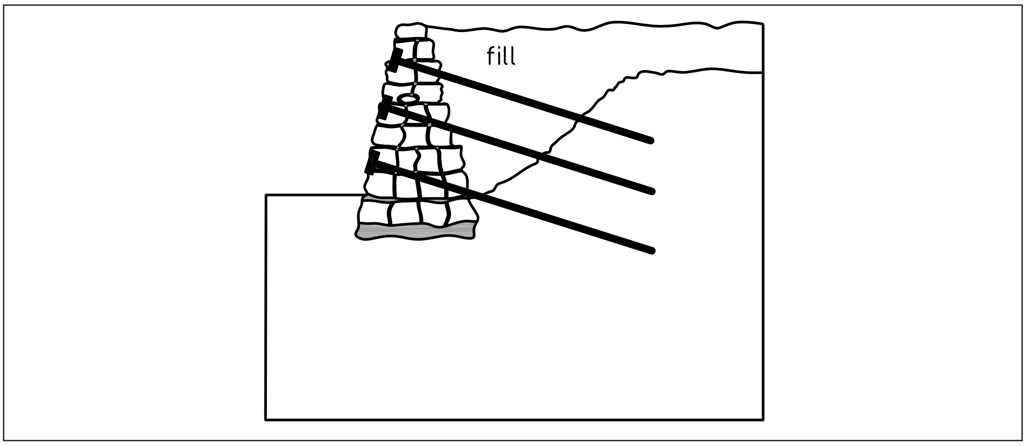
Soil nailing may be used for many retaining wall types including: stone, brick or concrete walls and abutments that are suffering distress, or likely to do so due to a change of usage, i.e. remedial or preventative works (see Figure 5). The soil nailing solution may be significantly different for the case of remedial works relative to that required for preventative works as a specific failure mechanism might have to be targeted.

The key element that should be considered in the design of works to existing retaining structures is the connection of the nails to the existing structure and the load transfer at this point to prevent pullout or damage to the structure. In these applications special measures may be required to stabilize the ground immediately behind the structure and permit nail installation, as often this comprises poor quality fill material.

Where drainage exists it is important to review the impact of the upgrading works in terms of construction process and long-term effectiveness. Where necessary, existing drainage should be protected.

As with existing slopes, it may be necessary for further movement of the structure to occur before the nails can be effective. Safe access for nail construction should be considered as this can have a significant impact on feasible solutions.

Figure 5 **Example of soil nailing of an existing retaining structure**



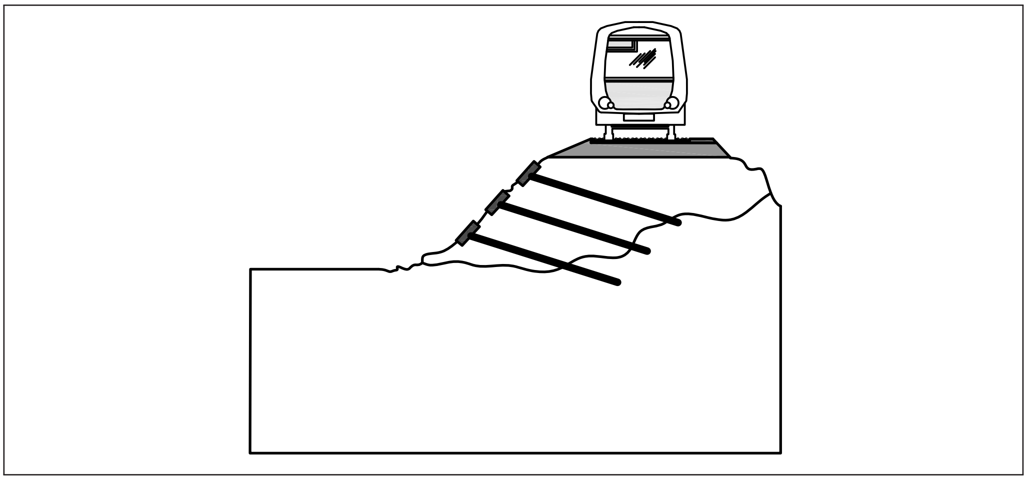
2.3.4 Embankment stabilization

NOTE 1 Embankment stabilization is one of the most common forms of soil nailing being used, especially by the rail industry (Figure 6), where increased trafficking requires a greater reliability of embankment performance. It has also been used on highway and dam embankments.

Many older embankments were constructed with variable and often uncontrolled fills and can often suffer from shrinkage and swelling due to seasonal moisture content and porewater pressure changes; in such cases soil nails may be used to stiffen the embankment and reduce the requirement for reballasting. They may also be used to provide enhanced stability to the slopes and prevent deep slips that could undermine the track or carriageway. The utilization of soil nails on embankments should be challenged at an early stage of the design as to whether their primary purpose is to improve serviceability or stability and how they will achieve this.

NOTE 2 Many of the issues relating the use of soil nailing for embankments are covered in CIRIA C591 [8].

Figure 6 **Example of soil nailing of an embankment**



2.3.5 Hybrid applications

There are a number of applications where soil nails may be used in combination with other forms of construction such as providing an anchorage for reinforced fills, or where they may be used in combination with rock bolts and ground anchors to stabilize a variably weathered slope. Ground improvement techniques such as grouting or electro-osmosis may be used to treat the ground into which the soil nails are inserted. In such cases it may be appropriate to use certain elements of this standard for the soil nails or facing materials. However, many of the mechanisms discussed for design may not be wholly appropriate and reference to other standards should be considered.

2.3.6 Applications of a similar nature but not relevant to this standard

This standard is not applicable to the design of ground anchors or rock bolts despite their similarities with soil nails; this is because the fundamental mechanisms by which they stabilize the ground are different and often geological features, such as jointing or bedding planes, dictate the failure mechanisms used for design; reference should be made to BS 8081.

Other applications that involve similar construction processes such as reticulated minipiles, or where the soil nails work predominantly as shear dowels, should not be designed using this standard.

2.4 Construction design considerations

*COMMENTARY ON **2.4***

Soil nailing execution is covered by BS EN 14490:2010. Further detail on construction considerations and examples are given in CIRIA C637 [1], Clouterre [19] and FHWA [20]. Recommendations and guidance in this standard relating to construction are provided where the construction operation might have a direct influence on the design model, method or parameters chosen.

2.4.1 Preliminary works

Any preliminary works requiring design such as an intrusive site investigation, drainage to control surface runoff, temporary dewatering, edge protection, underpinning of adjacent footings, working platforms or access scaffolds, should be designed by a competent person, prior to commencement on site. The location, identification, assessment and if appropriate protection of any adjacent services or utilities should be carried out. The requirement for easements to install soil nails should be established before a design is finalized.

Where monitoring is required as a part of the design then baseline readings and background influences should be recorded prior to commencement to permit the necessary control during construction. The designer should also provide alarm and intervention trigger levels and the appropriate actions to be taken in event of them being exceeded.

2.4.2 Excavation and face preparation

Where the soil nailing process includes excavation, the designer should consider placing limits on the maximum depths, lengths and slope angles that can be excavated and specifying these to the constructor. The maximum height of cut slope that can safely stand unsupported until nail installation and facing construction should be taken into account when specifying the nail spacing.

The design of an excavated slope needs to address both the local (internal) stability of the soil nailed face and the global (external) stability of the whole slope (Figure 7). The criteria for internal and external stability are addressed in Section **4**.

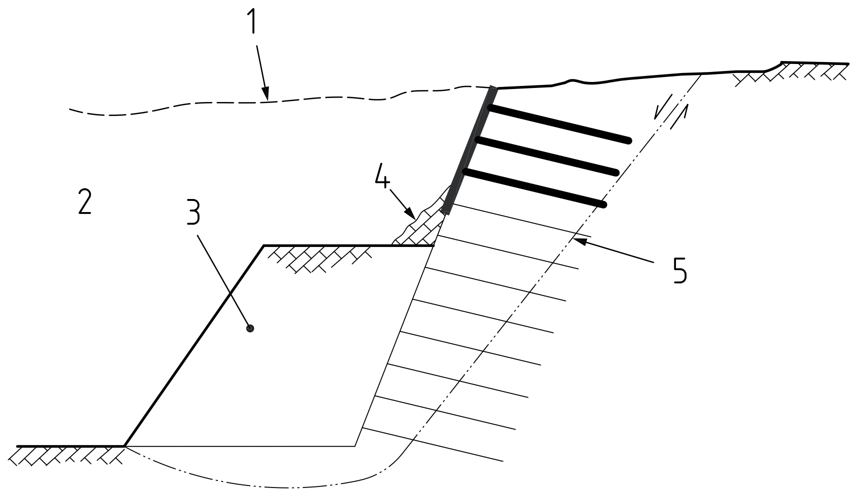
BS EN 14490:2010 gives details of face stability assessment tests that should be considered in the assessment of temporary stability and excavation bench heights. The maximum bench height needs to be considered when determining the vertical nail spacing.

The risk of undercutting increases with the steepness of the cut face. For very steep faces this could lead to dramatic reductions in stability. In such cases greater control of face trimming is required. The designer should specify the tolerances upon which the design is based, taking into account the structure and fabric of the ground and the likely excavation methods to be used. As a general rule slope excavation should start within 50 mm of the design crest. The excavation should be cut to within 2.5° of the design slope angle (β) [Figure 8a)]. Deviation from the design slope between nails should not exceed the lesser of either the nail spacing divided by 20 mm or 50 mm [Figure 8b)]. Local bumps and indentations are permitted, provided they do not affect the integrity of the facing.

NOTE Control of face trimming is important as an uneven face can affect the performance of, or ability to, correctly construct the facing. The positioning of nail heads can become problematic and wastage of materials increases.

If a design is sensitive to the changes in ground conditions, the design should clearly state the expected conditions and acceptable variations in strength or location of individual strata, with instructions on reporting back to the designer.

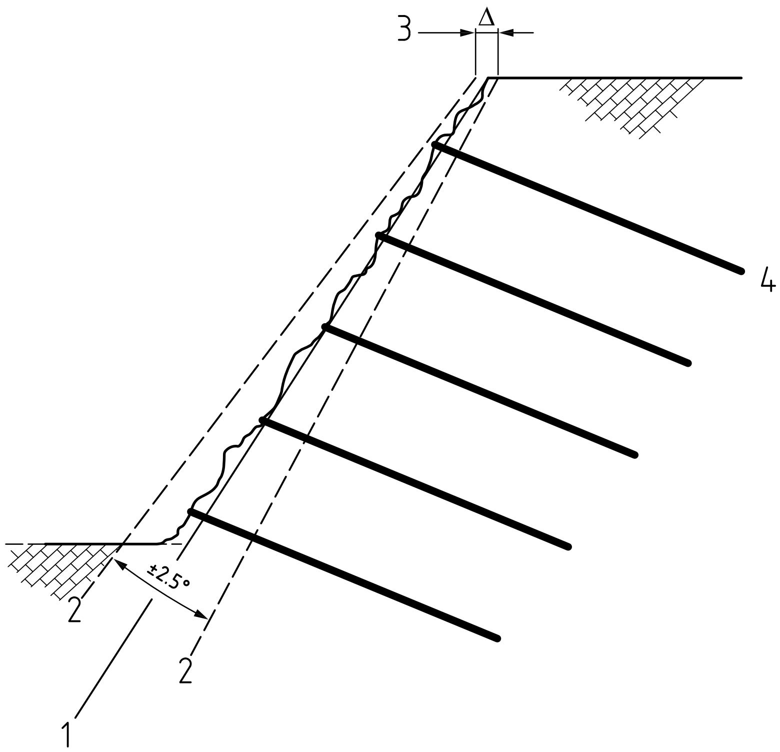
Figure 7 **Bulk excavation and requirement to check overall stability**



Key

1 Original ground surface
2 Soil excavated during bulk excavation
3 Berm to provide nail installation access and prevent deep slip failure until nails are installed
4 Local face trimming before installing nail/facing on current bench
5 Potential deep failure if berm is removed prior to nail installation

Figure 8 **Excavation tolerances**

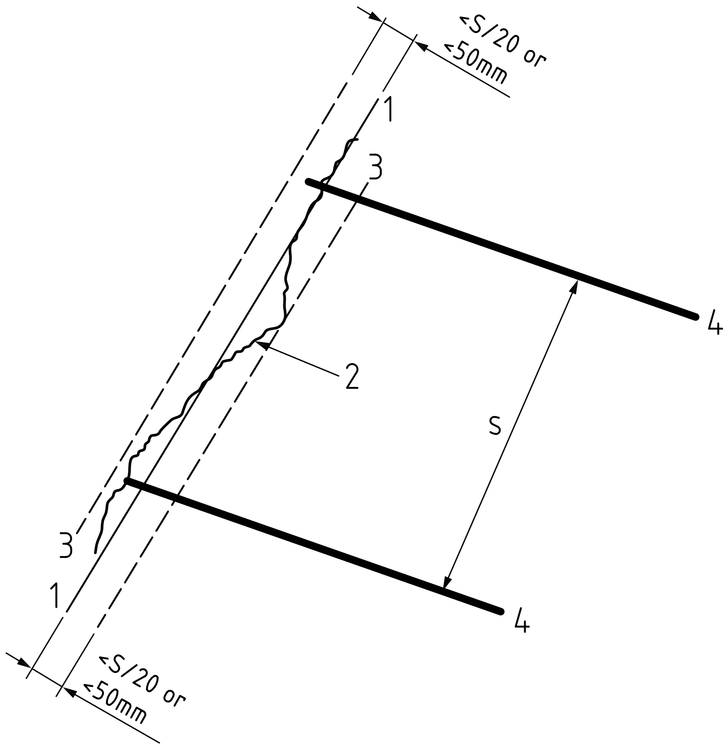


a) Excavation tolerance relative to crest

Key

1 Design excavation line
2 Excavation tolerance limits ±2.5° from design excavation line relative to slope crest
3 Slope crest
△ Crest tolerance (set by designer)

Figure 8 **Excavation tolerances**



b) Suggested local trimming tolerance between nail rows

Key

1 Design excavation line

2 Actual excavated face

3 Tolerance limit between nail rows

4 Soil nails

S Spacing between nail rows measured parallel to the face

2.4.3 Nail installation

2.4.3.1 General

The method of nail installation chosen will have a direct impact on the pullout resistance that may be used in the design. Some methods of nail installation may be deemed inappropriate because of the specified nail length and diameter, ground conditions, required durability or equipment required for installation when taking into account the safe access or manual handling requirements.

2.4.3.2 Soil nail installation tolerance

The designer should take into account the requirements of the facing when specifying nail installation tolerances. The nail location on the face should generally not be specified as less than ±100 mm and the nail orientation to less than ±5°. Potential clashing of soil nails should be considered.

2.4.3.3 Direct installation methods

Direct installation methods include driven (percussive and vibratory), jacked and fired soil nails. When selecting these methods the designer should consider the implications of refusal of the nail before the design penetration has been reached and the potential for damage during installation that could affect durability. Compared to other installation methods, direct installation tends to utilise relatively short, small diameter, soil nails. The relatively small surface area of each nail generally results in a higher density of nails than for an equivalent drilled soil nail design.

2.4.3.4 Drilled installation methods

Drilled methods may be used in all soil types, for a wide range of nail diameters, lengths and durability systems. They should nearly always include grouting.

The drilling method and flushing medium (if used) can affect the friction between the nail and the ground and consideration of potential effects should be made.

NOTE Potential adverse effects of drilling methods include softening of cohesive soils by water flush, scour of granular soils by air flush and smearing of the bore walls through augering.

A requirement for casing to be used should be assessed and the ability to install the nail element whilst keeping it clean prior to grouting should be considered carefully and where appropriate taken into account in the design.

In general, keeping the cycle time between nail drilling and grouting to a minimum is beneficial with respect to the frictional resistance available; test nails should be constructed to be representative of the production nails.

The nail design should ensure that the method of installation addresses continuity of the corrosion protection system over its entire length and into any facing system or head plates/bearing pads.

2.4.4 Grouting

*NOTE 1 Effective grouting is important in achieving the design frictional resistance and required durability. Most soil nails involve grouting under gravity, but in some soils grouting as a part of the drilling process or grouting at an elevated pressure can enhance the friction available between the nail and the ground (see **4.3**).*

Where the design is based upon the benefit of elevated grout pressures then the designer should consider the use of design investigation tests or suitability tests to confirm the enhancement achieved.

NOTE 2 Grout take is the volume of grout that has to be injected to completely fill the annulus around a drilled soil nail. Grout loss can be defined as the volume of grout take exceeding the design or expected volume. It occurs when injected grout escapes into the interstitial voids of the soil in greater quantities than predicted. It is a particular risk in clean, coarse granular soils. Possible mitigation measures include increasing grout viscosity (e.g. using sanded grout mixes or gelling agents) or applying multiple injections of grout. The soil nail designer needs to be alert to the increased risk of grout loss in granular soils and make adequate provision to deal with the consequences.

The designer should specify the characteristic grout strength to be achieved in samples before loading of the nails to prevent damage and to achieve the durability required. Minimum characteristic strengths of 5 N/mm^2 and 25 N/mm^2 prior to loading and at 28 days respectively should be specified, as required by BS EN 14490:2010.

NOTE 3 BS EN 14490:2010 requires a minimum cube strength of 5 N/mm^2 to be achieved prior to excavation below a nail.

Typically grout should comprise water mixed with a BS EN 197-1:2000, CEM I or CEM II cement in the ratio in the range 0.40 to 0.45. Sand filler and admixtures may be used in the grout in some instances.

NOTE 4 The high cement content of cementitious grouts generally makes them durable. At the time of writing, however, there is limited information on grout durability in contact with ground and groundwater. Guidance developed for concrete (e.g. BRE SD1 [3]) is not directly applicable.

Time should be allowed for the grout to develop sufficient strength before planned loading of the soil nails. This requirement is particularly applicable to situations where soil nails are installed to support a planned excavation. In such cases the designer should clearly state the requirements for time to elapse between installation of the soil nails and start of excavation.

2.4.5 Drainage installation

General slope drainage and that required to control runoff are not considered in this standard but should be considered as a part of the overall scheme design. However, it is essential that the soil nail designer considers the nature of any drainage required to ensure design assumptions on groundwater or pore-pressures can be met during the design life of the structure. Where membrane type drains are positioned behind facings then the risk of damage and the hydraulic continuity of joints should be considered. If deep-drilled drains are to be installed there may be restrictions on the angle and head elevation of these imposed by the installation equipment. Access for maintenance should be considered, especially where new structures are located in front of the soil nailed slope. Maintenance requirements for drainage should be clearly specified by the designer and safe access considered as a part of the design.

2.4.6 Facing installation

There are too many facing types to outline the key construction features that interface with design (as described in **4.7**), however, attention to detail in the design is critical for the facing to achieve its aesthetic, serviceability and durability requirements. Specific construction issues that should be raised at an early stage of the design process include:

a) relative tolerances on the prepared ground face, soil nails and facing connections;

b) how joints between facing sections can be achieved;

c) connections between head plates and facings;

d) continuity of nail durability system/corrosion protection through the facing;

e) drainage and facing interfaces;

f) the sequencing of facing construction and access;

g) interfaces between safety barriers, etc., and the facing and whether relative movement during construction needs to be considered.

Soil nail facings should be constructed to prevent loosening of the ground immediately behind.

Section 3 : Suitability of ground and groundwater conditions

3.1 General

NOTE 1 Not all soils are suitable for nailing. Some soils present problems either during construction or in the long term, which make soil nailing impractical, uneconomical or unsafe as a solution. Recognition of such conditions at an early stage is vital to prevent selection of an inappropriate scheme.

Designers should obtain site-specific information on the geological and hydrogeological conditions and use this information to determine the specific risks and suitability of soil nailing.

Recommendations on the suitability of different ground conditions for soil nailing are discussed in this standard, based on the following principal material types and their characteristic properties.

a) Cohesive soils.

b) Granular soils.

c) Soft/weak rocks.

NOTE 2 Soil nailing has been used in a wide variety of the ground conditions that exist in the UK. A range of soil nailing equipment and construction techniques has been developed by specialist contractors to accommodate different ground conditions. Selecting the right technique requires knowledge of the prevailing conditions at the site.

3.2 Understanding the site geology

By its very nature, the successful application of soil nailing is highly dependant on the suitability of the ground conditions; a thorough understanding of the geological and hydrogeological setting of the site is imperative to assess the suitability of the technique for a given project. This understanding should include not only the material within which the soil nails would be installed but also the underlying strata, which may affect the global stability or deformation of the soil-nailed system.

The designer should consider a number of key questions about the geology of the site to determine its suitability for soil nailing including the following.

a) Does the site comprise natural or made ground?

b) What are the characteristic properties of the soil to be nailed? Is it cohesive or granular (or something in between) in its behaviour? Is it stiff/dense or soft/loose? Can the ground conditions be considered as homogeneous?

c) What is the nature of the underlying soil/rock?

d) What is the structure of the deposits: bedding, fissures, joints, fractures and/or laminations?

e) What chemical and mechanical processes are occurring or have occurred at the site?

f) What has the site previously been used for? Could there be any sources of contamination on the site? How might they affect construction workers and end users of the site? How might they affect durability of materials used in the proposed scheme and the need for special measures?

g) Does the site have a history of stability problems (pre-existing slope failures)?

h) What are the groundwater conditions?

i) What are the potential deformation mechanisms (including groundwater and/ or vegetation induced mechanisms and collapse potential)?

Many of these questions should and can only be adequately addressed by a comprehensive site investigation following a desk study and scrutiny by a geotechnical specialist. However, to make a preliminary assessment of the suitability of the ground for soil nailing, sufficient knowledge of the site conditions may usually be obtained from published geological information combined with a walkover survey of the site.

3.3 General requirements for suitability of soils and rocks for soil nailing

The success of a soil nailing scheme relies on the ability to form an adequate bond between the nail and the soil into which it is installed; for soil nailing to be economical, the soil should have sufficient shear strength to develop such a bond.

Steepening of existing slopes is often achieved through a cyclic sequence of benching and nailing. The faces to be nailed are typically between 1 m and 3 m high and need to stand unsupported pending installation of the soil nails. It is essential that the soil has sufficient shear strength to allow the bench faces to be formed and remain adequately stable in the temporary condition.

NOTE The suitability of typical UK soils and rocks for soil nailing is summarized in Table 1.

3.4 Suitability of cohesive soils for soil nailing

3.4.1 Soft clays and silts

Soft clays and silts are generally unsuitable for soil nailing.

3.4.2 Firm and stiff clays

Soil nailing is generally well suited to firm and stiff clays (i.e. clays with an undrained shear strength of 50 kN/m^2 or greater). Such clays are usually overconsolidated. They generally provide sufficient stand-up time during excavation of benches to permit progress without immediate support of the cut face. They may also be assumed to provide good bore stability for drilled and grouted nails without the need for casing. Sufficient bond strength may usually be achieved, particularly in slopes where tension forces in the nails are relatively low.

3.4.3 Potential problems associated with soil nailing in cohesive soils

3.4.3.1 Shrinking and swelling

High plasticity clays have the potential to develop significant shrink/swell behaviour due to changes in moisture content. This might be due to cyclic changes in moisture content through precipitation and/or the effects of vegetation. Soil nailing may only be appropriate in these soils if it can be demonstrated that the design will cater for such potential deformations.

NOTE High plasticity, overconsolidated clays are very prone to volume change on wetting and drying. This is due to the presence of clay minerals of the smectite group, such as montmorillonite, which have "shrink/swell" properties. Soil-nailed slopes in such materials can suffer from deterioration, particularly at the crest and the face, due to repeated seasonal wetting and drying causing cyclic swelling and shrinking of the clay. This can result in corresponding fluctuations in the nail loading as well as localized slumping at the face. Clays having lower plasticity, due to a lower proportion of clay particles or lower proportions of smectite minerals are less inclined to shrink and swell.

Water flush should not be used to facilitate drilling in clays, as it will cause softening of the clay.

3.4.3.2 Previous disturbance and slope instability

Shear strength parameters for design of soil nailing in clays should be selected to reflect the degree of disturbance caused to the clay by previous activity and by the proposed works. For example, if a slope has suffered from previous slip failures, pre-existing shear planes might exist over which lower strengths apply and allowance should be made for the existence of such planes in design, both within the soil-nailed block and beneath it (Figure 9).

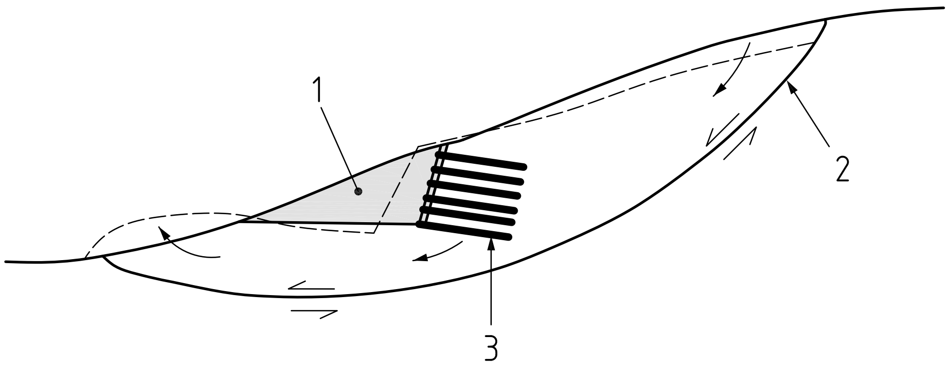
NOTE For high plasticity clays, the residual strength of the clay when it is sheared is much lower than the peak strength of the intact material.

Table 1 Summary of ground conditions best suited and less well suited to soil nailing

Developed from FHWA, 1998 [19] (This is Table 4.1 from CIRIA C637 [1].)

	Ground conditions best suited to soil nailing	Ground conditions less suitable for soil nailing	Possible measures to improve suitability of ground conditions
Material to be nailed	firm to stiff, low plasticity clays	soft cohesive and organic soils prone to creep deformation high plasticity or highly frost susceptible soils and rocks	none will improve these soils sufficiently for soil nailing provide adequate protection against wetting and drying
	well graded soils having a cohesive matrix such as some glacial tills, provided that cobbles and boulders do not obstruct nail installation fine to medium sands and silty sands with some apparent cohesion medium dense to dense sands and gravels with some apparent cohesion.	loose, clean sands and gravels with little or no apparent cohesion	pre-grouting or ground freezing to improve temporary stability limit excavation heights/lengths
	weathered rock without adverse joint orientation	weathered rock with adverse joint orientation or voids	
	engineered fills comprising selected natural or uniform, non-aggressive materials that have been carefully placed and well compacted to achieve characteristics similar to those of the natural soils described above	non-engineered fills, particularly those containing variable, aggressive and/or degradable constituents that are prone to collapse and differential settlement (and obstructions)	excavate, sort re-usable material and replace (unlikely to be cost effective) if fill constituents are suitable, ground improvement methods may be used to reduce the risk of collapse settlement, such as grouting, vibro-stone columns and dynamic compaction
Groundwater conditions	above the groundwater table with a dry excavated face	below the water table artesian groundwater at depth	temporary and permanent dewatering should be accommodated within the design, both in terms of internal and external stability
		perched water or groundwater seepage through granular soils or pockets	temporary dewatering and permanent drainage measures to ensure long term stability of the slope or wall.
Underlying ground conditions and geological features	underlying conditions and geological features that do not compromise the stability and performance of the soil nailed structure	adverse underlying ground conditions and geological features such as: pre-existing slip surfaces or adverse jointing soft, compressible layers unless strengthened before soil nailing voids such as solution features or mining cavities liquefiable silty soils	possible measures will depend on nature of the adverse conditions/features and has to be assessed on a site specific basis

Figure 9 **Examples of the effect of pre-existing shear surfaces on soil-nailed structures**



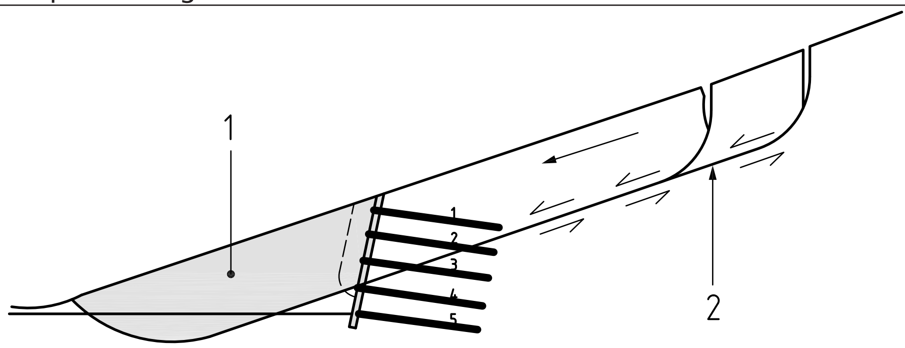
a) Pre-existing shear surface below a soil-nailed structure

Key

1 Toe weight is reduced during excavation of soil nailed structure

2 Reactivation of pre-existing shear surface

3 Soil nails do not extend below the shear surface and offer no resistance to slip



b) Pre-existing shear surface through a soil-nailed structure

Key

1 Toe weight is reduced during excavation of soil nailed structure

2 Reactivation of pre-existing shear surface

NOTE 1 Soil nails 1, 4 and 5 are above or below shear surface and offer no resistance to slip.

NOTE 2 Soil nail 2 has little anchorage below the shear surface and is likely to pull out.

NOTE 3 The wall facing may offer nominal shear resistance but the majority of the driving force from the slip is transferred to nail 3, which is likely to fail by pullout or rupture.

From CIRIA C637 [1].

3.4.3.3 Effect of grading

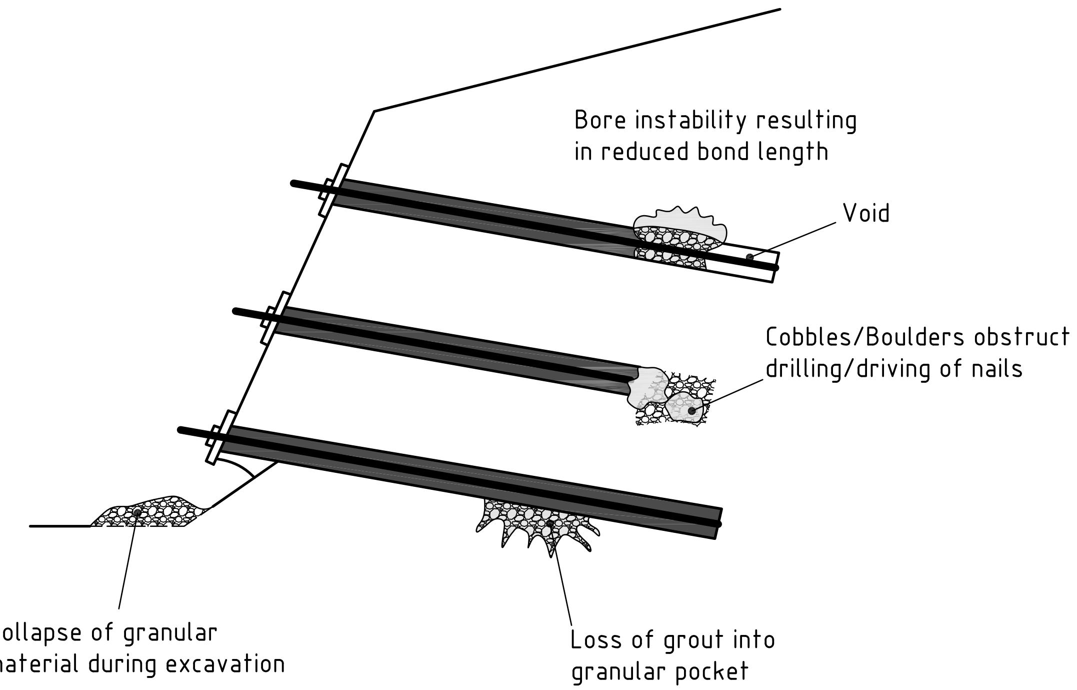
*COMMENTARY ON **3.4.3.3***

The presence of coarse material in a predominantly cohesive soil can cause problems during construction and in the long term performance of the soil-nailed structure. Interbedded clays have a more rapid response to pore water pressure change, particularly where layers are relatively thin, and drained conditions can be achieved in the short-term. This can affect temporary excavation stability.

The engineering properties of glacial tills are discussed in detail in CIRIA Report C504 [2]. The variability of grading of some of these materials makes them worthy of particular consideration. Glacial tills occur over much of the UK. Due to the nature of their deposition, the grading of glacial tills is highly variable, ranging from clay-sized particles to cobbles and boulders. The characteristic behaviour of the till depends on the proportion of fine material.

*Glacial and periglacial deposits where the clay fraction dominates, might still contain a significant proportion of coarse material, variably distributed within the finer matrix. Soil nailing is well suited to such mixed soils although larger particles and localized granular pockets can present some difficulties (Figure 10). Boulders and cobbles can obstruct drilling or driving or cause deflection. Localized problems can occur with stand-up time of temporary excavations and with stability of bores where zones of more granular material occur, particularly where seepage of groundwater occurs, with resulting wash-out of fines and loss of cohesion. Softening of the clay matrix occurs around seepage pathways (see **3.8**). Loss or washout of grout can occur into the granular material, although this rarely takes place in large volumes since granular material generally occurs in discrete, discontinuous pockets. Problems can also occur with the loss of air flush during drilling in glacial till of variable grading.*

Figure 10 **Problems caused by granular material in glacial till**



Based on CIRIA C637 [1].

3.4.3.4 Aggressive ground conditions

NOTE Compared with other solid geology, the weathered zones of the sedimentary clays are the most likely soils to contain high concentrations of sulfates (BRE SD1 [3]). This presents a risk of sulfate attack on soil-nail grout and concrete facings. Minerals such as pyrite and gypsum in an unweathered state can rapidly deteriorate if exposed to air and water during construction. Weathering of these minerals will release water soluble sulfates.

3.4.3.5 Drilling/driving problems

NOTE Very stiff clays are likely to be too resistant for successful construction of directly installed soil nails; drilled soil nail systems might be more appropriate. Ground consisting of interbedded clays, granular layers of rock can produce difficult conditions for augering and direct soil nail installation.

3.5 Suitability of granular soils for soil nailing

3.5.1 General

*COMMENTARY ON **3.5.1***

Many naturally occurring granular soils are well suited to soil nail applications. They offer high angles of shearing resistance, resulting in a strong bond. However, granular soils are only suitable provided they offer adequate short term cutting stability to allow application of a temporary facing. Sufficient temporary stability might be given by the presence of fine material to provide a small amount of cohesion between the grains. An apparent cohesion might also be given to sands by capillary tensions in the moisture between the grains.

In coarse granular soils with no appreciable fine matrix, capillary tensions are generally not sufficient to provide the apparent cohesion required for temporary stability.

Loose, clean granular soils, having a standard penetration test (SPT) N value of less than about 10 or relative density of less than about 30% are not well suited to and should not be used for soil nailing with steep cut slopes, due to poor stand-up times, poor bond and sensitivity to the vibrations caused by construction plant (FHWA [19]).

3.5.2 Potential problems associated with soil nailing in granular soils

3.5.2.1 Temporary stability problems

Where little or no cohesion or apparent cohesion exists, temporary drill casing is usually required, or drilling is carried out using a drilling fluid, such as a cement grout.

Short term stability of cut faces in granular soil is limited by the shear strength of the soil. Where the design requires a cut slope to stand at an angle greater than the angle of shearing resistance of the soil and places reliance on effective cohesion or soil suction, then close control is required and the designer should consider how the slope face is to be supported in the temporary condition. The use of controlled excavation, in short bays for limited periods of time, or temporary support, such as sprayed concrete, may be considered.

NOTE Apparent cohesion can be readily destroyed by the presence of groundwater and can only be relied upon for limited periods of time. Maintaining this apparent cohesion is vital to stability – ravelling can occur if the exposed face is allowed to dry out or if the saturation of the sand increases significantly due to infiltration of rainwater in the soil, for example. Observation under controlled condition trials can aid the understanding of the ability of a cut slope to remain stable during the period between excavation and completing face stabilization.

3.5.2.2 Grout loss

The use of grouted nails in clean coarse granular materials (coarse gravels, cobbles, rockfill) having little or no fine matrix might be problematic due to the risk of grout loss in voids; sanded grout mixes or grouts with gelling agents to increase viscosity and reduce flow may be considered as options in such situations. Failure to ensure a complete annulus of grout around the grout tendon might compromise pullout capacity or long-term durability.

3.5.2.3 Driving/drilling problems

The designer should take care in selecting a soil nail installation method appropriate for the grading of the soil. The presence of cobbles and boulders in a derived soil or unweathered layers in a residual soil may preclude the use of certain techniques.

3.6 Suitability of weak rocks for soil nailing

3.6.1 General

Steep slopes in weak or highly weathered rocks may be amenable to stabilization using soil nails. An appropriate method of nail installation, capable of penetrating inter-bedded strata of contrasting hardness, should be selected by the designer. Drilling to produce a rough bore surface generally provides an excellent bond. Provided the geological structure is favourably oriented (see Figure 11), weathered and weak rocks may be relied upon to offer good stand-up times for excavated faces. Particular guidance on the properties of specific rock types should be sought – examples of published guidance include CIRIA C570 [4], which provides detailed guidance on the behaviour of Mercia Mudstone, and CIRIA C574 [5], which gives advice on engineering in chalk.

3.6.2 Potential problems associated with soil nailing in weak rocks

3.6.2.1 Adverse bedding and jointing

NOTE Adverse jointing can result in stability problems in both the short and long term, particularly if joints contain low strength fine material on which blocks of intact rock can slide (Figure 11).

3.6.2.2 Effects of weathering/disturbance

NOTE Weathered rocks which are prone to degradation result in materials which exhibit the same advantages and disadvantages as the equivalent soil type.

3.6.2.3 Dissolution

*NOTE Calcareous rocks (chalk, limestone) are prone to dissolution. The risk of dissolution features in chalk is significant. Voids form by gradual dissolution of the chalk, particularly along joints and fissures where water flows readily. These features are generally infilled with material that has infiltrated from overlying recent deposits. Dissolution features are often relatively small (up to a few cubic metres in volume) so their presence might not be identified during routine site investigations (see **3.10**).*

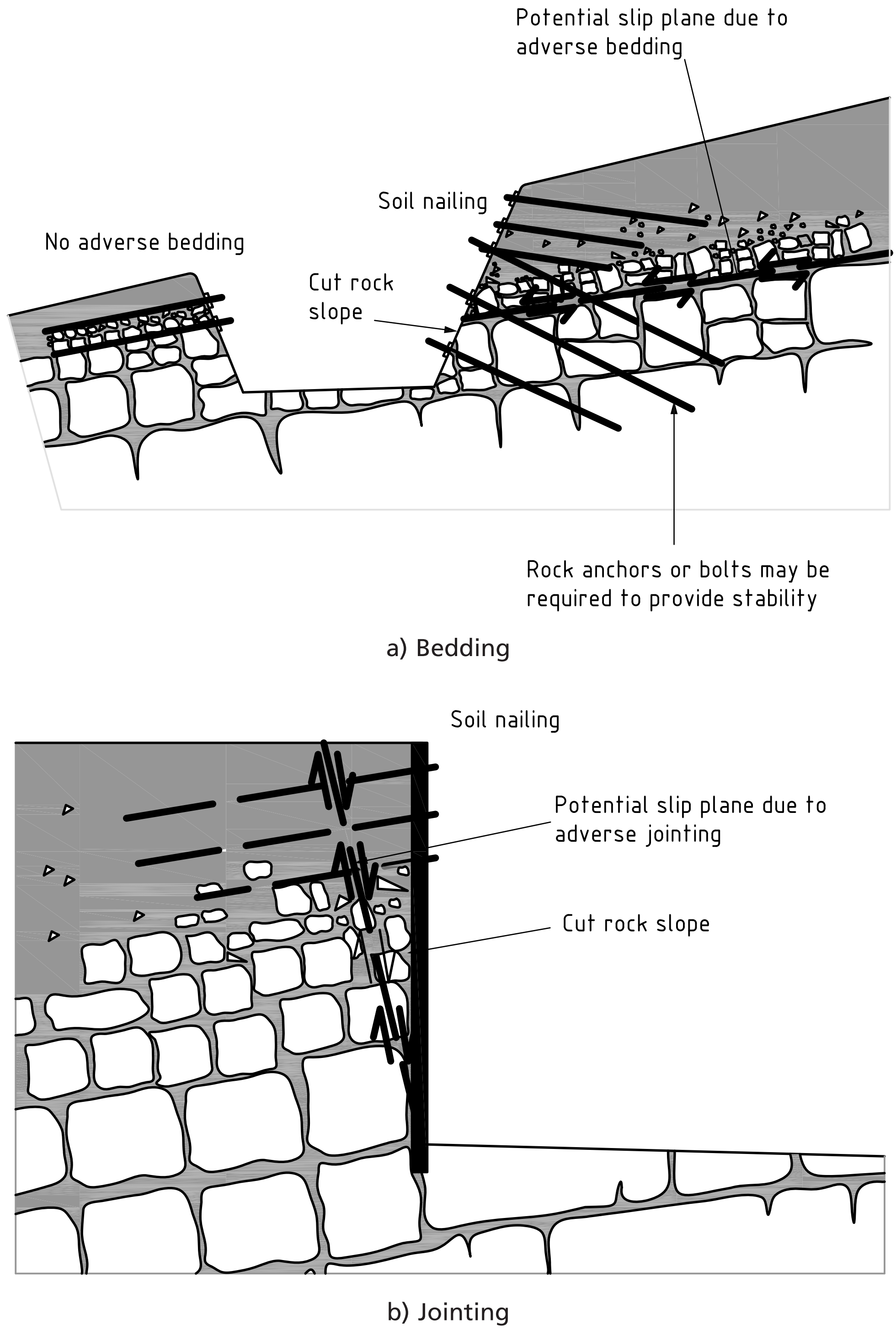
3.6.2.4 Other voids

NOTE Shallow voids resulting from mining operations such as bell-pits and adits are common where the coal measures outcrop in the Midlands and North of England (particularly in Lancashire and Yorkshire) and in Wales and Scotland. Localized quarry pits used to obtain rocks and minerals for construction and industrial processes can also result in hollow or infilled voids in the ground.

3.6.2.5 Aggressive ground conditions

*NOTE Sulfates are commonly found in the weathered zone of most mudstones and shales where sulfurous minerals such as pyrite and gypsum (in relatively high concentrations) have oxidized in the presence of air and water. Sulfates attack cementitious materials and can result in degradation of grouted nails and concrete facings (see **3.14** and **3.15**).*

Figure 11 **Adverse effects of jointing and bedding on cut slopes in weak or weathered rock**



Based on CIRIA C637 [1].

3.7 Suitability of fill for soil nailing

3.7.1 General

Fill material that has been graded, selected, placed and compacted in a controlled manner to give an essentially homogeneous material may be used to provide an ideal environment to install soil nails.

*COMMENTARY ON **3.7.1***

This is commonly referred to as engineered fill. Fills placed in more recent developments, such as reclamation works, highway and rail embankments tend to be engineered fills.

However, the majority of existing fill materials encountered in the UK do not fall into this category. More commonly, fills have historically been produced and placed without careful consideration of their properties and behaviour. They can have adverse characteristics including some or all of being:

a) variable in terms of nature and distribution of its constituents (heterogeneous);

b) soft or weak;

c) degradable;

d) loose due to poor placing and compaction;

e) poorly graded;

f) gas bearing; and/or

g) chemically aggressive.

Fill may be judged to be suitable for effective treatment by soil nailing broadly depending on:

1) the nature and grading of the constituent particles of the fill, which generally depends on the source of the fill material;

2) the density and strength of the fill resulting from the method of its placement and compaction and its age as well as the nature of the constituent particles; and

3) the potential for volume change due to external or internal mechanisms other than by gravity loading or imposed surface loads, for example collapse on wetting and chemical instability.

NOTE The Building Research Establishment report Building on fill: geotechnical aspects *(Charles and Watts [10]) gives detailed guidance on the properties and behaviour of fills commonly encountered in the UK and the geotechnical problems associated with them.*

3.7.2 Potential problems associated with soil nailing in engineered fills

Providing a fill has been placed and compacted in a controlled manner, its suitability for soil nailing may generally be judged depending on the characteristics of the source material. The advantages and limitations of using soil nailing in a natural granular deposit may also be assumed to apply to a granular fill. Likewise, cohesive fill may be assumed to exhibit similar advantages and limitations as a naturally occurring clay soil; however, the process of excavating, transporting, placing and compacting the material results in the following changes to the engineering properties of the fill material compared with its original intact state.

a) The excavation process can result in some mixing with other soils, particularly where material is excavated from a narrow or interbedded stratum. Excavation and handling can cause damage to the soil particles, resulting in a reduction in the grading size.

b) In granular soils, excavation destroys any apparent cohesion, which might not be re-established when the fill is placed and compacted. When placing and compacting rock fill and granular soils, it may not be possible to achieve the density of the previously intact material without breaking the grains. This can result in a lower bond strength than might be predicted from the properties of the natural source material. Safe heights and stand-up time for excavations might also be reduced.

c) Cohesive soils are excavated in blocks or lumps. When placed and compacted, the clay "lumps" deform, causing remoulding and weakening of the clay material. Voids remain between the lumps, although they might be filled to some extent by fines generated during excavation and handling. This can result in locally lower bond strengths. Grout loss into voids can also occur.

d) The homogeneity of the resulting fill structure will depend on the degree of quality control of the placing and compaction process. Where site practices differ from the recommended methods, greater variability of the fill density than expected can result.

It is therefore important to understand the characteristic properties of the fill in its present, as-placed state.

3.7.3 Potential problems associated with soil nailing in historic fills

*COMMENTARY ON **3.7.3***

The age of the fill is significant. In older fills, the design and construction techniques used at the time differed significantly from modern day standards and methods. Thus a fill might have been placed with an engineering purpose in mind, such as for a road or railway embankment, but might not have been selected, placed and compacted to modern standards. Railway embankments many decades old were often constructed with ash and other materials derived from natural sources. Embankments were often constructed using end or side-tipping of material and might contain loose or variably graded materials. There are many examples of the use of soil nailing in such materials as a means of temporary support or to improve stability of existing embankment slopes (e.g. Pedley [6]). However, the suitability of soil nailing to over-steepen existing old embankments will depend on the quality of the fill material at a particular site. Acceptability of long-term deformations of soil-nailed structures compared with a more rigid solution such as a piled wall will depend on the design requirements.

*Many historic fills in the Midlands and south of England are derived from high plasticity overconsolidated clays. As with naturally occurring clay soils of high plasticity, such fills are prone to large seasonal shrink/swell deformations due to cyclic change in pore pressures caused by vegetation (see **3.4.3.1**).*

Vegetation plays a profound role in the deformation of old embankments, especially if composed of high-plasticity clays (O'Brien et al, [7]). Few published data are available on the deformations that occur as a result of the stabilization of existing slopes by soil nailing.

The design of soil nailing schemes for high plasticity cohesive fills should be given particular care, and the technique might not be appropriate in some cases.

If post-construction deformations of embankments are required to be minimized primarily to improve serviceability, then it should be borne in mind that soil nailing might not be suitable on its own (see also **2.2.4**). Furthermore extra measures such as careful continuing vegetation management (see CIRIA C591 [8], or placement of suitably graded gravel capillary break layers (see Vaughan [9]) might be required.

3.7.4 Potential problems associated with soil nailing in non-engineered fills

NOTE Non-engineered fill is defined as a fill that has not been placed with a subsequent engineering application in view (Charles and Watts [10]).
Non-engineered fills are generally poorly compacted and frequently involve disposal of waste materials. They are often end-tipped, placed in variable thickness layers or hydraulically placed.

Variability generally means non-engineered fills are poorly suited and not recommended for soil nailing. Common types of non-engineered fill, the characteristics associated with them and whether they are recommended as suitable for soil nailing are presented in Table 2.

3.8 Effects of groundwater on soil nailing

For most soil types, soil nails should be installed from a dry excavation.

COMMENTARY ON ***3.8***

Groundwater can have an adverse effect on bond, durability of the nail and the integrity of the grout, stability and durability of the facing, stability of temporary excavations and the overall stability of slopes.

As discussed in the preceding sections, seepage of groundwater through the unsupported cut face can lead to instability of temporary excavations, particularly in predominantly granular soils or cohesive soils containing pockets of granular material. Running sand failures can occur. Seepage at the cut face can also cause problems in establishing both temporary and permanent facings (Figure 12). Swelling of clay soils in the presence of groundwater can result in softening of the clay and progressive slumping at the face in the long term.

Groundwater control measures (see **4.8**) may be used to provide a stable, dry face for the design life of the soil-nailed structure; however, groundwater at depth can still introduce difficulties, both in terms of installation, and long-term performance of the nails. Groundwater flow through the soil can lead to instability of nail bores, unless casings are employed.

If grouted nails are to be used, casings may also be required to prevent wash-out of grout and the resulting reduction in bond due to the formation of voids in the ground; preferential pathways for drainage might be set up along the soil/nail interface, leading to long-term reductions in bond strength in clay soils due to softening.

NOTE Groundwater acts as a means of transport for aggressive chemicals such as sulfates and carbon dioxide from on-site and adjacent sources, which can affect durability of the nails.

Consideration should also be given to the effects of long-term changes in groundwater conditions on the soil nailing scheme. These may include climate change, different rates of abstraction from aquifers or changes in groundwater regime due to the proposed scheme: a rise in groundwater is often associated with old mining areas where groundwater pumping of the mines has ceased and equilibrium of the natural water table is gradually being restored; and variations in pore water pressure might have an effect on the bond due to the resulting changes in the drained strength of the soil.

Table 2 **Principal types and suitability for soil nailing of non-engineered fill**

This is Table 4.1 from CIRIA C637 [1].

Type of non-engineered fill	Characteristics	Issues associated with soil nailing
Opencast mining fill	Natural material remaining after mining operations, sometimes mixed with imported material. Often placed without controlled compaction, resulting in variable density	• risk of instability of temporary cuttings where fill is loose • highly variable soil/nail bond strength due to variable grading and density • potential for collapse settlement due to rising groundwater or inundation by surface water • risk of coarse material obstructing drilling or driving.

Table 2 **Principal types and suitability for soil nailing of non-engineered fill**

This is Table 4.1 from CIRIA C637 [1].

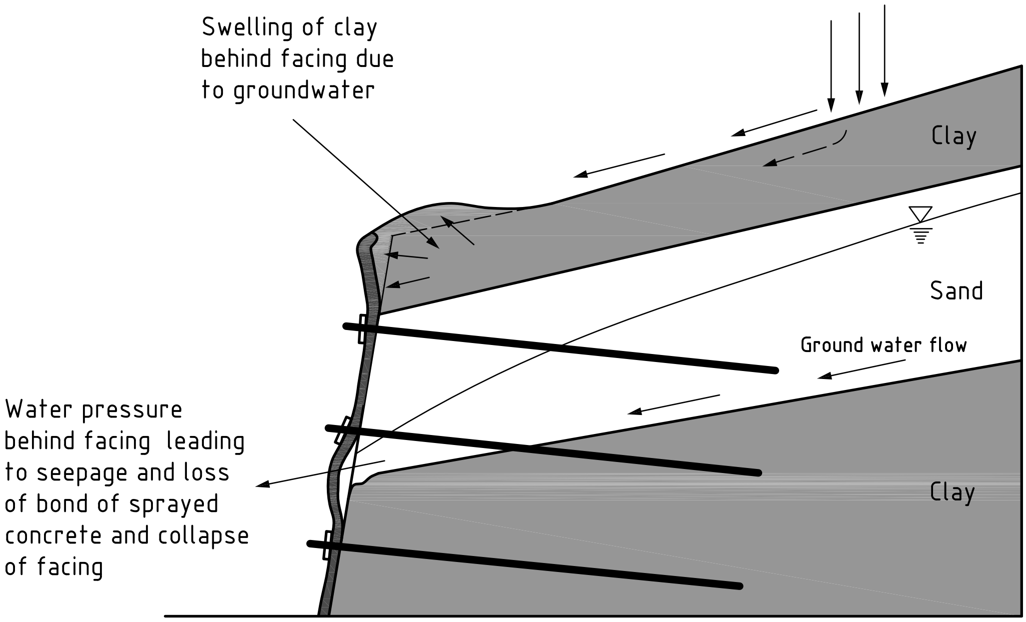
Type of non-engineered fill	Characteristics	Issues associated with soil nailing
Colliery spoil	Natural material adjacent to coal seams such as Carboniferous mudstone, siltstones and sandstones. Generally tipped in large spoil heaps in the coalfields of the Midlands and northern England, Scotland and south Wales.	• risk of instability of temporary cuttings where fill is loose • highly variable soil/nail bond strength due variable grading and density • potential for collapse settlement due to rising groundwater or inundation by surface water • risk of coarse material obstructing drilling or driving • slope stability problems such as the Aberfan disaster of 1966 have resulted in tighter control over placing and compaction of colliery spoil • aggressivity - risk of sulphate attack on soil nail grout from pyritic shales which oxidise slowly to sulfate and sulfuric acid. Oxidation can be rapidly accelerated by exposure of pyrites to air and groundwater due to extraction • risk of combustion particularly in older spoil due to the presence of coal.
Pulverized fuel ash (PFA)	Waste product of burning pulverised coal. Uniformly graded as a coarse silt to fine sand. High proportion of metal oxides. Commonly used as an engineered lightweight bulk fill in road construction, placed at a controlled moisture content (conditioned PFA). Also hydraulically placed as waste (lagoon PFA).	• short-term stability of cuttings is highly dependent on moisture content • frost susceptible – prone to deterioration at the face unless a frost protective facing is provided • toxic trace elements and dust also create safety risks. Lagoon PFA: • generally unsuitable for soil nailing • prone to settlement, slumping and liquefaction.
Industrial/ chemical waste	Broad range of material types and characteristics – may include hard coarse materials such as foundry sand, fine grained chemical waste slurries and silts placed in lagoons and biodegradable materials. Variable and often poorly compacted material	Suitability for soil nailing depends on nature of the fill but generally unsuitable due to: • risk of instability of temporary cuttings where fill is loose • highly variable soil/nail bond strength due variable grading and density • potential for collapse settlement due to rising groundwater or inundation by surface water • risk of coarse material obstructing drilling or driving • aggressivity – may contain a range of aggressive chemical constituents such as acids, sulfates and chlorides • toxic materials create health and safety risks.
Demolition rubble	Concrete and brick rubble from demolition of structures. Commonly used as oversite fill and backfill to walls in redeveloped sites. Often placed uncontrolled and unsorted. May contain timber, cloth, plastics, metal reinforcement, wire and larger obstructions	• risk of instability of temporary cuttings if fill is loose • variable soil/nail bond strength if variably graded and compacted • potential for collapse settlement due to rising groundwater or inundation by surface water • possible settlement due to any degradable fill materials • risk of coarse material obstructing drilling or driving • aggressivity – plasterboard waste: sulfates

Table 2 **Principal types and suitability for soil nailing of non-engineered fill**

This is Table 4.1 from CIRIA C637 [1].

Type of non-engineered fill	Characteristics	Issues associated with soil nailing
Domestic and commercial refuse	Domestic and commercial wastes due to the broad variety of loose degradable materials, such as organic material, paper, plastics and rags and the large voids between.	Soil nailing is unlikely to be suitable due to: • risk of instability of temporary cuttings if fill is loose • highly variable soil/nail bond strength due variable grading and density • potential for collapse settlement due to rising groundwater or inundation by surface water • high risk of settlement due to degradable materials • risk of flexible or coarse material obstructing drilling or driving • high risk of aggressive ground conditions.

Figure 12 **Effect of groundwater on wall facing**



Based on CIRIA C637 [1].

3.9 Effects of underlying geological features on soil nailing

It is important to understand any relevant geological features and structure and the effect that they can have on soil-nailed systems; some of these features relating to soils and weak rocks have been discussed in the preceding sections in the context of their effect on the installation and performance of the soil-nailed structure, such as pre-existing slip surfaces in clays (**3.4.3**) and solution features in chalk (**3.6.1**). However, features in underlying soil and rock strata should be regarded as equally important; while they might not affect the performance of the soil-nailed block itself, they can affect the overall stability of the scheme. They should be identified and, if necessary, additional measures implemented in order to provide overall stability of the scheme.

*COMMENTARY ON **3.9***

It is these underlying features that tend to provide the greatest risk to a soil nailing scheme, since a failure at depth creates a much larger area of influence at the surface than a shallow slip and therefore has the potential to cause greater damage and loss.

Existing slopes might have a history of instability or mask features that make them prone to slipping. Since the technique of soil nailing is most commonly used to remove material e.g. to form an over-steepened slope or wall, the process generally results in a reduction in vertical load on the underlying materials. Where such an excavation is made towards the toe of an existing slope, this results in removal of toe weighting and could trigger instability.

Evidence may be obtained from an initial desk study and walkover survey; previous slips are often apparent from the surface topography of the slope. Advice from an experienced geomorphologist may be sought and can be beneficial for assessing high risk areas. The presence of weak layers, laminations, slickensides or polished surfaces at depth should all be taken as indications of preferential slip planes within the underlying material; at depth, these features can only be observed from borehole samples but near the surface, they are more readily observed in trial pits or cuttings.

Slips can also occur along adverse bedding/joints in rock, particularly where surfaces contain fine weak material such as clay or silt generated by weathering or as infill from overlying deposits; the orientation of bedding planes and joints in the underlying rock in relation to possible slip planes should be considered. Weak layers (such as compressible clays or peat) at depth can result in unacceptable deformations of the soil-nailed structure and, in the limit, bearing capacity failure.

NOTE 1 Where soil nailing is provided to support a cutting, excavation results in reduction in overburden pressure on the underlying materials. In clay soils, this might result in movement due to swelling and a reduction in shear strength of the clay. The reduced overburden pressure can cause opening up of fissures and joints, leading to greater weathering and lower strengths along these surfaces.

The influence of these movements on both the stability and serviceability of the proposed scheme should be considered, and the designer should be aware that such mechanisms can take many years fully to develop.

*NOTE 2 Natural voids occurring at depth can give rise to subsidence. Examples include dissolution features in chalk (see **3.6.2.3**) or in limestone (karst topography) and buried alluvial channels. Man-made voids also occur as a result of mining operations. Even where mines have been filled in, poor compaction of the fill can result in a relatively higher void ratio and risk of collapse settlement. For a soil-nailed scheme above such features, there is a risk of large total and differential settlements, depending on the scale and position of the voids.*

Where a rock stratum occurs at a slope, cambering can occur, particularly where the underlying stratum is softer. Cambered blocks of rock can obscure the presence of a weaker layer at depth.

3.10 Site investigation

For guidance on general site investigations, reference should be made to BS EN 1997-2 and, inter alia, BS EN ISO 14688 (both parts), BS EN ISO 14689, BS EN ISO 22475 and BS EN ISO 22476 (all parts).

3.11 Soil-nailing related site investigation – Field trials

Probably the most valuable means of ground investigation that may be used to predict the likely behaviour of soil nails at a site is the use of a field trial (preliminary sacrificial soil nails are discussed in Section **6**). Field trials may also be used to highlight any construction problems in advance of the main contract works, such as problems with stability of temporary cuttings, capability of equipment to penetrate different strata, and problems with flush fluids. By identifying these problems at an early stage, they may be managed or eliminated, reducing the risk of contract delays.

3.12 Soil-nailing related site investigation – Chemical testing

Aggressive conditions in the ground can reduce the durability of the soil nails and laboratory testing should be carried out to quantify the aggressivity of the soil in which the nails will be installed and possible effects on drainage and facing materials. Consideration should also be given to aggressive conditions in groundwater percolating through the ground so aggressivity of adjacent and overlying materials should also be assessed. As well as assessing tendon durability, site investigation chemical testing should be done to assess the durability requirements for nail heads and facings.

*NOTE 1 Metallic nails are prone to increased rates of corrosion in the presence of chloride ions, acidic conditions, de-icing salts and in marine environments. Durability of composite nails can be affected by alkaline conditions and the presence of chlorides (see **4.6.5**).*

The approach given in this section may be applied to all sites where steel or fibre composite soil nails are to be used.

The chemical testing in the site investigation should include:

a) pH (of soil and groundwater);

b) water soluble sulfate content of soil and groundwater (with measurement of magnesium ions as a dependant option);

c) chloride ion content (of soil and groundwater);

d) soil resistivity; and

e) other tests required by BRE SD1 [3].

For brownfield sites, additional tests for nitrate ions may be required and, for pyritic sites, acid soluble sulfate tests should be carried out to determine the potential for sulfates forming when pyrites are exposed.

If adequately specified, such testing may all be carried out in one phase.

NOTE 2 Guidance on recommended test procedures is given in BS 1377-3:1990, TRL Report 447 [11], BRE BR279 [12] and BRE SD1 [3]. BRE SD1 provides detailed guidance on the chemical agents that are aggressive to concrete and grout and also provides recommended procedures for assessing the aggressive chemical environment for concrete class of the ground.

3.13 Preliminary assessment of degradation risk

There are many different adverse environmental conditions which should be considered because they can affect soil nails and these include:

a) relatively homogeneous soils with low salt content and benign water conditions;

b) partially saturated soils;

c) zones of fluctuating ground water levels;

d) strata with differing chemical composition and differences in water or gas content;

e) saturated clays with low oxygen content and high sulfate content;

f) sea water, or saline ground conditions;

g) contaminated soils;

h) humidity and ambient temperature.

It may therefore be deduced that the risk of degradation at a given location would be similarly variable and that degradation rates would be particular to a specific location. However, for the *preliminary* assessment of degradation risk, ground conditions may be simplified and divided into four classes:

1) undisturbed natural ground of pH 4.5 to 9;

2) disturbed natural ground or inert fill of pH 4.5 to 9;

3) acidic ground conditions with pH less than 4.5;

4) contaminated land.

When assessing the risk of degradation the term undisturbed ground may be defined as ground that has not been reworked by human intervention rather than geologically disturbed over a much longer period; the former conditions are often encountered in soil nailing.

Disturbed ground conditions may be defined as situations where filling has taken place (such as embankments or other areas of made ground), or where the ground has been excavated and then a component has been installed in the excavation and subsequently backfilled (such as for a cross-country pipeline or a culvert); disturbing the ground does not alter the degradation reactions that will occur but it makes the reactions, particularly those resulting from the transport of oxygen, more likely to occur and therefore degradation rates may be assumed to be much higher than in undisturbed ground.

Contaminated ground conditions can pose a higher degradation risk than natural ground or inert fill; where such conditions are encountered, a detailed degradation assessment should be undertaken similar to that used for disturbed conditions. In addition the test programme should identify the presence of any contaminants at the site.

The history and utility records of the site should be reviewed when assessing the degradation risk of a site. Leakage from sewers or effluents from chemical emitters can make soils at a site aggressive, and the existing and likely future effects of leakage should be considered.

NOTE A particular example of this risk reported in Hong Kong is leakage from salt water mains (GEO [14]).

The most important parameter in understanding degradation in the ground is the pH of the soil or ground water; this may only be assessed by on site testing (as discussed above). The results may be classified to give a preliminary degradation assessment of the ground as follows:

- pH greater than 9 indicates alkaline ground conditions where degradation risk in natural ground is low. However, high pH values may indicate a contaminated land site – although this should have been established from the desk study.
- pH of between 4.5 and 9 indicates near neutral conditions in which degradation rates are controlled by cathodic reduction of oxygen. Corrosion

risks for steel can be assessed for preliminary purposes using (conservative) published corrosion rate data (see Table 3 in this standard and Figure 9.8 in CIRIA C637 [1]).

- pH less than 4.5 indicates acidic or contaminated ground conditions and corrosion reactions (for steel) will be controlled by hydrogen evolution. In such conditions specialist advice should be obtained on possible degradation rates and methods of prevention.

Table 3 **Typical corrosion rates for uncoated steel in undisturbed ground conditions**

From Corus, 2003; this is also CIRIA C637 [1], Table 9.10.

Exposure condition	**Application for soil nailing**	**Mean corrosion rate (microns/year)**
Atmospheric	Nail head, head plate and proximal end of nail tendon	35
Undisturbed ground (but not if it includes ash, clinker or coke)	Most of nail tendon, except proximal end	15 (maximum)
Marine splash zone	Nail head, head plate and proximal end of nail tendon when used in marine works	45

3.14 Detailed assessment of degradation risk for buried components

The degradation risk of the ground, and groundwater where present, may be assessed using the method presented in BS EN 14490:2010, **B.3.4.2**, Table B.1 and Table B.2.

3.15 Detailed assessment of degradation risk for exposed components and surfaces

Exposed components and surfaces (i.e. nail head, head plate, proximal end of nail tendon and facing) should be assessed for degradation risk according to the aggressivity of the ground and groundwater where present, as described in **5.8**, and also according to the corrosivity of the atmosphere using the criteria in Table 4 and as described in the following. For soil nailing in highway environments, consideration should also be given to the effect on exposed surfaces of chloride ions from road salt.

The harshness of the atmospheric environment may be determined from the map *Relative values of acid deposition in the United Kingdom 1986 – 1991* published by ADAS [15]. The corrosivity (acid deposition) value should be taken as the average of the values indicated by the chart grid square in which the proposed slope or wall is sited and the three nearest adjoining squares. The atmospheric environment for the exposed surfaces should be determined from this corrosivity value using Table 4. However, any other significant environmental factors concerning the location should also be taken into account, such as wind blown salt near the coast or pollution from nearby industrial plant (the typical corrosion rate for atmospheric exposure given in Table 3 is for non-aggressive conditions). For aggressive and highly aggressive conditions, specialist advice should be sought.

Table 4 **Classification of surfaces exposed to the atmospheric environment**

From Highways Agency (2001).

Average corrosivity value (acid deposition value from ADAS, 1991 [15] map)	**Classification of the atmospheric environment**
≤ 2	Non-aggressive
> 2 but ≤ 4	Aggressive
> 4	Highly aggressive

Section 4 : Basis for design

4.1 Design method

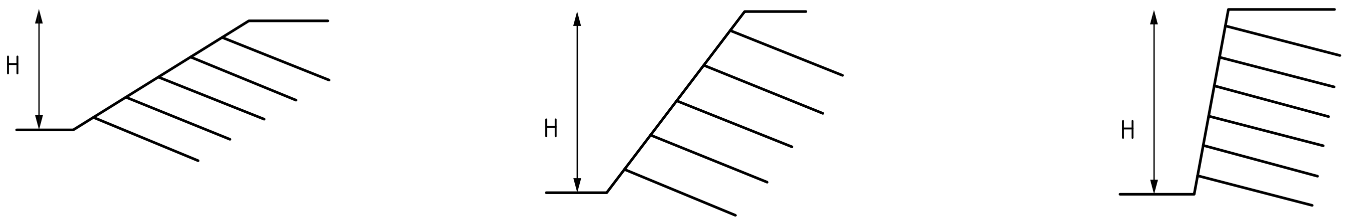
4.1.1 General philosophy

The design aim should be to arrive at a soil nailing layout which provides an adequate reserve of safety against failure for any postulated failure mechanism.

This standard is not prescriptive about how the soil nailing layout should be derived. A suitable soil nailing layout may be arrived at via an iterative, “trial and error” approach. Prescriptive methods of design or design charts may also be used provided that the final design layout meets the recommendations of **4.1.2** to **4.1.5**. Figure 13 gives typical soil nailing geometries for a range of slope angles that may be used as a first stage of design when using the “trial and error approach” for design.

NOTE Useful guidance on the choice of a preliminary nail layout and design sequence is provided in CIRIA C637 [1].

Figure 13 **Typical dimensions of soil nailing applications based on slope angle**



Slope angle (to horizontal)	up to 45°	45° to 60°	60° to 90°
Nail length (L/H)	0.5 to 2.0	0.5 to 1.5	0.5 to 1.2
Nail spacing – vertical – horizontal	1.5 m to 3.0 m 1.5 m to 3.0 m	1.0 m to 2.0 m 1.0 m to 2.0 m	0.75 m to 1.5 m 0.5 m to 2.0 m
Facing	Generally *soft* non-structural, for erosion control but with enlarged plates at nail head	Typically *flexible* facings used that perform a structural role in maintaining stability	Typically *hard* facings used that perform a structural role in maintaining stability and permit high forces to be mobilized at the facing connection.

NOTE May be used as a first design assessment tool. Actual nail spacings, lengths and facing requirements have to be determined by analysis.

4.1.2 Principles of design

The design of soil nailed systems should follow limit state principles and be compatible with the principles of BS EN 1997-1:2004. In situations where a conflict arises between the two documents, this document should govern.

Only the tensile (axial) capacity of the soil nails should be considered. The contribution of nail bending and shear resistance are second-order effects only seen at high deformation levels and should be conservatively ignored.

NOTE In contrast to reinforced soil, soil nailing construction generally starts at the top of the slope. The upper nails are loaded first and the pattern of nail loading develops as excavation proceeds downwards and more nails are inserted. Nail loads will continue to develop for a period after the end of excavation as pore pressures adjust and deformation takes place. The behaviour of the soil is modified by the presence of the nails in a complex fashion.

Numerical analysis may be considered for design, in some situations, in order to capture the full complexity of the soil nail system and provide realistic slope deformations (see **4.4**). However, for most practical purposes it should be sufficient to assess stability using 2-D limit equilibrium analysis methods and to assess serviceability using empirical means.

The observational method, in which the design of the soil nailed slope is reviewed during construction, may be used, but this is not discussed further in this standard.

4.1.3 Limit state approach

The design method should adopt a limit state approach; a limit state is reached when the nailed slope fails to satisfy any one of its performance criteria. Both ultimate limit states (ULS) and serviceability limit states (SLS) should be considered.

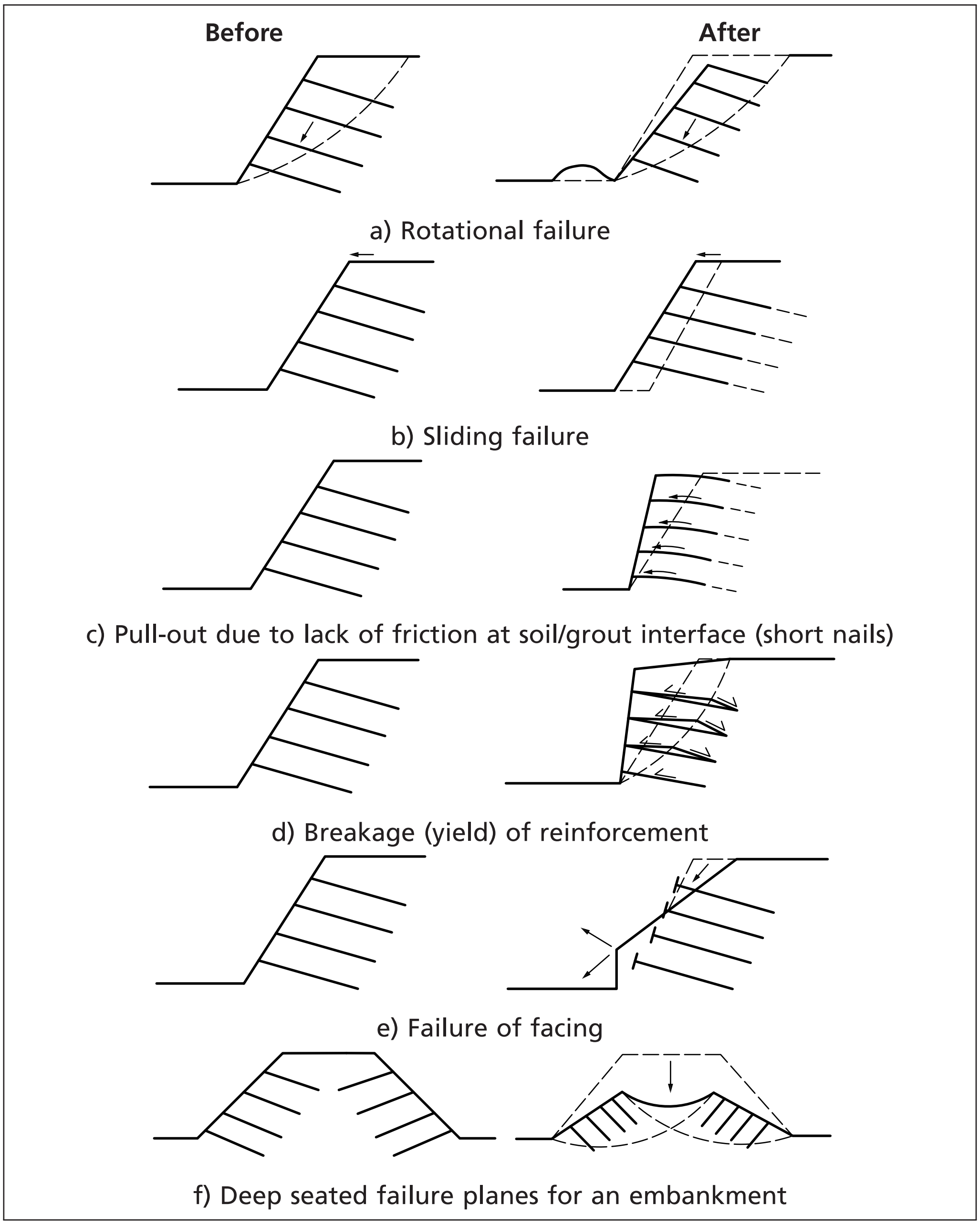
NOTE 1 Ultimate limit states are associated with instability, collapse or structural failure resulting from the disturbing forces exceeding the available restoring forces. Serviceability limit states are reached through excessive deformation, or when the serviceability of the structure is otherwise impaired.

The relevant modes of ULS and SLS that should be considered in soil nailing are shown in Figure 14; the assessment of ULS is discussed in this clause and the assessment of SLS is discussed in Section **5**.

Ultimate limit state modes of failure should, in the first instance, be assessed using limit equilibrium analysis. In limit equilibrium analysis it may be assumed that the structural elements of the slope (in this case the soil, the nails and the facing) each mobilize their respective allowable design strengths simultaneously. Either a force or a moment balance should be considered to determine stability, as described in **4.2**.

NOTE 2 In practice, the behaviour of the soil is likely to be modified by the presence of the nails in a complex fashion involving rotation of principal stresses, block behaviour, load redistribution and redundancy. This is conservatively ignored in limit equilibrium analysis.

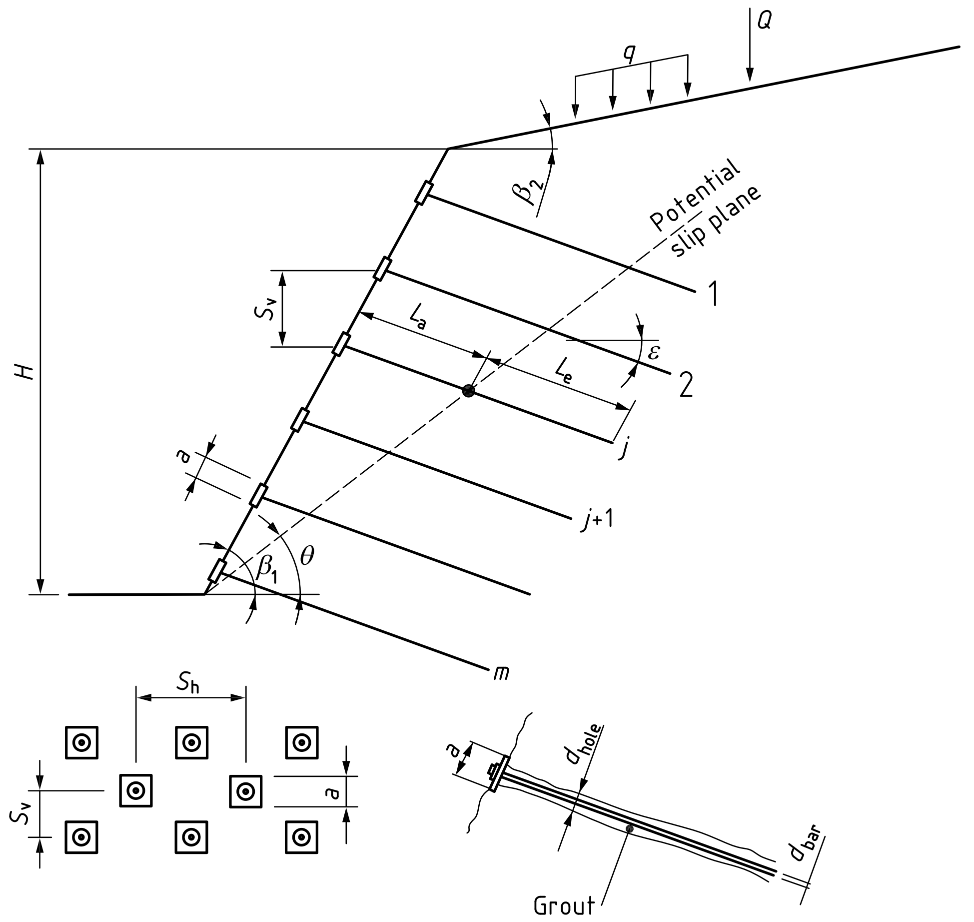
Figure 14 **Relevant modes of ultimate and serviceability limit states**



4.1.4 Geometry of the soil nail zone

The geometry and notation used to describe a soil nailed slope is defined in Figure 15; the principal variables controlling the soil nailing layout are the length of the nails L, the nail declination ε, the vertical and horizontal nail spacings S_v and S_h, the nail hole diameter d_{hole}, the bar diameter d_{bar}, and the size of the nail plates a. The slope geometry may be assumed to be controlled by the slope height H, and front face angles β_1 and β_2. The slope may consist of several parts (e.g. a steep slope followed by a shallower slope, followed by a crest).

Figure 15 **Geometry and dimensions of a soil nailed slope**



Nail length, $L_{nj} = L_{aj} + L_{ej}$

4.1.5 Partial factors

In keeping with BS EN 1997-1:2004 a partial factor approach should be adopted in order to minimize the risk of a soil nailed structure attaining a limit state. Table 5 summarizes the appropriate partial factors that should be applied to actions, material properties and soil nail resistances; actions are sets of forces or imposed deformations applied to the structure, and can be either favourable or unfavourable.

The design value of an action should be obtained by multiplying the representative value by the partial factor. The design value of a material property should be obtained by dividing the characteristic value by the partial factor. Soil nail resistances arise from the bond stress acting at the interface of the nail and the soil, and from the strength of the nail tendon; the design value of a soil nail resistance should be obtained by dividing the characteristic values of bond stress and nail tendon strength by the partial factor.

Table 5 provides two sets of partial factors (1 and 2) and the designer should ensure that an ultimate limit state will not occur with either set.

*NOTE It is assumed that the designer will design the structure for the worst credible geometry, taking into account the tolerances stated in **2.4.2**, therefore no partial factor relating to geometry is included in Table 5.*

The partial factors given in Table 5 do not explicitly take account of whether the works are of a temporary or permanent nature; this should instead be reflected in the selection of appropriate characteristic values for the material properties and characteristic soil nail resistances.

The characteristic value of a material property should be selected as a cautious estimate of the value affecting the occurrence of the limit state. Further guidance may be found in BS EN 1997-1:2004, **2.4.5.2**(10) and **2.4.5.2**(11). The representative value of an action should be the characteristic value, or an accompanying value as defined in BS EN 1990 (all parts).

The selection of a partial factor for an action in Table 1 should depend on whether the action has a stabilizing effect (favourable) or a destabilizing effect (unfavourable).

In certain situations, where an action may be considered as being both stabilizing and destabilizing within the same calculation (e.g. soil weight contributing to both the driving and resisting moments in an effective stress analysis), only a single partial factor need be applied to that action per calculation. (This is consistent with the Note to BS EN 1997-1:2004, **2.4.2** and is referred to as the single source principle.)

Pore water pressures should be assumed to be the worst credible under normal operations, that is to say the most onerous that is considered reasonably possible during the design lifetime. There may be an element of engineering judgement involved in this assessment. Maintenance of drainage schemes (if any) and the possible risk of their failure should be taken into account.

Where soil nails are being used to remediate a failed slope or where relic shear surfaces are known to exist, relevant shear strength parameters should be used on existing failure surfaces, including residual values where appropriate. Alternative partial factors for soil nail design to those given in Table 5 may then be considered [this is consistent with BS EN 1997-1:2004, **11.5.1**(8)].

Progressive failure should be considered in strain softening, brittle soils (e.g. high plasticity clays).

Table 5 **Partial factors for soil nail design**

Design values are to be obtained by multiplying the representative values of the actions, and dividing the characteristic values of the material properties and soil nail resistances, by the following partial factors. A)

			Set 1	Set 2
Actions	Self-weight of soil, W	dst B)	$\gamma_g = 1.35$	$\gamma_g = 1.0$
		stb B)	$\gamma_g = 1.0$	$\gamma_g = 1.0$
	Permanent surcharge, q_p	dst	$\gamma_{qp} = 1.35$	$\gamma_{qp} = 1.0$
		stb	$\gamma_{qp} = 1.0$	$\gamma_{qp} = 1.0$
	Variable surcharge, q_v	dst	$\gamma_{qv} = 1.5$	$\gamma_{qv} = 1.3$
		stb	$\gamma_{qv} = 0$	$\gamma_{qv} = 0$
	Ground-water pressure, u	dst	$\gamma_u = 1.0$	$\gamma_u = 1.0$
		stb	$\gamma_u = 1.0$	$\gamma_u = 1.0$
Material properties	$\tan \varphi'_k$		$\gamma_{\tan\varphi'} = 1.0$	$\gamma_{\tan\varphi'} = 1.3$
	c'_k		$\gamma_{c'} = 1.0$	$\gamma_{c'} = 1.3$
	cu_k		$\gamma_{cu} = 1.0$	$\gamma_{cu} = 1.4$
	γ_k		$\gamma_\gamma = 1.0$	$\gamma_\gamma = 1.0$
Soil nail resistances C)	Bond stress D), τ_{bk}	Empirical	$\gamma_{\tau b} = 1.1$	$\gamma_{\tau b} = 1.5$
		Effective stress E)	$\gamma_{\tau b} = 1.1$	$\gamma_{\tau b} = 1.5$
		Total stress E)	$\gamma_{\tau b} = 1.1$	$\gamma_{\tau b} = 1.5$
		Pull-out tests F)	$\gamma_{\tau b} = 1.1 – 1.7$	$\gamma_{\tau b} = 1.5 – 2.25$
	Tendon strength, T_k		$\gamma_s = 1.0$	$\gamma_s = 1.15$ for steel
Model factor	Applied to the effect of unfavourable actions G) (e.g. to $M_{driving}$ in the case of Bishop's slip circles)		γ_{Sd}	γ_{Sd}

A) In the case of abnormal risk or unusual or exceptionally difficult ground or loading conditions (Geotechnical Category 3 structures as defined in BS EN 1997-1:2004), all non-unity values in the above table should be increased by 10%.

B) stb = stabilizing; dst = destabilizing.

C) The factors shown for soil [C1) nail (C1] resistances are a summary of the fuller explanation given in Table 6.

D) Refer to **4.3** for appropriate choice of characteristic bond stress to reflect whether nails are temporary or permanent.

E) Calculated on characteristic soil properties and ignoring q_v.

F) See **4.3.6**.

G) The values of γ_{Sd} will be dependent on the calculation method used. γ_{Sd} may be set to unity for Bishop's simplified method of slices, or for calculations performed with the two-part wedge mechanism assuming a vertical inter-wedge boundary with a friction value not exceeding $0.5\varphi'$. For other methods of analysis a safe value of γ_{Sd} should be determined through calibration.

4.2 Analysis of stability

4.2.1 Internal stability

4.2.1.1 General

NOTE Internal stability concerns the assessment of mechanisms which are either fully contained within the soil nailed zone or pass through some part of it (the latter is also sometimes referred to as "compound stability").

The design aim should be to arrive at a soil nailing layout and facing design for which the factored resisting forces exceed the factored driving forces for any postulated failure mechanism, both in the short term and the long term and for each stage of construction. Additionally each nail and facing element should be capable of resisting the local stresses acting on it.

The relevant ULS modes of failure for internal stability analyses should be:

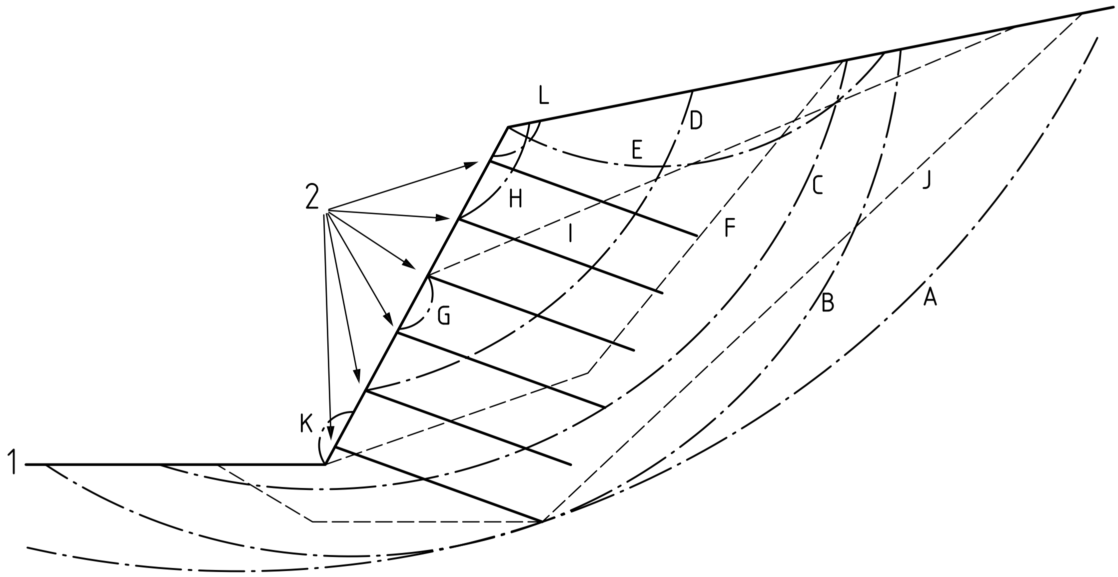
a) rotational failure either partially or totally through the soil nail block involving breakage and/or pull-out of soil nails, and/or failure of the facing, e.g. C, D or H in Figure 16 (**4.2.1.2**);

b) translational failure (i.e. forward sliding) through the soil-nailed zone involving either breakage and/or pull-out of soil nails, and/or failure of the facing, e.g. F or I in Figure 16 (**4.2.1.3**);

c) local overstressing of nails, e.g. K in Figure 16 (**4.2.1.4**); and

d) local front face failure, e.g. G or L in Figure 16 (**4.7**).

Figure 16 **Ultimate limit state modes of failure**



Key

A, B, E External rotational failures

C, D, H Internal rotational failures

F, I Internal translational failures

1 Surface

G, L Internal, local front face failure

J External translational failure

K Internal, local overstressing of nails

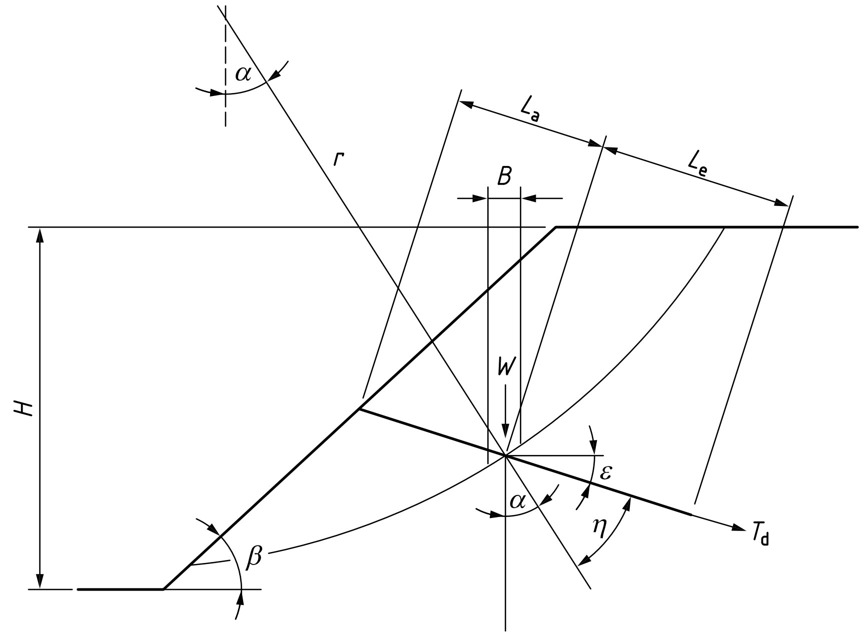
2 Soil nails

4.2.1.2 Bishop's simplified method of slices

NOTE The most common method of limit equilibrium analysis for the purposes of assessing rotational stability, available in most commercial design software packages, is Bishop's simplified slip circle method (Bishop [21], Bishop and Morgenstern [22]).

Method: Divide the free body into a number of slices with vertical sides as shown on Figure 17. For Bishop's simplified method, effectively assume that the inter-slice shear forces are zero (i.e. frictionless and cohesionless) in order for the forces acting on the base of each slice to become determinate.

Figure 17 **Slip circle method of slices**



Key

B	width of slice (m)	α	angle as defined by Bishop (1955) [21]
H	height of slope (m)	β	angle of slope
L_a	length of nail to base of slice (m)	ε	nail declination
L_e	length of embedment zone (m)	η	angle between normal to slip plane and nail = 90 ° – α – ε
r	radius of slip circle (m)		
T_d	design tension in nail at point (kN)		

Calculate moments about the centre of rotation using factored values for each of the variables as set out as follows, and compare the driving moment $M_{driving}$ and the resisting moment $M_{resisting}$. This approach conforms to that set out in BS EN 1997-1:2004, which requires the following inequality to be verified:

$$E_d \le R_d$$

where E_d is the design value of the effects of actions and R_d is the design resistance.

Carry out a comprehensive search for all potential rotational failure surfaces passing through the slope. Check each stage of construction.

Design. The soil nail design should evolve by an iterative process of checking and refinement until for all possible slip circles it can be shown that:

$$\gamma_{Sd} M_{driving} \le M_{resisting}$$

where,

$$M_{driving} = \sum_{i=1}^{n} \left[W_i \gamma_g + \left(q_{pi} \gamma_{qp} + q_{vi} \gamma_{qv} \right) B_i \right] r \sin \alpha_i$$

$$M_{resisting} = M_{soil} + M_{nails}$$

$$M_{\text{soil}} = \sum_{i=1}^{n} \left\{ \frac{c'_{\text{d}} B_i + \left[W_i \gamma_{\text{g}} + \left(q_{\text{p}i} \gamma_{\text{qp}} + q_{\text{v}i} \gamma_{\text{qv}} \right) B_i - u_i \gamma_{\text{u}} B_i \right] \tan\varphi'_{\text{d}}}{\cos\alpha_i + \sin\alpha_i \tan\varphi'_{\text{d}}} \right\} r$$

$$M_{\text{nails}} = \sum_{j=1}^{m} \left[T_{dj} \cos\left(\alpha_j + \varepsilon_j\right) + \frac{T_{\text{d}j} \sin\varepsilon_j \tan\varphi'_{\text{d}}}{\cos\alpha_j + \sin\alpha_j \tan\varphi'_{\text{d}}} \right] \frac{r}{s_{\text{h}j}}$$

and

γ_{Sd} is a model factor;

where, values for all the partial factors γ featuring are given in Table 5, and:

r, B, α, and ε are geometrical parameters as shown on Figure 17;

tan φ'_{d} is the design value of the tangent of the angle of shearing resistance (tan $\varphi'_{\text{k}}/\gamma_{\tan\varphi'}$) and c'_{d} is the design value of the cohesion intercept in terms of effective stress ($c'_{\text{k}}/\gamma_{c'}$) in kN/m²;

W is the unfactored self weight of the slice;

q_{p} and q_{v} are the permanent and variable unfactored surcharge pressures acting on the surface of the slice respectively;

u is the unfactored pore pressure acting at the base of the slice;

NOTE 1 Each of the actions W, q_p, q_v and u are characteristic values and are multiplied by their corresponding partial factors in the above equations.

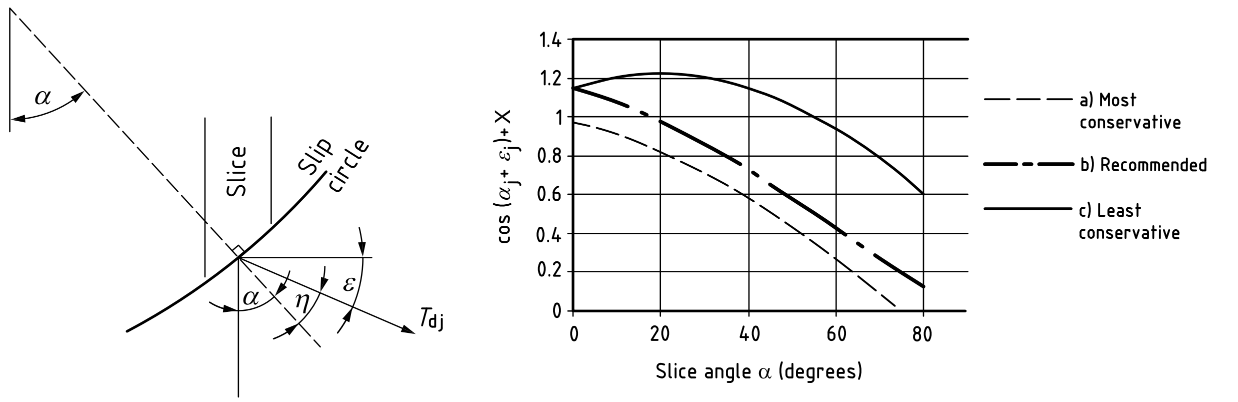
T_{d} is the design nail force (in kN), which is the lesser of the nail's factored characteristic pull-out capacity ($\tau_{\text{bk}} \pi d_{\text{hole}} L_{\text{e}}/\gamma_{\tau\text{b}}$) and its factored characteristic bar strength ($f_{\text{yk}} A_{\text{s,nom}}/\gamma_{\text{s}}$) (see **4.3** and **4.5** respectively); and

S_{h} is the horizontal spacing of the nails.

M_{nails} should only be calculated for slices where a nail intersects the slip surface (and for these slices α_{i} may be assumed to be equal α_{j}).

NOTE 2 The second term in the square bracket expression for M_{nails} assumes that each nail contributes to the frictional shear resistance acting on the base of the slice upon which it acts (obtained by including the nail forces when determining slice vertical equilibrium). There is some variance in approach on this point for different methods. Some calculation methods conservatively ignore this effect by setting the second term to zero. Other calculation methods (e.g. CIRIA C637 [1]) propose replacing the second term with the simpler but less conservative expression T_{dj} sin ($\alpha_j + \varepsilon_j$) tan φ'_d. The expression for M_{nails} set out above represents a midway course, as demonstrated by Figure 18.

Figure 18 **Methods of resolving nail force and degree of conservatism**



Graphs correspond to ε = 15°, [C1› φ'_{d} ‹C1] = 35°

$$M_{\mathrm{nail}j} = \left[\cos\left(\alpha_j + \varepsilon_j\right) + \mathrm{X}\right]\frac{T_{\mathrm{d}j}r}{S_{\mathrm{h}j}}$$

There are three options for X which may be considered:

a) $$\mathrm{X} = 0$$

[most conservative]

b) $$\mathrm{X} = \frac{\sin\varepsilon_j \tan\varphi'_{\mathrm{d}}}{\cos\alpha_j + \sin\alpha_j \tan\varphi'_{\mathrm{d}}}$$

[recommended]

c) $$\mathrm{X} = \sin\left(\alpha_j + \varepsilon_j\right)\tan\varphi'_{\mathrm{d}}$$

[least conservative]

NOTE 3 In practice, commercial software will normally do all of the above for the user, provided that the user inputs into the program the relevant characteristic values and appropriate partial factors, and that the user then looks for an overall factor of safety of more than one (or more than γ_{Sd} if γ_{Sd} is not equal to one). Alternatively the user may choose to input design values directly into the program and set the partial factors to unity.

The value of the model factor γ_{Sd} may be taken to be unity when Bishop's simplified method of slices is used. Alternative limit equilibrium methods of analysis may also be adopted (e.g. two-part wedge, or log spiral method), but it should be recognized that these other methods of analysis are likely to predict different reserves of safety for an identical problem. An appropriate value of γ_{Sd} should therefore be chosen to reflect this.

4.2.1.3 The two-part wedge mechanism

*COMMENTARY ON **4.2.1.3***

*Adequate bi-linear approximations of many curved failure surfaces may be achieved using the two-part wedge mechanism. The two-part wedge method may therefore be used as an alternative to (or as a convenient method of checking) Bishop's slip circle method for the analysis of internal stability rotational-type failure mechanisms (**4.2.1.2**) which exit the slope at its toe or on its front face.*

The two-part wedge mechanism involves less calculation than Bishop's method of slices. It is therefore simpler to use and may be readily formulated using a desk-top spreadsheet program. It has the advantage of being fully explicit and the calculations performed being reassuringly understandable.

The two-part wedge is also particularly well suited to analysing forward sliding across a planar surface. It cannot however be used to analyse deep seated rotational failures which outcrop beyond the toe of the slope.

Method. Examine the stability of each mechanism in terms of a force balance across the inter-wedge boundary, rather than a moment balance as used earlier for Bishop's slip circles; the principal requirement is that the factored resisting force provided by the lower wedge normal to the interwedge boundary exceeds the factored driving force from the upper wedge.

Using geometry and variables defined in Figure 19, take the inter-wedge boundary as vertical and having a reduced inter-wedge friction angle compared to the parent soil (typically half or less, zero being the simplest assumption).

Design. The following simple inequality should be satisfied by the design:

$$\gamma_{Sd} N_{21} \le N_{12}$$

where

$$N_{21} = \left\{ \frac{\left(W_1\gamma_g + Q_{p1}\gamma_{qp} + Q_{v1}\gamma_{qv}\right)\left(\sin\theta_1 - \cos\theta_1 \tan\varphi'_{d1}\right)}{\cos\theta_1 + \sin\theta_1 \tan\varphi'_{d1}} \right\} + \left\{ \frac{-\left[\sum \frac{T_{d1}}{S_h}\right]\left[\cos(\theta_1+\varepsilon) + \sin(\theta_1+\varepsilon)\tan\varphi'_{d1}\right] + U_1 \tan\varphi'_{d1} - K_{d1}}{\cos\theta_1 + \sin\theta_1 \tan\varphi'_{d1}} \right\}$$

$$N_{12} = \left\{ \frac{-\left(W_2\gamma_g + Q_{p2}\gamma_{qp} + Q_{v2}\gamma_{qv}\right)\left(\sin\theta_2 - \cos\theta_2 \tan\varphi'_{d2}\right)}{\cos\theta_2 + \sin\theta_2 \tan\varphi'_{d2}} \right\} + \left\{ \frac{\left[\sum \frac{T_{d2}}{S_h}\right]\left[\cos(\theta_2+\varepsilon) + \sin(\theta_2+\varepsilon)\tan\varphi'_{d2}\right] - U_2 \tan\varphi'_{d2} - K_{d2}}{\cos\theta_2 + \sin\theta_2 \tan\varphi'_{d2}} \right\}$$

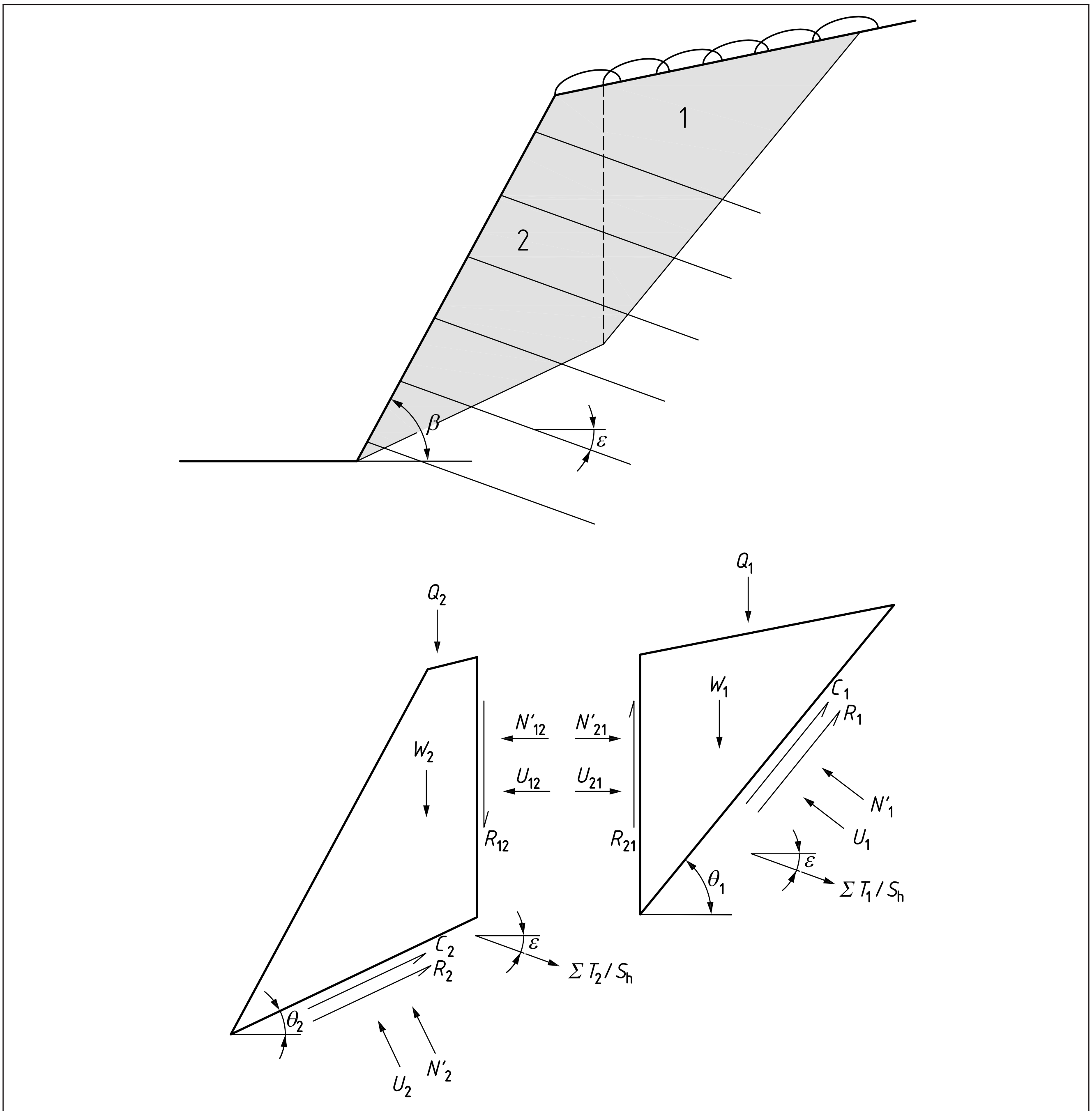
NOTE These equations correspond to the simplified case of zero friction on the inter-wedge boundary. Equations describing the more general case of a non-zero friction on the inter-wedge boundary may be found in HA68/94 [23] Appendix A (para A7).

At the time of publication it is not thought that there has been a comprehensive study comparing the two-part wedge method and Bishop's simple method of slices for soil nailed slopes incorporating different inter-wedge friction angles and inclinations. Some useful general guidance is provided in Appendix A of HA68/94 [23] on the effect of inter-wedge friction angle.

The assumption of a frictionless inter-wedge boundary is always likely to be conservative compared to Bishop's simple method of slices (and will also simplify the equations), while the assumption of full friction on the inter-wedge boundary is always likely to be unconservative.

The value of the model factor γ_{Sd} may be taken as unity for two-part wedge mechanisms adopting a vertical inter-wedge boundary with an inter-wedge friction angle not exceeding $0.5\varphi'$ (see Table 5). Two-part wedge mechanisms with a non-vertical inter-wedge boundary may also be considered, for example aligning with the rear of the soil nailed zone and mobilizing full friction. However such designs are unlikely to have the same reserve of safety as the normal method and an appropriately higher value of γ_{Sd} should be adopted for such cases.

Figure 19 **Two-part wedge**



4.2.1.4 Limitation on nail spacings

In order to prevent overstressing of nails locally and the risk of progressive failure, nail spacings should be limited such that each nail is capable of withstanding the loads placed upon it locally.

In uniform ground vertical spacings between nail rows are traditionally kept constant with depth, or decreasing with depth in stages as appropriate; variations to this general trend may be appropriate in layered ground of varying strength. The vertical spacing between nail rows should be limited in any case to 2 m in intermediate slopes and 1.5 m in steep slopes.

NOTE If performed thoroughly the upper bound methods described in ***4.2.1.2*** *and* ***4.2.1.3*** *ought on their own to be sufficient to ensure that vertical spacings between nail rows are not excessive and that each row of nails is able to withstand the loads placed upon it. This is because these methods will (or ought to) include, within their comprehensive search of slip circles (or two-part wedge, or other chosen mechanism), mechanisms which daylight on the front face just above each nail row.*

An alternative method that may be adopted to ensure that each row of nails is able to withstand the loads placed upon it locally is to adopt a lower bound “stress state” approach (as for example embodied in HA68/94 [23]). According to this approach, the maximum nail spacing at any point may determined by the expression:

$$S_v = \frac{T_d}{K \gamma z S_h}$$

where

T_d is the design nail strength (in kN);

K is the earth pressure co-efficient in terms of total stress and factored soil strength (refer to HA 68/94 [23] for evaluation);

γ is the weight density (in kN/m^3);

z is the depth to mid-point between the row of nails in question and the next row below (m);

S_v, S_h are the vertical and horizontal nail spacings (m).

4.2.2 External stability

4.2.2.1 General

COMMENTARY ON ***4.2.2.1***

External stability analysis concerns the assessment of mechanisms which affect the stability of the soil nail zone but do not intersect it at any point (also referred to as “global stability”).

The relevant ULS modes of failure for external stability analyses should be:

a) deep seated rotational failure, e.g. A or B in Figure 16; and

b) translational failure (i.e. forward sliding) beneath the soil nail zone, e.g. J in Figure 16.

From an analytical point of view the external stability check for deep seated rotational failure may simply be seen as an extension to the internal stability check (**4.2.1.2**) except that no nail forces are involved. Similarly, the external stability check for translational failure (i.e. forward sliding) may be seen as an extension to the internal stability check (**4.2.1.3**).

The same methods of analysis described in **4.2.1.2** and **4.2.1.3** may therefore be employed for these external stability checks.

While there is little or no distinction from an analytical point of view between internal and external stability, an important distinction may be made contractually between the two if different parties are responsible for external and internal stability.

If there is an upper slope above the nailed slope then the stability of this slope should also be checked (e.g. E in Figure 16).

4.2.2.2 Additional methods of analysing external stability for near vertical soil nailed slopes

NOTE Traditionally it has been a requirement to check soil nailed slopes for external stability as if they were gravity retaining walls or reinforced earth structures. Recommended checks have included bearing capacity, forward sliding and overturning.

In reality the soil nail zone does not act as a rigid block, nor do discrete soil boundaries normally exist either behind the soil nail zone or below it: the front facing can be soft or flexible, the soil is generally continuous, the nails are normally inclined downwards and represent individual 3-D inclusions rather than 2-D layers; consequently soil nailed slopes fit more naturally into slope stability philosophy and should be analysed as such, using the methods described above.

In certain circumstances however (e.g. near vertical slopes with hard facings) it may be appropriate additionally to carry out the traditional lower bound external stability checks for bearing capacity, forward sliding and overturning, treating the soil nailed zone as if it were a gravity retaining wall structure. In this instance the procedures set out in BS 8006-1:2010, **6.5** should be followed.

An indication of whether such checks are likely to be necessary or not will be given by the upper bound methods described in **4.2.2.1**:

a) potential bearing capacity issues are likely to be indicated by a properly performed slip circle analysis;

b) potential forward sliding issues are likely to be indicated by a properly performed two-part wedge analysis.

4.3 Soil nail pullout resistance

*COMMENTARY ON **4.3***

*Provided the nail tendon is sufficiently strong, then as relative movement occurs between the soil nail and the ground shear stresses will be mobilized between the surface of the nail and the ground. The relative movement will be different within the active and passive zones of a soil nailed application (Figure 20). This stress is known as bond stress and it has a limiting value, dependent upon a number of factors as described in **4.3.1**, **4.3.2** and **4.3.3**.*

*When the limiting value of bond stress is reached the nail will pull out of the ground or bond failure will occur. For calculation purposes it is convenient to establish a nail strength "envelope" for each nail in the slope by taking into account the limiting bond stress available at any point along the nail, the tendon strength R_t (see **4.5**) and the nail head force at the front face of the slope (T_f, **4.2.1.4**), Figure 21.*

From the nail strength envelope it is then simple to establish the tension in the nail at the point at which an assumed failure surface from the stability assessment crosses the nail.

Figure 20 **Mobilization of bond stress as a function of relative soil-nail movement**

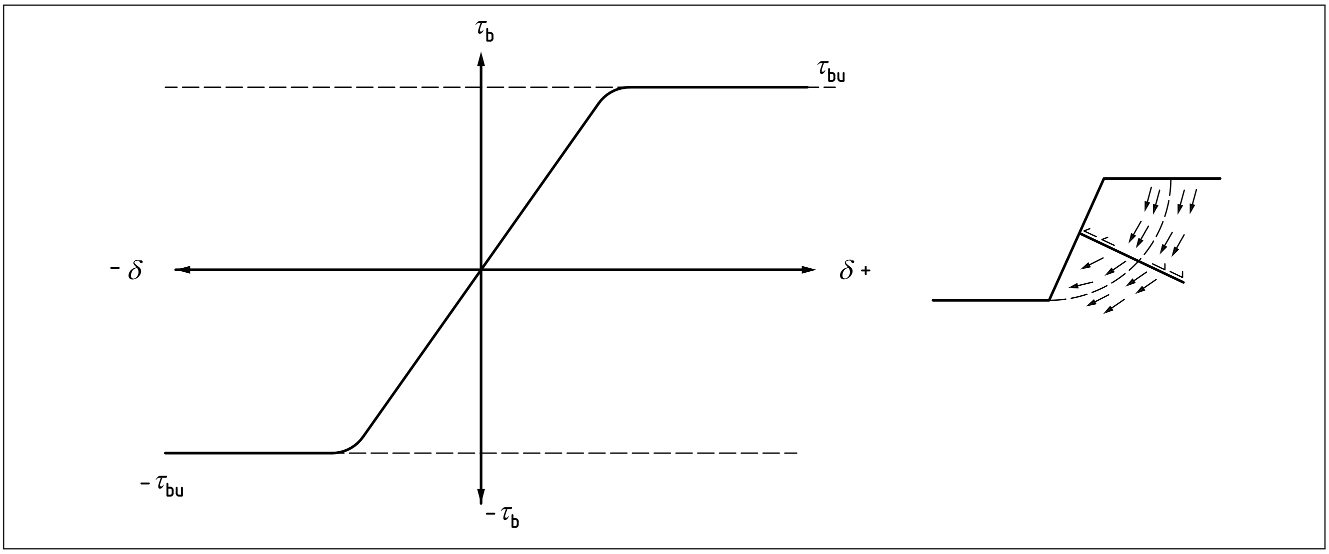
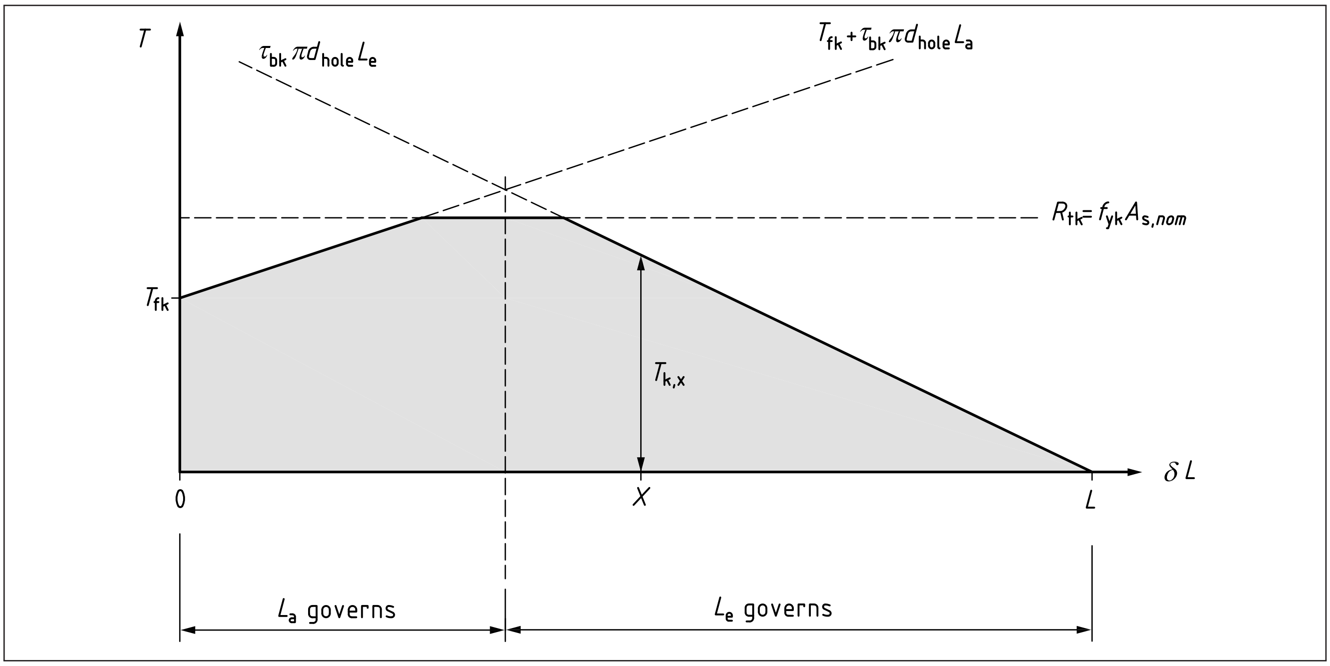


Figure 21 **Limiting nail strength envelope**



4.3.1 Factors affecting pullout resistance

There are a number of factors that affect the bond stress and hence pullout resistance between a nail and the ground and an appreciation of these is essential when designing a soil nailed structure or interpreting soil nail test data. The key factors may be grouped into three categories:

a) ground and groundwater conditions;

b) installation effects;

c) soil nail geometry; and

d) relative stiffness effects.

NOTE In some cases there is an indistinct boundary between the categories.

4.3.2 Soil and stress state effects

As with all soil-structure interaction problems, soil strength and stress state should be regarded as critical factors. The shear stress mobilized between the ground and a soil nail should therefore be assumed to be dependent upon the mobilized frictional strength φ' and the radial effective stress σ'_r at the interface of the nail and the ground.

The first parameter may be considered a function of the ground and nail surface roughness, however, the degree of mobilization will be dependent on the accumulated shear strain.

The post peak behaviour should be assumed to come into play at larger shear strains. The second parameter of the normal effective stress acting on the soil nail interface should be assumed to be influenced by numerous factors including:

a) rate of loading;

b) whether dilation or contraction occurs as relative movements and shear strains develop;

c) the ground's permeability and recharge potential; and

d) nail installation effects and stress state changes within the far field of the slope (due to overall slope movements arising from excavation or unloading).

With respect to this final criterion the radial stress σ'_r should be expected to be significantly different for a point on a nail where the horizontal stress σ'_h (in the direction of the slope face) and the in-plane stress σ'_L can change, such as in the active zone, relative to where the change is likely to be less significant, such as in the passive zone, (Figure 22).

*COMMENTARY ON **4.3.2***

Early work by Schlosser and Guilloux [24] attempted to explain how dilation resulted in bond stresses measured in pullout tests greater than those derived from the soil's shear strength φ' and the vertical effective σ'_v stress with limited success. More recent investigations by Standing [13], Luo [25] and Luo et al. [26] have advanced the understanding of the factors involved. However, the complex interaction of the factors involved means at best oversimplified and conservative models are only available for design purposes.

It is generally only the vertical effective stress at any point along a nail that can be estimated with a degree of reliability. However, most published methods for estimating ultimate soil nail bond stress contain a radial effective stress σ'_r term and are in the form:

$$\tau_b = \lambda_{pf}\sigma'_r \tan\varphi' + \lambda_{pc}c'$$

where λ_{pf} and λ_{pc} are interface factors on friction angle and cohesion respectively. Generally the term relating to cohesion c' is ignored. A number of proposals have been put forward to relate radial effective stress σ'_r to the vertical effective stress σ'_v in a slope. BS 8006-1:2010 sets σ'_r as equal to σ'_v. However, O.G.S.(1990) [27] suggested that the radial effective stress around a nail could be conservatively estimated from the average of the vertical and lateral stress from

$$\sigma'_r = \frac{\sigma'_v\left(1+k_L\right)}{2}$$

where k_L was in the range 0.5 to 0.8. In HA68/94 [23] the value of k_L can be calculated from

$$k_L = \frac{1+k_a}{2}$$

where k_a is the Rankine active earth pressure co-efficient and it is assumed that in a deforming slope the in-plane stress cannot be less than the average of the vertical and active horizontal stress. Standing [13] develops an alternative definition for k_L of:

$$k_L = b(1-b)k_a$$

where 0.2 < b < 0.35.

For typical characteristic values of φ' in the range 25° to 40° the relationship between the radial and vertical effective stress based on the various methods can lie in the range 0.55 to 0.9 (see Figure 23 and Figure 24).

As the ratio between radial and vertical effective stress reduces with increasing frictional resistance it may be appropriate to place a factor on the derived bond stress rather than the frictional strength in any partial factoring approach.

It should be noted that all of these methods give a generally lower ultimate bond stress than that derived from short-term pullout tests.

NOTE Luo et al. [26] provide an explanation for this. This is because the lateral stress is based on active conditions, whereas such conditions do not exist around a nail in a pullout test and possibly not for the portion of the nail in a passive zone. Furthermore, dilation at low stresses might not be catered for when a constant characteristic φ'_k value is assumed.

Consequently, a lower partial factor on the derived design bond stress may be used when calculating design values using a simple effective stress method.

Figure 22 **Effect of far field stress on mobilized bond stress**

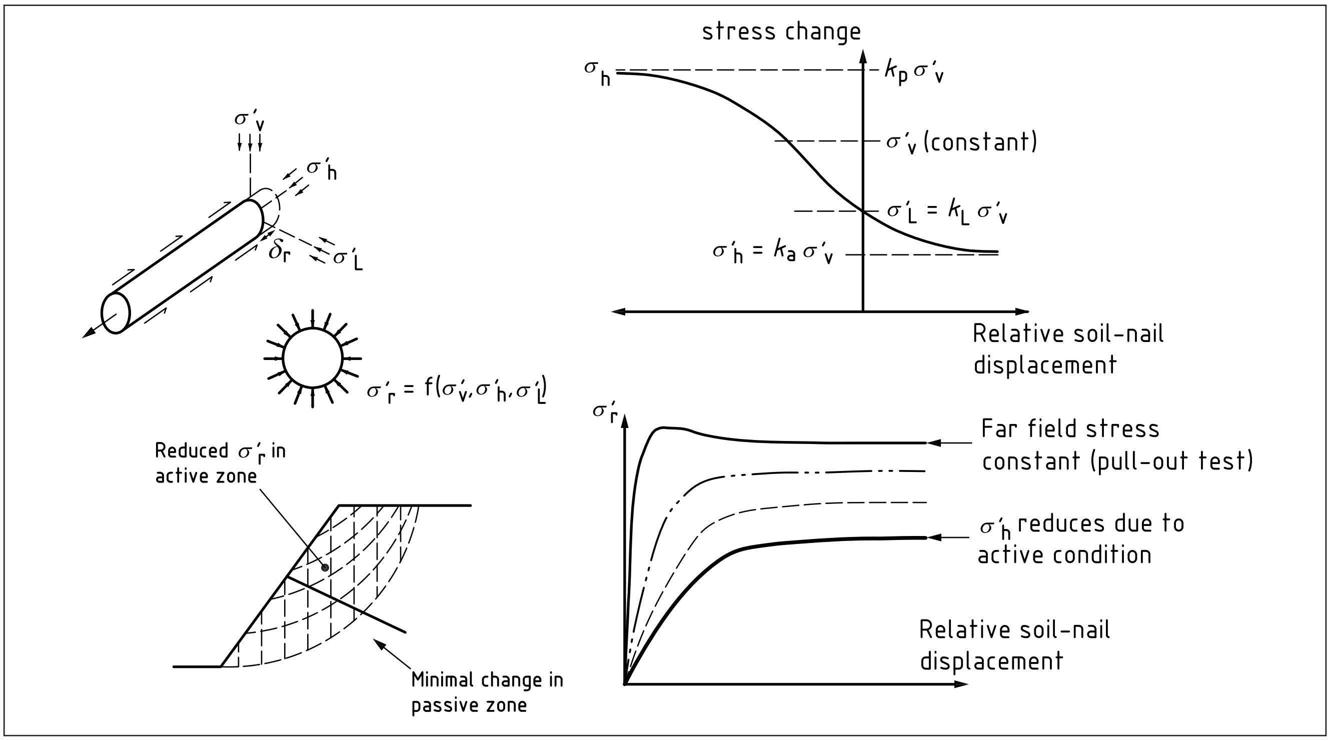


Figure 23 **Modification of interface stresses due to far field stress changes**

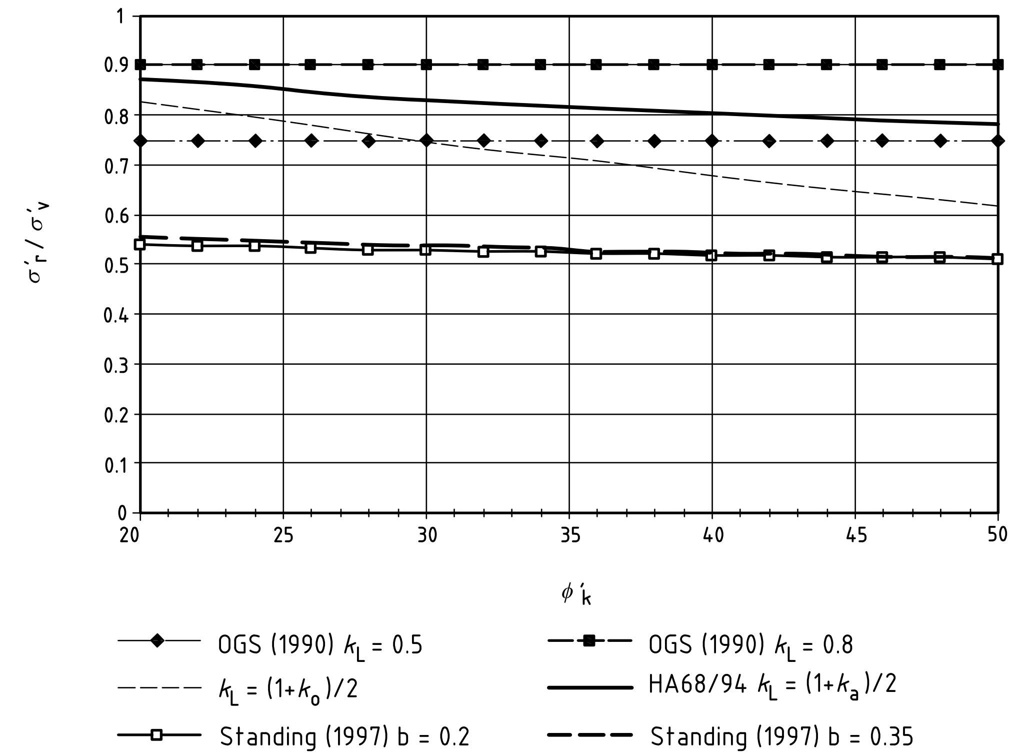
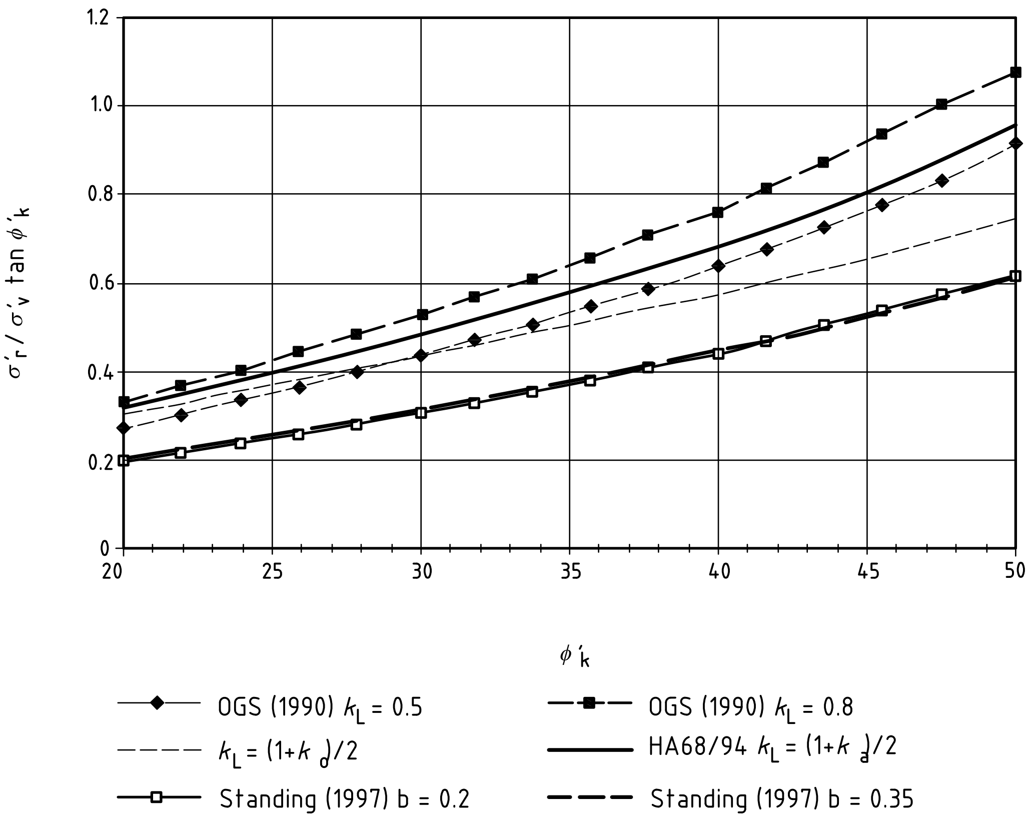


Figure 24 **Relationships between radial friction normalized by vertical effective stress for a range of characteristic friction angle**



4.3.3 Effect of nail construction method

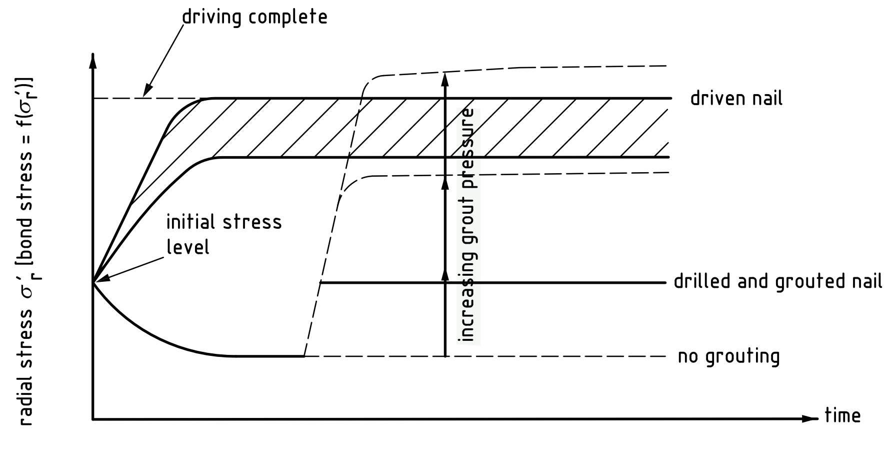
There are numerous methods of installing soil nails that should be considered, some of which will result in local stress increases and others in reductions in local stresses (see Figure 25). Generally methods that involve driving the nail into the ground may be expected to increase the stress locally around the nail. Drilled methods may be assumed to result in some reduction of stress, however, the actual magnitude will be dependent on the duration the nail bore remains open, the diameter of the bore and whether radial arching maintains stresses locally, whether casing is used, and whether during nail grouting initial stresses are reinstated or possibly exceeded.

In addition to changes in stress the mechanical effects of soil nail installation should be considered as to whether smearing of the borehole can occur, whether the borehole is smooth or rough, whether groundwater could result in softening and whether flushing media such as air or foam could result in clogging of natural pores or fissures. Installation methods that ensure a high degree of mechanical interlock should be encouraged.

NOTE Chu and Yin [28] report on a laboratory study investigating nail installation and grouting effects.

As the method of installation is likely to have an effect on the final pullout resistance then care should be taken when comparing the results of nail tests in similar ground to ensure the installation processes are understood.

Figure 25 **Modification of local interface stresses due to nail installation effects**



NOTE Changes in far field stress ignored.

4.3.4 Soil nail geometry and tendon stiffness

*COMMENTARY ON **4.3.4***

The peak bond stress measured during a pullout test in a given soil will vary depending on both nail diameter D and the length of the test section L_{bt}. The former effect is believed to be due to arching of stresses around the soil nail, where the ability for arching to occur is greater for smaller diameters than for larger diameters. Luo et al. [26] have proposed a model that takes this effect into account and which provides a good fit to experimental data. This observation is consistent with frictional data from ground anchors, minipiles and bored piles in similar ground where lower shaft friction is generally noted for larger diameters.

In addition to nail diameter *D*, the length of the nail being pulled out of the ground also has an effect that should be taken into consideration. Generally shorter test bond lengths should be expected to result in a higher average mobilized bond stress than for longer bond lengths.

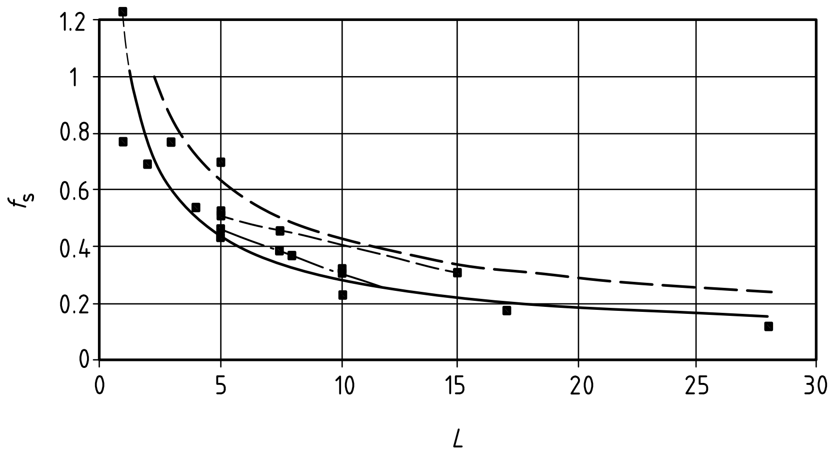
NOTE 1 This is an observation reported in BS 8081, after the work on ground anchors by Ostermeyer, and is considered to be due to different degrees of mobilization of bond stress along the test length. This effect is partly a result of non-uniform extension of the nail and is more prevalent in ground anchors than soil nails, as they typically exhibit lower axial stiffness and potentially longer bond lengths. Because of the reducing efficiency of long anchor bond lengths relative to short anchor lengths BS 8081 recommends limiting fixed lengths to 9 m.

The results of pullout tests on short nails that are applied to long soil nails may therefore need corrections to be applied. Furthermore the effect may be assumed to be greater in soils that have a significant strain softening behaviour.

NOTE 2 Barley and Graham [18] report on a test programme where soil nails with different axial stiffness have been tested and how the average bonds stress at failure varies with nail length, Figure 26. They propose empirical correction factors, or efficiency factors, dependent upon test length and axial nail stiffness to be applied to the nail bond length.

Empirical corrections would be complex to apply in a stability model, but should be taken into account when applying the results to a design where the likely nail bond length could be significantly greater than a test length.

Figure 26 **Effect of test length and axial stiffness on measured average bond**



Key

GRP nails, $f_s = 1.19L^{-0.62}$

Anchors, $f_s = 1.60L^{-0.57}$

50 mm steel bar nails, $f_s = 0.61 - 0.02L$ (15 > L > 5)

20 m steel bar nails, $f_s = 0.61 - 0.3L$ (10 > L > 5)

f_s Efficiency factor

L Fixed length (m)

After Barley and Graham [18].

4.3.5 Methods of assessing ultimate bond stress or soil nail pullout resistance

*COMMENTARY ON **4.3.5***

There is a range of methods by which soil nail ultimate pullout resistance or ultimate bond stress can be assessed. During the project design phase it is likely that a selection of approaches will be followed with a greater emphasis on empirical data at an early design stage. As knowledge of the ground conditions and parameters improves the option of using an analytical approach may arise, but it is not uncommon to develop a design on the basis of empirical data, that is subsequently validated by pre or post construction pullout tests. The designer in all cases will need to assess the degree of certainty that can be relied upon by any particular method. The options for assessing a characteristic design bond stress or pullout resistance are described below.

4.3.5.1 Assessment of ultimate bond stress by empirical approaches

NOTE It is common practice in soil nail design to employ empirical data when assessing the ultimate bond stress τ_b or pullout resistance T_b. Published guidance such as CIRIA C637 [1], FHWA [20] and BS 8081 provide an indication of characteristic bond stresses achieved in a range of soils and rocks for a variety of nail installation methods and diameters. The critical issue when using such data is in the understanding of the degree to which they can be relied upon in the new application.

Relative to the design being undertaken, the designer should challenge the similarity of the ground conditions, the nail installation method, the details of the test and test procedure, nail diameter, test length, the number of tests, the similarity of the design application and the proposed validation process.

The ultimate bond stress or bond force that may be derived by an empirical approach is denoted as $\tau_{b,ue}$ or $T_{b,ue}$ with respective units kN/m^2 and kN/m (length of nail). The characteristic and design bond resistances for use in the analysis may be derived from the ultimate empirical values as described in **4.3.6**.

Other methods of determining ultimate bond values from empirical correlations with soil tests, such as that proposed in Clouterre [19] for the Menard Pressuremeter should be treated in the same manner as direct empirical assessment from pullout test data as above.

4.3.5.2 Ultimate bond stress derived from effective stress methods

As discussed in **4.3.1** the state of stress acting around a nail is complex depending on the degree of slope movement and nail installation method; for simplicity the ultimate bond stress should be taken as the characteristic bond stress and calculated from the vertical effective stress and characteristic soil shear strength from:

[C1]
$$\tau_{bu} = \lambda_f k_r \sigma'_v \tan\varphi'_k$$
[C1]

where λ_f is an interface factor dependent upon the nail installation method and k_r is a factor relating the average radial effective stress around the nail to the vertical and has a value typically in the range 0.55 to 0.9, depending on the relative density of the soil and degree of stress reduction due to slope movements in the active zone of the slope.

NOTE This has an implication on the type of facing used.

The interface factor λ_f should be taken to be between 0.7 and 1.0 with the lower value relating to smooth interfaces and the higher value relating to rough interfaces.

Effective stress assessment of ultimate bond stress τ_{bu} tends to give low values when compared with pullout test results as it is based on a reduced stress state in the active zone and this should be acknowledged when assessing the characteristic bond stress τ_{bk} by employing a partial factor of 1.0.

4.3.5.3 Ultimate bond stress derived from total stress method

This method is analogous to the method used to derive shaft friction in pile design and relates the characteristic bond stress to the undrained shear strength c_u of the ground using an "alpha" coefficient.

$$\tau_{bu} = \alpha c_u$$

The value of the coefficient α lies in the range 0.3 to 0.6 for bored piles, however, if based on the efficiency factor proposed for anchors by Barley and Graham [18] is likely to be in the range 0.5 to 0.9 for bond lengths ranging from 7 m to 3 m.

It should be noted that the ultimate bond stress determined by this method is relatively high when compared with the effective stress method. Furthermore, in high plasticity soils where a large difference occurs between peak and residual shear strength, consideration should be given to overall slope displacement and the likelihood of residual strengths being mobilized. Consequently a higher degree of conservatism should be used when deriving characteristic bond stresses from total stress shear strengths.

4.3.5.4 Ultimate bond stress from pullout tests

Subclause **6.2** provides details of pullout tests that may be used in the execution of soil nailing. Design investigation and suitability soil nail pullout tests should be used to determine ultimate bond stresses for design or design verification respectively.

For UK applications the maintained load test method as detailed in BS EN 14490:2010 is recommended as the CRP test is relatively difficult to control and has a tendency to overestimate pullout resistance.

The characteristic bond stress τ_{bk} should be based on a cautious estimate through consideration of the number, location and consistency of the test results. Unless justified by an appropriate number of tests, then a reduction factor of between 1.1 and 1.3 should be applied to the average or 1.0 to 1.1 to the lowest result (see **6.2.4**).

*NOTE Derivation of the design value from pullout tests is detailed in **4.3.6**.*

4.3.6 Derivation of design bond strength

Subclauses **4.3.1** to **4.3.5** detail some of the factors that should be considered in the derivation of ultimate and characteristic bond resistances. Table 6 sets out the approach that should be followed to derive design values to be utilized in any ULS assessment of stability of a soil nailed structure or slope.

NOTE 1 Unique partial factors, applicable to all methods for deriving the characteristic and bond resistances, are not given but rather method specific ranges. This is necessary because a level of knowledge of the implication of how the ultimate bond resistance has been derived is necessary along with an understanding of the ground conditions, rates of loading, etc.

NOTE 2 The values in Table 6 have been selected to result in equivalent experience with lumped factors of between 1.5 and 3.0 on ultimate bond resistances (and micropile/ground anchor designs). The range given for γ_k is to reflect whether nails are used in a temporary or permanent application and the degree to which full dissipation of pore pressure is relevant.

As a range of values is given, the designer should consider the criticality of the design bond stress in the overall limit equilibrium model, brittleness of bond failure and the degree of validation specified.

Table 6 **Ultimate limit state approach to deriving design values**

Method of determining ultimate bond stress, τ_{bu}	**Factors for determining characteristic bond stress from ultimate values** $\tau_{bk} = \tau_{bu} / \gamma_k$	**Factors for determining design bond stress from characteristic values for set 1,** $\tau_{bd} = \tau_{bk} / \gamma_{\tau b}$	**Factors for determining design bond stress from characteristic values for set 2,** $\tau_{bd} = \tau_{bk} / \gamma_{\tau b}$
Empirical pullout test data	γ_k = 1.35 to 2.0 Selected value to be based on degree of confidence relative to proposed structure, soils, construction method, etc.	$\gamma_{\tau b}$ = 1.11	$\gamma_{\tau b}$ = 1.50
Effective stress *NOTE* τ_{bu} *derived from characteristic* φ'	γ_k = 1.0 to 1.35 Selected value to account for potential for dilation and degree slope deformation in active zone	$\gamma_{\tau b}$ = 1.11	$\gamma_{\tau b}$ = 1.50
Total stress *NOTE* τ_{bu} *derived from characteristic* c_u.	γ_k = 1.35 to 2.0 selected value to account for potential for strain softening, plasticity and shrink swell effects	$\gamma_{\tau b}$ = 1.11	$\gamma_{\tau b}$ = 1.50
Pullout tests	See BS EN 14490:2010 *Characteristic selected as a cautious estimate of the test data, taking into account the number of test results, location and consistency.*	$\gamma_{\tau b}$ = 1.1 to 1.3 for coarse grained soils $\gamma_{\tau b}$ = 1.5 to 1.7 for medium and high plasticity soils	$\gamma_{\tau b}$ = 1.5 to 1.7 for coarse grained soils $\gamma_{\tau b}$ = 2.0 to 2.25 for medium and high plasticity soils

4.4 Numerical analysis

Numerical methods may be used in the design of soil nail structures or specific components. They may be used to provide a clearer understanding of soil-structure interaction, deformations and collapse mechanisms in complex geological or geometric situations. Their use is widespread in geotechnical and structural engineering and provided appropriate expertise and comparable experience is used then reliable predictions and assessments of performance can be achieved.

Soil nail structures are generally complex and involve a variety of structural elements with varying material properties. Furthermore, construction sequences often involve 3-dimensional geometric changes all likely to result in challenging changes in stress and strain states. It is unlikely that a single numerical method will be able to provide an optimum analysis of a soil nail structure in its entirety and therefore it may be necessary to analyse components separately.

The choice of numerical method employed should take into account:

- the nature of the ground conditions;
- the interaction of structural components with the ground;
- compatibility of strains at the limit state being investigated;

- the sensitivity of the model to small changes in geometry during construction;
- previous or comparable experience and calibration of the numerical method or constitutive model in the situation to which it is being used.

NOTE The factors provided in this standard have not been established for use in conjunction with numerical methods.

4.5 Soil nail element design

4.5.1 Structural design of nail tendon

The following are principles and rules that should be implemented for soil nail tendons in the form of ribbed bars fabricated of steel in accordance with BS EN 10080 with a characteristic yield strength f_{yk} not exceeding 0.2% of the proof strength load in direct tension divided by the nominal cross-sectional area or $f_{0.2k}$, and in the range 400 N/mm² to 600 N/mm².

Where other materials are used for the tendon, including non-metallic materials and components such as couplers then manufacturers' test data should be relied upon in relation to the partial factors. The designer should take into account the linearity of the stiffness over the service load range of the tendon.

The requirements for the properties of the tendon should be for the material as placed in the ground. If site operations can affect the properties of the tendon then the properties should be verified.

The surface characteristics of the ribbed bars should be such to ensure adequate bond to any surrounding grout.

If the maximum design load used in the stability model is $T_{d\ max}$ then the design tensile resistance R_{td} of the nail should satisfy:

$$T_{d\ max} \leq R_{td}$$

$$R_{td} = R_{tk} / \gamma_s$$

where R_{tk} is the characteristic yield strength of the nail, bar or tendon and γ_s is a partial factor on tensile strength.

The characteristic yield strength of the tendon R_{tk} should be based on the nominal cross-sectional area of the tendon at the end of its design life $A_{s,nom}$ and the characteristic yield strength f_{yk}:

$$R_{tk} = A_{s,nom} f_{yk}$$

The partial material factor γ_s should be taken as:

1.15 for persistent and transient situations;

1.00 for accidental situations.

4.6 Influence of durability and degradation on the choice of nail tendon

*COMMENTARY ON **4.6***

The vast majority of soil nails installed in the UK are located in partially saturated soils, often subject to seepage and seasonal variations in water content and atmospheric exposure creating environmental conditions which might favour degradation of the soil nail components. The durability of a soil nail tendon and its associated components can be highly variable and is primarily influenced by the aggressivity of the soil mass into which it is placed. Based on case histories and research, the specified lifespan of soil nails can vary from only a few months to more than 120 years depending on the type of protection system provided, the aggressivity of the ground and the external environment.

4.6.1 Assessment of degradation risk

Refer to **3.13**, **3.14** and **3.15** for the recommended approaches that should be adopted for assessing the degradation risk of buried and exposed components.

NOTE For the most common incidence of corrosion, it is well understood that under unconstrained conditions, steel will react with oxygen and water to form oxides and/or hydroxides. Aside from environmental factors, the class of corrosion depends on the type of electrochemical cell formed.

For example, differential aeration cells can occur in soil nails as the steel tendon passes from soil of one permeability or pore structure into another, e.g. gravel into clay, or disturbed ground into undisturbed ground. Parts of the nail are surrounded by a high concentration of dissolved oxygen and form cathodic areas, whilst parts in a low oxygen concentration form anodic areas. Pitting or localized corrosion attack is usually concentrated close to the boundaries of the cathodic areas.

For simplicity, corrosion may be categorized into three broad classes, namely:

i) generalized attack representing an approximate uniform attack where individual anodic and cathodic sites fluctuate across the exposed surface;

ii) localized attack where separate corrosion cells are present, distinguished by variations of the electrode potential over the steel surface; attack may be very severe resulting in pitting yet the loss of metal is very small; and

iii) cracking due either to hydrogen embrittlement or environmentally assisted stress corrosion cracking as a result of the conjoint action of internal and external static tensile stress and localised corrosion.

4.6.2 Service environments

Consideration should be given to the environmental conditions during the design life of the soil-nailed structure; the conditions encountered by the buried nail components and the surface nail components are often different.

The conditions which exist at the time of construction may usually be determined relatively accurately by carrying out site investigations; however, the environmental conditions which might exist during the life of the soil-nailed structure can be more difficult to predict.

*NOTE Section **3** provides guidance on the ground conditions best suited to soil nails and the nature of ground investigation appropriate for soil nail design.*

4.6.3 Nail heads

*COMMENTARY ON **4.6.3***

The nail head constitutes a potential weak spot in the soil nail system due to the environmental exposure to which it could be subjected.

With soft or flexible facings, the nail head can generally be seen and the condition of its protection observed throughout its design life. However, where a hard facing is used, the nail heads are usually covered with sprayed concrete and cannot be seen. Similarly, the underside of the head plate and the junction between the nail head and the buried nail cannot be observed over time. The environmental conditions at the exposed nail head can markedly affect the durability of the nail head components.

The following conditions should be considered when selecting a nail head, coating type, or head protection method:

a) marine environments;

b) humidity;

c) temperature variation;

d) exposure to run-off water, drainage or seepage;

e) degree of cover of crack free concrete, which can be beneficial, although it should be noted that on exposed structures rust staining can be indicative of the limitations of concrete cover.

The primary corrosion risk for nail heads is removal of pre-applied coatings during handling and placement, particularly at bolted connections, and post-installation repairs may be required. The last option (full encapsulation) may be assumed to require the least maintenance during the design life provided that the encapsulation is fully integral with the nail tendon protection and cracking is limited.

NOTE 1 If the facing has a shorter design life than the rest of the soil-nailed slope or wall, it is likely that it will need to be replaced during the design life of the slope or wall. Grease filled steel caps are an example of a nail head system which will enable the nut and head plate to be removed with relative ease at a later date to install such replacement facings. Other nail head protection systems such as sacrificial thicknesses or encapsulation in concrete would not permit this.

Concrete bearing pads should be designed for durability as per the recommendations of BRE SD1 [3] and an appropriate type of cement should be used.

*NOTE 2 Chemical testing of the soil and groundwater are required during the site investigation for such assessments, as discussed in **3.12**.*

4.6.4 Materials

*COMMENTARY ON **4.6.4***

The materials commonly used for soil nails are inclusive of the following:

*a) uncoated steel (**4.6.4.1**);*

*b) coated steel (**4.6.4.2**);*

*c) austenitic stainless steels (**4.6.4.3**); and*

*d) fibre composite materials (**4.6.4.4**).*

4.6.4.1 Uncoated steel

Steel nails may be made, and are usually made, of carbon manganese steel.

4.6.4.2 Coated steel

4.6.4.2.1 Galvanized steel

Steel nails may be galvanized with protection provided by a coating that forms a series of alloy layers on the steel surface; thus the coating is chemically integral with the surface and is more robust than other types of coating that rely on adhesion to the steel surface.

NOTE 1 The primary means of protection is as a barrier between the steel and the environment. However, in many environments the galvanized coating will corrode, albeit relatively slowly and at a rate of several orders of magnitude less than the base steel. The design life of the coating depends on the environment to which it is exposed and the thickness of coating supplied. Generally galvanized components are supplied with a standard coating thickness (or weight) that is related to the section thickness of the component as defined in standards such as BS EN ISO 1461:1999.

It should be noted that galvanized steel performs best in conditions of atmospheric exposure in near neutral pH and low chloride conditions. Galvanized coatings rapidly corrode in even mildly acidic conditions as might be found in peat or fills associated with colliery spoil and in these environments alternative materials should be considered.

NOTE 2 There is a risk of damage to galvanized coating during transportation, handling and installation. The risk is probably increased when nails are installed by driving, rotating or by pneumatic firing.

4.6.4.2.2 Epoxy and other coatings applied to steel

Steel nails may have an epoxy coating consisting of a fusion-bonded epoxy resin applied to the tendon prior to delivery to the construction site. A minimum thickness of 0.3 mm should be applied, as recommended by (FHWA [20]) and commonly used in the USA.

NOTE In reviewing the historical use of epoxy coated strand in dams in the United States of America, Bruce [29] found that attention to fabrication details is essential for production dependability. The main source of failures resulted from problems of epoxy adhesion to the steel.

In addition to fusion bonded epoxy coatings, tar-epoxy, tar-polyurethane may be used on steel surfaces provided they are abrasively blast cleaned prior to coating, as suggested by BS EN 1537. BS EN 1537 also recommends that such measures may be used as corrosion protection to tendons of temporary systems provided they are factory applied and quality procedures ensure the integrity of the cured coating film.

4.6.4.3 Stainless steel

NOTE 1 Stainless steels are a group of steel alloys that have more than 10.5% chromium. They are classified in Table 7.

The commonly used stainless steels that soil nails may be made from are resistant to general corrosion in all naturally occurring environments; however, they are susceptible to localized corrosion in environments that contain chlorides.

NOTE 2 Localized corrosion is most likely to take the form of pitting or crevice corrosion. This risk will generally increase with increasing chloride concentration in the ground and decreasing alloying content. The common alloy materials used for soil nails are austenitic stainless steels grades 1.4301 and 1.4401 (as per BS EN 10088). These are readily available steels regularly used for geotechnical applications. They can be supplied in most types of tendon, coarse threaded bar and as stainless steel threadbar and are available for use throughout the soil nailing industry.

Table 7 **Types of stainless steel**

BS 970 Grade	BS EN 10088	Performance
304 S 31	1.4301	Good all round corrosion protection controlled amount of bulk corrosion
316 S 31	1.4401	Extra molybdenum for improved resistance to chloride attack suitable for marine applications
Duplex	1.4462	Has extra resistance to pitting corrosion where the extra expense warrants the inclusion of this material.

4.6.4.4 Fibre composite soil nails

4.6.4.4.1 General

Fibre reinforced plastic (FRP) tendons may be used in soil nailing that comprise stiff strong fibres embedded in a resin matrix; the fibres are designed to accommodate the required loads and the matrix is to transfer load between fibres and protect the fibres from mechanical and environmental damage.

NOTE Carbon fibre does not deteriorate in the alkaline environment provided by cement grout and as a tensile member has numerous advantages as a replacement of high grade steels which themselves are highly vulnerable to corrosion.

4.6.4.4.2 Glass fibre reinforced plastic composites

Glass fibre is the most widely used reinforcement that may be chosen and is available in a variety of types as outlined in Table 8.

NOTE As is the case for most fibre reinforced soil nails, a glass reinforced plastic composite soil nail relies primarily on the fibre properties for its tensile performance and the resin matrix for its lateral shear resistance. Glass fibre reinforcing material is relatively light weight with moderate strength, but with relatively low stiffness and is used extensively in the mining industry.

Table 8 **Types of glass fibre (after Littlejohn [30])**

Glass type	Relative advantages
E-glass	Lowest cost type commercially available, used typically where strength, electrical resistance and acid resistance are important.
S-glass	Higher strength, stiffness and ultimate strain than E-glass but more susceptible to degradation in alkaline environments.
C-glass	Greater chemical stability in acidic environments, c.f. E-glass.
AR-glass	Alkali-resistant to minimize weight and strength loss in alkaline environments.

4.6.4.4.3 Aramid fibres

NOTE The Federal Trade Commission (USA) definition of Aramid fibres is a manufactured fibre in which the fibre forming substance is a long-chain synthetic polyamide in which at least 85% of the amide (-CO-NH-) linkages are attached directly between two aromatic rings.

Aramid fibres may be used for their superior strength/weight ratio (when compared to glass fibres) and to provide excellent abrasion and impact resistance in a composite. It should be noted, however, that they are known to be poor in compression, and creep, and are susceptible to degradation by ultra-violet light.

4.6.4.5 Durability of facings

The durability of hard (sprayed concrete) facings should be assessed according to the recommendations of BRE SD1 [3] and an appropriate type of cement should be used. Chemical testing of the soil and groundwater are recommended during the site investigation for such assessments, as discussed in **3.12**.

*COMMENTARY ON **4.6.4.5***

Flexible and soft facings are usually formed from proprietary materials, which when used in the UK or European Union have to have a Conformité Européen (CE) mark. Some of the methods for assessing their durability include third-party accredited certification, test results from independent laboratories and self certification Quality Assurance/Quality Control testing to ISO standards by the manufacturers. Test methods are detailed in BS 8006-1:2010 and by more recent European standards.

Commonly used hexagonal steel wire mesh for soft and flexible facings is normally zinc coated or galvanized. Such metallic meshes are typically coated with plastic to improve their durability and the design life. The potential for damage to such coatings, especially at the edges of head plates, needs to be taken into account where the coating is relied upon to provide durability. It is areas of incorrectly repaired coating damage that often determine the real life of coated meshes and facings. The issue of ultra-violet stability of plastic coatings in the long-term also has to be considered.

4.6.5 Interaction of materials and environments

4.6.5.1 Uncoated steel directly within the ground

Nails made of carbon manganese steel may be driven, rotated or pneumatically fired into the ground for slope stabilization; such systems offer economic advantages gained by rapid installation, but there is little information on their durability.

Consideration should be given to the fact that steels of this type corrode in all buried and exposed environments that are likely to be encountered by soil nails, and in the long term will encounter some loss of steel section. Exposure environments that designers should be aware of because they increase the rate of corrosion include:

a) combinations of oxygen and chlorides;

b) anaerobic conditions in the presence of sulfates.

The type and severity of attack will be affected by the specific environment but uncoated, or otherwise unprotected, steel may only provide a durable solution in the most benign conditions or where exposure is for relatively short periods of time. Temporary reinforcing steel elements should normally be expected to last two years in benign conditions (BS 8081:1989) and this period should be regarded as the limit for such elements in any circumstances. In very aggressive conditions or where there is a risk of local damage or corrosion by pitting, unprotected reinforcing elements may only be expected to last a few weeks.

Where possible, the corrosion history of buried metals in the vicinity of the proposed works should be established. This may serve as a useful guide to the degree of protection required.

Therefore, steel nails should normally be provided with additional protection either as a coating or by using additional section thickness that may be regarded as sacrificial within the life of the nail.

4.6.5.2 Coated steels directly in contact with the ground

To increase durability some form of protective coating may be applied to the steel tendon. Coatings such as galvanizing (see **4.6.4.2.1**) and epoxy (see **4.6.4.2.2**) may be used to provide protection of the steel against corrosion; the durability of the coating will depend on the aggressivity of the environment, the thickness and quality of the coating applied and the preservation of the coating during installation.

The integrity of the coating after installation using driven methods should be checked by excavating test nails or by pulling out the nails during test trials.

NOTE Driven, coated soil nails are rarely used in the UK and there is little information available about the condition of the coating after driving. However, if the coating is subjected to extensive damage during the installation process the benefit provided by the coating becomes questionable.

4.6.5.3 Methods that isolate the tendon

4.6.5.3.1 General

Where soil nails are required to be part of a permanent structure then the use of the sacrificial loss of section concept should be limited to low risk category structures where ground conditions are not aggressive. The method is not recommended for soil nail tendons with small cross sectional area. As recommended by BS EN 14490:2010, the sacrificial steel thickness should not be used as the only means of protection where the percentage loss of cross section exceeds half of its initial cross section unless nails are installed at fairly close centres and a degree of redundancy exists.

4.6.5.3.2 Cement grout surround – Uncoated steel

The most common method that may be used to install soil nails is by predrilling holes, installing nails and grouting the bore. Direct drilling of a hollow nail using grout as the flush medium may also be used (known as self-drilled or self-drilling nails).

*COMMENTARY ON **4.6.5.3.2***

It is inevitable that the grout surrounding a steel tendon in the ground will crack due to axial movement. There is little field or laboratory data to suggest the surface crack width at which corrosion of the tendon becomes a serious risk. However, BS 8081 proposes an upper limiting crack width of 0.1 mm. This crack width is considered appropriate for high yield prestressing steel with high carbon content (i.e. with specified yield stress > 1 500 MPa or hardness > $450H_{v10}$).

Other relevant points from BS 8081:1989 are as follows.

*a) BS 8081:1989, **8.1.2.1** states that steel is protected against corrosion when maintained in a high pH environment free of aggressive ions. Such an environment is provided by hydrated hydraulic cement which will give excellent protection over the long term while the high level of alkalinity remains. However, loss of protection can occur as a result of lowering the alkalinity, through cracks or carbonation, or the presence of aggressive ions, especially chloride.*

*b) BS 8081:1989, **8.1.2.2** in relation to corrosion of steel in cracked concrete, corrosion is likely to start first where a bar intersects a crack. In the short term (for example two years) the crack width has a significant effect on the amount of corrosion found near a crack. In the long term (for example 10 years) based on observations of 0.05 mm to 1.5 mm cracks, the effect of differences in crack width on the amount of corrosion reduces dramatically.*

The presence of cracks that exceed the limiting crack width of 0.1 mm does not render the grout ineffective as a means of protection since the alkaline environment (pH 9.5 to 13.5) due to the grout can maintain the steel in a passive condition providing there is no water flow through the crack and no chlorides are present. In most cases any corrosion where steel tendons cross a crack in grout would be no worse than if that tendon were exposed to the surrounding ground without grout.

More recently, John and Littlejohn [31] proposed a relaxation of crack widths to 0.2 mm for standard and high strength reinforcement steels (i.e. with specified yield stress < 500 MPa or hardness > 210H_{v10}).

4.6.5.3.3 Cement grout surround – Coated steel

The performance of steel nails surrounded by cement grout may be enhanced by the inclusion of coated steel, either galvanized or epoxy coated. With careful handling during transportation and installation the risk of coating damage may be considerably reduced from that of driven coated steels.

NOTE There is little information about the use of epoxy coated steel in the UK soil nailing industry. However, epoxy coated nails are commonly used in South Korea and in the USA.

4.6.5.3.4 Steel surrounded by grouted impermeable duct

Impermeable ducting may be used to surround a steel tensile member and to isolate it from the environment, a technique that was developed for ground anchor protection in the late 1960s. As BS 8081:1989 acknowledges, the protective elements around a tensile member should be used to transmit tensile stresses, and the use of a semi-rigid corrugated duct should be used, provided that the strength and deformability characteristics have been proved through adequate testing.

NOTE Recent standards and manuals relevant to ground anchors and nails (BS 8081:1989, Clouterre [19], BS 8006:1995, BS EN 1537 and BS EN 14490:2010) acknowledge the benefits of isolation of the tensile member in the ground where appropriate, but have differing requirements with respect to ducts.

As recommended in other standards, the duct material should be proven as an effective impermeable barrier by testing. For long nails, where lengths of duct need to be joined, the test regime should also address the effectiveness of the jointing method.

The nature of the materials used for impermeable ducting should be considered. Corrugated duct corrosion protection may (as is most commonly the case) consist of encasing the tendon in a grout filled corrugated PVC (poly-vinyl chloride) or HDPE (high density polyethylene) tubing. The annular space between the tendon and the corrugated tube, commonly specified as a minimum of 5 mm, should be filled with neat cement grout. Internal spacers should be used to achieve the grout cover inside the encapsulation.

*COMMENTARY ON **4.6.5.3.4***

Duct diameters are influenced and controlled by internal grout cover between the nail and the inner duct face or by grout cover between the coupler body and the inner duct face. The minimum cover to enable an annulus of grout and transfer of load is in the order of 2 mm to 3 mm (Barley [32]) when using neat cement grouts. These factors generally control duct diameters to 40 mm without couplers (see Figure 27) and up to a maximum diameter of 80 mm to accommodate the largest couplers. Typical duct wall thicknesses vary from 0.8 mm to 1 mm to ensure impermeability in these diameters and to minimize installation damage.

Unless otherwise approved, corrugations should be uniform and generally sinusoidal in shape, conforming to the following:

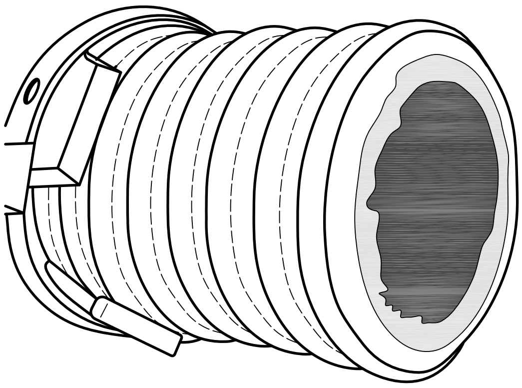
Wall thickness (w) of ducts > 60 mm minimum internal diameter: *w > 1.5 mm*

Wall thickness (w) of ducts < 60 mm minimum internal diameter:	*w > 0.8 mm*
Pitch of corrugations (p):	*12w > p > 6w*
Amplitude of corrugations (a):	*a > 3w*

The profile cannot allow voids to be formed in the rising grout column.

Ducts containing the nail tendon are often grouted whilst the nail tendon and duct are in situ or (less often) are pre-grouted. The first method involves installing the centralized duct in the borehole, installing the steel nail tendon into the duct with appropriate centralizer(s) and carrying out grouting from the top of the duct. The grout will pass down the annulus between the nail tendon and the duct, returning up the outer annulus. This system generally ensures efficient grouting and filling of all voids, with the exception of the small isolated air voids in the roof of the corrugations. Where soil nail tendons are pre-grouted into the corrugated duct, prior to installation into the borehole, each encapsulation system should be checked to ensure adequate internal clearances for the passage of the grout.

Figure 27 **A 25 mm diameter steel tendon with a 40 mm diameter impermeable duct**



Source: Barley and Mothersille [33].

4.6.5.4 Centralizers

Centralizers may be employed primarily to maintain a minimum grout cover to an external duct or soil nail tendon within the borehole or within the duct. In many instances it may be inappropriate to demand concentric location of the nail and the duct in borehole.

*COMMENTARY ON **4.6.5.4***

An annulus of good quality, but inevitably cracked, cement grout around the nail tendon offers some protection against corrosion. Centralizers installed at centres not greater than 3 m along the soil nail tendon are essential to maximize the likelihood that there is a continuous grout annulus. In some instances 1.5 m to 2 m spacing might be more appropriate.

Maintenance of this grout annulus is particularly a potential problem at the locations of couplers (which join tendon lengths together). Without adequately spaced centralizers, couplers and nail tendons can lie on the drilled bore with no cover, and/or cover can be reduced at the top of the nail bore.

Centralizers should be made of a non-corrodible material having no deleterious effect on the tendon itself and the use of metals dissimilar to the tendon should be avoided. They should be sufficiently robust and correctly secured so that they remain properly attached to the tendon and are not damaged during installation of the soil nail.

NOTE This is especially important with self-drilled nails as the nail installation process involves partially inserting and withdrawing the nail several times during drilling, which can lead to an increased likelihood of damage to the centralizers.

Construction supervisors should check frequently that the designer's and manufacturers' instructions regarding installation of centralizers on self-drilling nails are carefully followed. Similarly the grout column should be carefully topped up prior to installing the head components to ensure continuous grout cover to the tendon all the way up to the back of the head plate.

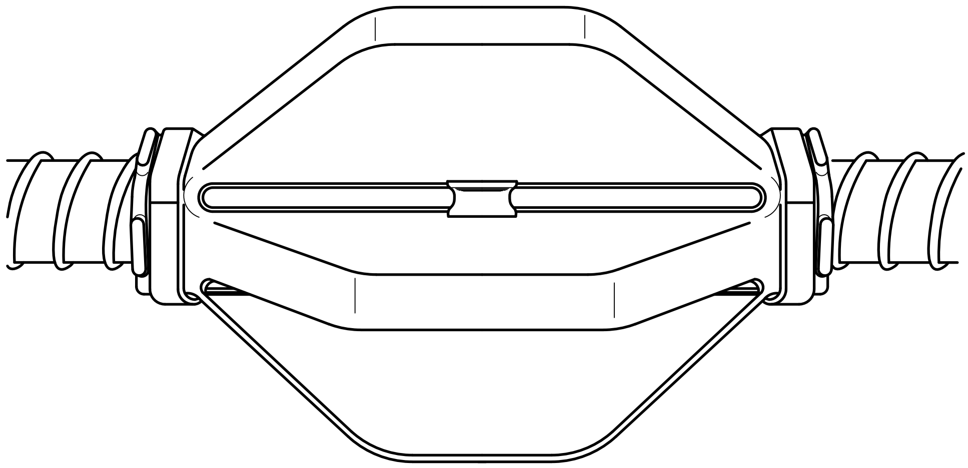
Centralizers may also be used to protect the tendon from damage during installation; this particularly applies to:

a) coated steel tendons, where adequately spaced centralizers can significantly reduce the risk of coating removal during installation (Figure 28);

b) self-drilled coated nails, to reduce wall contact and to maintain the integrity of the entire coating during drilling; however damage within the thread length may be inevitable;

c) composite nails.

Figure 28 **A centralizer to provide cover to a coated nail to reduce the risk of damage to the coating during installation**

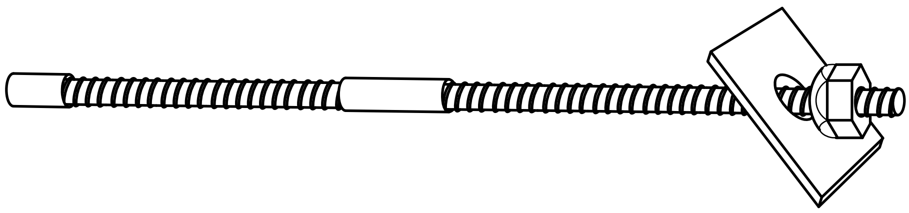


Courtesy of Grove Distributors, Ltd.

4.6.5.5 Self-drilled nails

Self-drilling nails (also known as "self-drilled" nails, see Figure 29) may be obtained in steel, coated steel and stainless steel. Where self-drilled coated nails are installed they are probably subjected to less coating damage than displacement nails because there is less nail-to-soil contact, but a centralizer system should still be used. Stainless steel self-drilling nails may give appropriate longevity in conjunction with bore grouting methods.

Figure 29 **A stainless steel self-drilling tendon complete with drill bit, hollow tendon, coupler and head plate**



Courtesy of Stainless Steel Ltd.

4.6.5.6 Steels and composites with enhanced corrosion protection

*COMMENTARY ON **4.6.5.6***

*BS 8081:1989, **8.2.1** gives the following guidance on anchorages: "Double protection implies the supply of two barriers where the purpose of the outer barrier is to protect the inner barrier against the possibility of damage during tendon handling and placement. The outer barrier therefore provides additional insurance, given the distinction between the degree of protection of the tendon once installed in the ground, and that of the tendon supplied".*

For soil nails the most vulnerable protective barriers are the galvanized or epoxy coating or the potential long term deterioration of a composite or stainless steel tendon, followed by the potential damage to an impermeable duct.

When nail tendons are long (greater than 20 m) and heavy (incorporating a number of couplers), the risk of damage to a single protection layer increases; thus, consideration should be given to enhancing the protection.

Enhanced corrosion protection may be provided by:

a) coated steel within a single impermeable duct;

b) composite nails within a single impermeable duct;

c) stainless steel with a single impermeable duct;

d) steel pre-grouted within two concentric impermeable ducts (as commonly used in anchor practice).

Where enhanced corrosion protection is used it should extend over the entire length of the nail and especially over the couplers and the nail head.

4.6.6 Recommendations relating to materials, environment and risk

4.6.6.1 Nail installation techniques and nail tendon systems

*COMMENTARY ON **4.6.6.1***

There are few nail installation techniques but many nail tendon systems available in the UK soil nailing industry. These systems can offer:

a) tendons with estimated rates of corrosion or degradation, and hence an estimated design life;

b) tendons partially or totally isolated from the environment.

4.6.6.1.1 Driven nails

Nails installed by soil displacement methods generally have no cement cover, although some may have post grouting facilities which could enhance protective measures. Displacement nails are recommended for short-term use or may be designed for loss of section with a limited design life or for low risk categories. (BS EN 1997-1:2004, Category 1).

The durability of coated displacement nails should be greater than uncoated nails but loss of coating during handling and installation is a risk and difficult to quantify.

4.6.6.1.2 Bored and grouted nails

Soil nails constructed using pre-boring techniques may be used to allow inspection of the integrity of the soil nail, its components, and its coating or duct containment prior to controlled installation.

NOTE They can offer a wide range of systems and durability. Centralizing of tendons and couplers and good grouting techniques help to give grout cover and a guaranteed alkaline environment.

The grout column will often become cracked due to soil mass movement and the consideration of cement grout as a continuous protection barrier layer is not advised for long life high risk categories, i.e. it should only be used for BS EN 1997-1:2004, Category 1 and Category 2 structures.

*COMMENTARY ON **4.6.6.1.2***

Coatings are available to supplement the steel protection offered by the cementitious environment and the availability of a range of composite nails gives other options involving greater and lesser durable materials. Stainless steels are generally available in three grades which give several ranges of durability.

Nails installed in pre-bored holes can be fully isolated from the environment by containment in a grouted impermeable duct. Care is required to prevent damage to the outer duct face by bore centralizing and to the inner duct face by preventing contact with the steel and couplers during installation. These are appropriate for use in all BS EN 1997-1:2004 Categories. As soil nails become longer and heavier the potential risk of duct damage might increase. Such risk can be reduced by pre-grouting the nail within the ducting prior to careful installation. Attention needs to be given to the coupler protection applied in situ if installed in multiple lengths. Table 9 summarizes the recommended use of these nail protection systems.

4.6.6.1.3 Self-drilling nails

Self-drilled nails may be selected from a range of material types from uncoated steel to coated or stainless steel, providing a range of durability characteristics from BS EN 1997-1:2004, Category 1 to Category 3. Grout cover is generally recommended although in some circumstances cover cannot be guaranteed and as with pre-bored nails the grout column might become cracked.

NOTE The presence of the alkaline environment is generally beneficial in retarding steel corrosion (suitable for Category 1 and Category 2).

4.6.6.1.4 Enhanced corrosion protection systems

Enhanced protection to reduce the risk associated with damage to an individual protection barrier during handling, installation or passive nail loading may be considered appropriate for high risk category (BS EN 1997-1:2004, Category 3) structures, particularly where nails are long and heavy and might be vulnerable to protection damage. Such protection systems that may be considered include:

a) coated steel within an impermeable duct;

b) vinylester composite within an impermeable duct;

c) stainless steel within an impermeable duct;

d) uncoated steel within two concentric impermeable ducts.

Where pre-inspection of the full corrosion protection is required prior to nail installation then the use of pregrouted nail tendons may be required. This is consistent with the demands of ground anchor technology but, as in **4.6**, coupler protection against corrosion should be applied in situ.

NOTE Table 9 summarizes the recommended use of enhanced protection systems.

4.6.6.2 Nail heads

NOTE 1 The load transferred to the nail head generally increases as the slope angle increases. The degree of durability required for the nail head depends on the slope angle and other factors.

For shallow slopes, the durability requirements at the nail head may, when appropriate, be less stringent than those for the buried nail. However, when nail heads make a major contribution to stability, they should be detailed for the same design life as the nail itself.

NOTE 2 The exposed nail head environment can differ considerably from the buried nail environment.

An overlap in protection should be made at the junction between the nail head and the buried nail tendon as this is a particularly vulnerable location. Nail head durability may often be achieved in a different way from the nail tendon protection used within the bore.

Table 9 **Summary of recommendations for different soil nailing systems in relation to different categories of risk**

After Barley and Mothersille [33].

Type of soil nail	**Category of risk**								
	Low risk			Medium risk			High risk		
	T or P in SCE	T in HCE	P in HCE	T or P in SCE	T in HCE	P in HCE	T or P in SCE	T in HCE	P in HCE
Steel directly in soil	R	R	NR	R	NR	NR	NR	NR	NR
Coated steel directly in soil	R	R	R	R	R	NR	NR	NR	NR
Steel surrounded by cement grout	R	R	R	R	R	NR	R	NR	NR
Self drilled steel surrounded by cement grout	R	R	R	R	R	NR	R	NR	NR
Coated steel surrounded by cement grout	R	R	R	R	R	NR	R	NR	NR
Self drilled coated steel surrounded by cement grout	R	R	R	R	R	NR	R	R	NR
Polyester composite surrounded by cement grout	R	R	R	R	NR	NR	R	NR	NR
Vinylester composite surrounded by cement grout	R	R	R	R	R	R	R	R	NR
Stainless steel surrounded by cement grout	R	R	R	R	R	R	R	R	NR
Self drilled stainless steel surrounded by cement grout	R	R	R	R	R	R	R	R	NR
Steel surrounded by grouted impermeable ducting	R	R	R	R	R	R	R	R	R
Coated steel surrounded by grouted impermeable ducting [A)]	R	R	R	R	R	R	R	R	R
Stainless steel surrounded by grouted impermeable ducting [A)]	R	R	R	R	R	R	R	R	R
Steel surrounded by pregrouted double impermeable ducting [A)]	R	R	R	R	R	R	R	R	R

Key

T = Temporary (< 2 years)	SCE = Slightly corrosive environment [B)]	R = Recommended
P = Permanent (> 2 years)	HCE = Highly corrosive environment [B)]	NR = Not recommended

[A)] System particularly suitable for heavy or long nails for permanent works where one of the two protective layers may become damaged during handling or installation. This approximately equates to double corrosion protection required for permanent anchors.

[B)] As defined in BS EN 14490:2010.

4.7 Design of facing

4.7.1 Introduction

Most soil nailed structures should have a facing. The type of facing adopted by the designer may depend on both structural and aesthetic considerations.

Structural considerations: except in only marginally unstable shallow slopes it is unlikely that the nail bond stress through the active zone will alone be sufficient to resist the disturbing forces; in most cases some structural contribution from the facing should be relied upon.

The facing may be relied upon to provide this structural contribution either through discrete elements such as nail plates, which might only require the addition of a soft facing, or by a continuous structural flexible or hard facing spanning between nails, or by a combination of nail plate and flexible facing.

Aesthetic considerations: Many early soil nailed structures were constructed with sprayed concrete faces which may be considered inappropriate for many potential applications. More recently a variety of methods have been developed, which may be used to provide vegetated faces, particularly to slopes of 60° or less while other proprietary hard facings have also been developed, which may be used to offer a variety of alternative finishes.

4.7.2 Type of facing

4.7.2.1 Soft

Soft facings should not be used to perform a long term role but may be provided to stabilize the slope surface while vegetation establishes itself. Their main function may be to retain a topsoil layer and prevent erosion. Soft facings should not be used for slopes that are steeper than where the soil forming the slope would be naturally stable when protected from weathering. However, the need to protect topsoil from wash off in the short term may limit most soft face installations to 45° or less. Many types of geogrids, cellular geofabrics and light metallic mesh may be used, including degradable coir mats. The solution chosen should depend on the steepness of the slope and the type of vegetation envisaged.

4.7.2.2 Flexible

Flexible structural facings may be used to provide long-term stability of the face by supporting the soil between nail locations and transmitting the load from the soil to the soil nails via the nail heads. Flexible facing may be used to provide support through the mobilization of tensile forces within them and therefore some deformation is required in order for a component of these forces to act normal to the face. Flexible facings are not normally recommended for permanent slopes in excess of 70° (60° for complex flexible faces; see **4.7.6.4.3**).

Materials used often comprise coated metallic meshes of either the woven or welded types which are considered "continuous" in the design process; detailed consideration should be given to the jointing of these materials (either by physical connections or overlaps) in order to ensure continuity. Edge and termination details should also be considered carefully.

The concentrations of load around the nail plate should be given particular attention to ensure that adequate resistance to punching and rupture is provided.

NOTE Geosynthetic materials rarely meet these criteria.

4.7.2.3 Hard

Hard facings may be used to provide the same function as flexible structural facing but require far less deformation in order to mobilize their strength and may be designed for faces up to 90°. Hard facings may be formed from: reinforced concrete (sprayed, cast in situ or pre-cast panels), crib work or gabions. Existing retaining walls which are to be strengthened/stabilized by soil nailing may also be considered to act as a hard facing.

Where hard facings are largely impermeable they may require additional drainage arrangements beyond that which might be required for a more permeable face.

4.7.3 Materials

NOTE Third-party certification of materials is accredited by UKAS (www.ukas.com) in the UK and members of the IAF (www.iaf.nu) in the rest of the world. For example, BBA and BRE are UKAS accredited.

4.7.3.1 Nail plates

NOTE 1 Nail plates are usually square and fabricated from mild steel plate to a size and thickness dictated by structural and durability considerations.

Doming around the central hole of a nail plate may be provided to assist nail/plate articulation.

NOTE 2 Proprietary plastic products are also available which offer advantages in terms of durability and chemical resistance.

4.7.3.2 Concrete

Concrete facings should conform to the requirements of BS EN 14490:2010 and be designed in accordance with BS EN 1992-1-1.

4.7.3.3 Metallic meshes

Welded and woven wire meshes should conform to the requirements of BS EN 14490:2010.

4.7.3.4 Polymeric materials

Particular consideration should be given to UV resistance and strain characteristics of synthetic materials used at the face.

4.7.3.5 Other facings

Other facing types are permissible provided they meet the engineering and durability requirements of the design.

4.7.4 Finishes

4.7.4.1 Concrete

Concrete finishes may be in situ, cast or sprayed, or precast in the form of modular blocks or panels.

NOTE A variety of surface finishes are available.

4.7.4.2 Masonry

Masonry may be provided as a finish to hard concrete facings but adequate consideration should be given to deflections.

4.7.4.3 Stone

Dry stone finishes may be achieved by the use of gabions or mesh panels filled with stone. Angular crushed stone should be selected that has the necessary durability and frictional characteristics to limit the imposed force on the facing.

4.7.4.4 Cribwork

Both timber and pre-cast concrete crib walls may be used to provide effective facings which may also be vegetated.

4.7.4.5 Vegetation

4.7.4.5.1 General

In the great majority of cases vegetation should provide the most acceptable finish to earthworks slopes and can make a significant contribution to the stability of these slopes. Vegetation should be chosen to reflect the surrounding environment and may include grasses and shrubs but not new trees which may lead to damage to the face.

Particular care should be taken when choosing vegetation type, face angle, method of seeding and the soil medium at the face given the often hostile environment in which these slopes are constructed.

Vegetation may routinely be established on slopes up to about 45°, but landscaping specialists should be involved for slopes steeper than 45° and up to 60° to maximize the likelihood of satisfactory establishment of vegetation.

4.7.4.5.2 Role of vegetation

The various roles of vegetation may be categorised as mechanical, hydrological, thermal, ecological or environmental with the beneficial effects being protection, stabilisation and water removal.

Vegetation may be selected to enhance the performance of a soil nailed slope by one or more of the following:

a) providing protection against UV radiation to any geosynthetics in the face;

b) controlling erosion by shielding against impact of weather, by acting as a reservoir, as a soil binder through root action and a barrier to downward movement of debris;

c) through evapotranspiration;

d) increasing shear strength of the retained soil by reduction of pore water pressures;

e) increasing the shear strength of the retained soil by root reinforcement.

4.7.4.5.3 Plant characteristics

The suitability for particular plant types is dependent on a number of chracteristics:

a) vigour;

b) resistance to erosion;

c) drought-tolerant;

d) ability to grow in poor infertile soils;

e) ability to grow in shady conditions.

Planting should be chosen for its ability to withstand waterlogging for aquatic applications.

4.7.4.5.4 Site appraisal

It is vital that the relevant characteristics and conditions of a site are taken into account when designing the vegetation for the face. In difficult or hostile environments, such as adjacent to a motorway where de-icing salts might be present, the following additional information may be required:

a) site observations: geology, hydrology, microclimate;

b) site investigation and analysis: grading, pH, nutrient levels, contaminants.

4.7.4.5.5 Plant types

The most common type of vegetation that is adopted is grass. Species that are drought resistant and low growing are generally used. A mixture of species is likely to increase the probability of success i.e. grasses and legumes as these will provide rapid growth and erosion protection.

It is recommended that advice is sought from a specialist consultant with respect to the appropriate species.

4.7.4.5.6 Planting and seeding

4.7.4.5.6.1 Grass

Establishment of grass may be achieved in a number of ways.

a) Turf can be supplied with a range of seed mixes and sizes to suit particular site conditions and construction methods. Reinforced slopes with a face angle up to 45° do not generally require a structural facing and in such situations the turf can be placed directly on a prepared topsoil surface although it will be necessary to pin it in place. As the face angle increases the turf is retained behind some form of grid and is required to be smaller in section.

b) Seeded topsoil, either placed within erosion matting materials, contained behind a mesh or in degradable bags behind a wrap-around face is also common.

c) Pre-seeded mats, blankets or meshes are also used to provide short-term protection to the face until the vegetation establishes. Some meshes will remain durable and provide longer-term assistance to the vegetation in situations where the erosion forces are significant.

d) Hydroseeding has also been used successfully with careful specification and a sufficient thickness of carrier material.

4.7.4.5.6.2 Discrete planting

The planting of individual plants on soil nailed slopes may be achieved by planting in prepared holes. The structural facing should not be cut unless this has been specifically allowed for in the design.

4.7.4.5.7 Design considerations

The following factors should be considered when deciding on the vegetation selection.

a) *Direction of slope.* The direction in which a slope faces is significant as south and west facing slopes receive more direct sunlight and so ambient temperatures can be high and vegetation growth difficult.

b) *Prevailing wind.* The direction of the prevailing wind and its moisture carrying capacity also need to be considered.

c) *Drought.* Some species have longer roots which will survive drought better than shallow rooted species. In particular circumstances irrigation may form part of the solution and allow a wider range of vegetation to be used although there are cost implications.

d) *Growing season.* The growing season for vegetation in the more northern areas of the UK may require installation of the permanent vegetation to be carried out at a different time to the main works. This may influence the construction programme and require temporary measures to be considered.

e) *Soil type.* The type of soil has a large influence on the types of vegetation that can thrive on the soil nailed slopes. Sandy soils are prone to dessication and may need special measures to ensure that moisture is present whereas clay soils can be resistant to root penetration.

4.7.4.5.8 Maintenance

It is generally found that vegetative ground cover requires little maintenance although continuity of cover and any cutting of vegetation need to be considered.

For soil nailed slopes the most important recommendation is to ensure continuity of the cover as this greatly assists in maintaining integrity of the soil.

It is usual to only cut vegetation at the bottom of slopes where sight lines need to be maintained.

4.7.5 Design of nail plates for soft or flexible faces

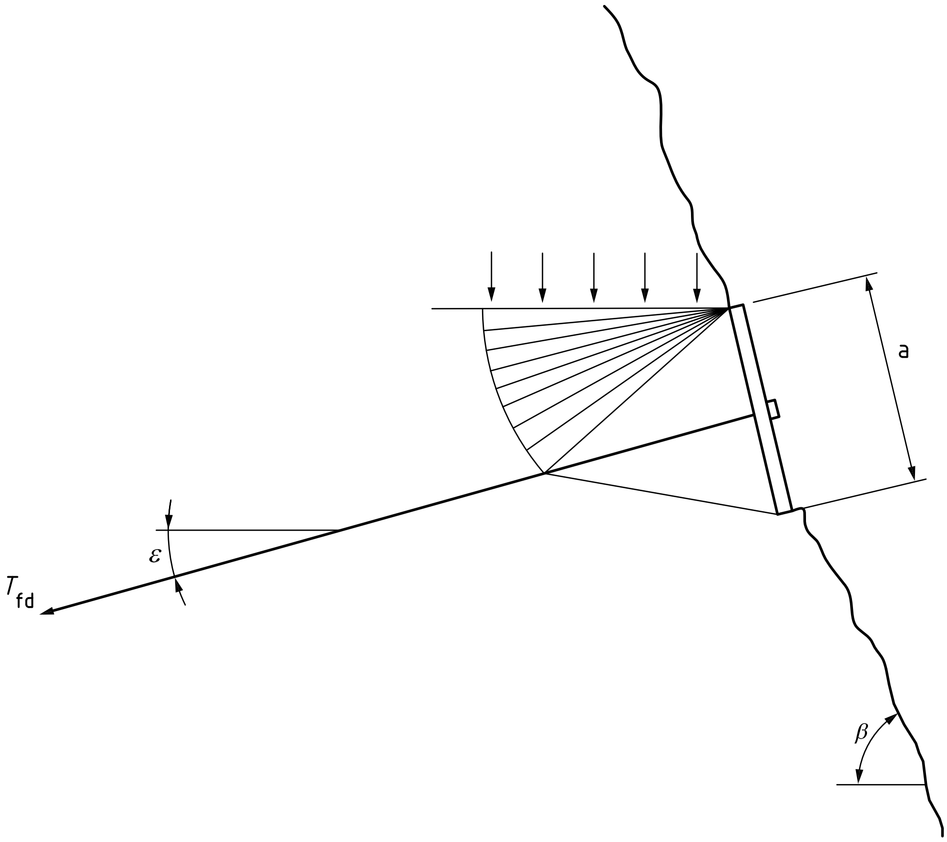
The primary function of the nail plates is to ensure that the required nail force, T_f assumed in the internal stability analyses (**4.2.1** and Figure 21) may be mobilized at the front face. A method for calculating the required nail plate size for a given nail force is set out in Figure 30 (adapted from HA 68/94 [23]).

NOTE Front face plate bearing tests may be undertaken to measure the plate capacity directly.

The equation for a_{req} in Figure 30 ignores any contribution from the front facing acting between the nail positions.

Figure 30 **Calculation of required nail plate size for a given design nail force**

All angles in radians.



$$a_{req} = \left(\frac{T_{fd}}{\eta_d}\right)^{\frac{1}{3}}$$

where

$$\eta_d = \frac{\gamma_d (1-r_u) \tan\beta \exp\left[3\left(\frac{\pi}{4} - \frac{\varphi_d}{2} + \varepsilon\right)\tan\varphi_d\right]}{2\cos\left(\frac{\pi}{4} + \frac{\varphi_d}{2}\right)(1-\sin\varphi_d)}$$

T_{fd} is the design nail force (kN per nail) assumed to exist at the front end of the nail during the internal stability analysis (**4.2.1**)

4.7.6 Design of the front facing

4.7.6.1 General

The primary function of the front facing should be to restrain the surface soil between nail plate positions. The front facing system may be either soft, hard or flexible.

Consideration should be given to factors which might affect the face specifically including swelling, steep relic shear planes and the effects of ground and surface water.

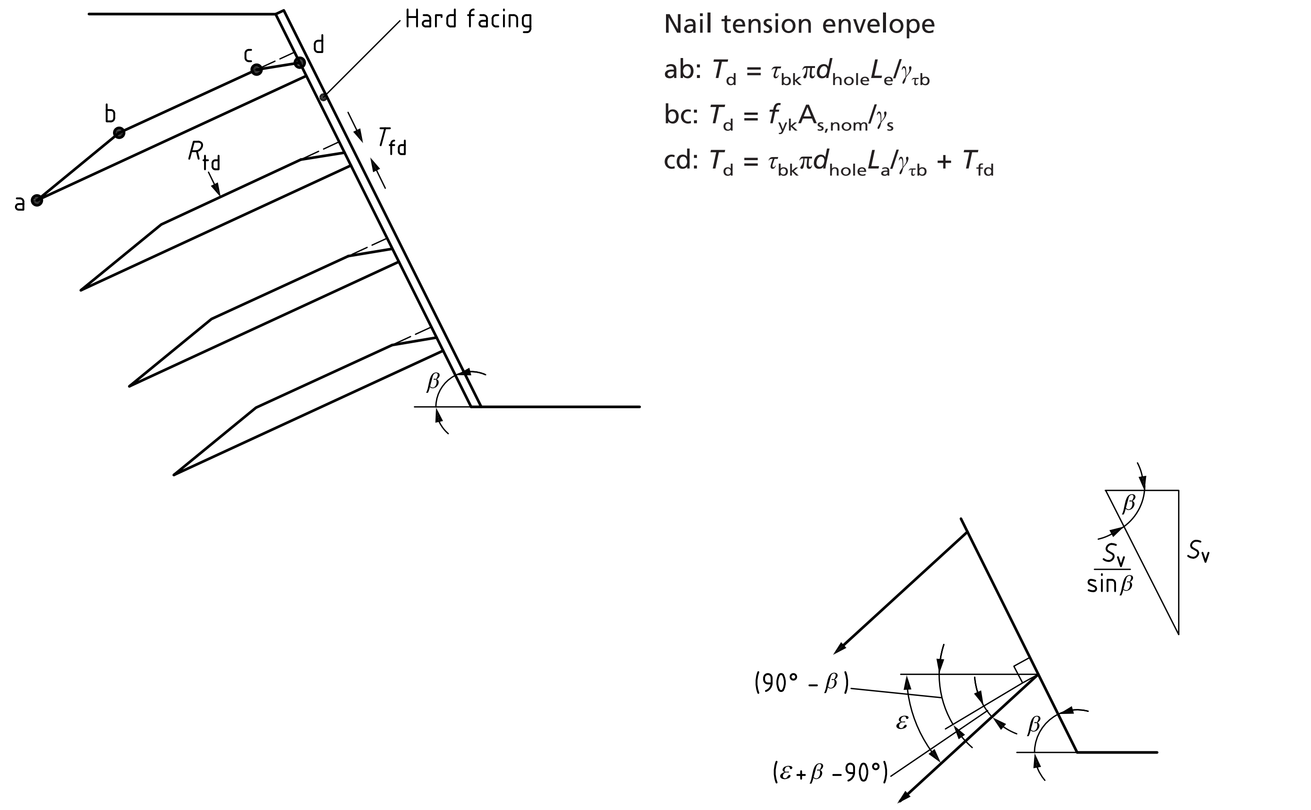
4.7.6.2 Soft facings

In soft facing systems, the facing does not perform a structural role. The soil between the nail plates may be assumed to be self supporting. Soft facings may nevertheless perform a valuable role in limiting erosion and/or promoting the growth of vegetation.

4.7.6.3 Hard facings

In hard facing systems, the facing may be assumed to provide an effectively rigid restraint across the whole face. As such, the hard facing not only restrains the soil between nail locations but also does the job of the nail plates. The facing should be designed for a uniform pressure based on T_{fd} divided by the face area per nail, as set out in Figure 31.

Figure 31 **Calculation of design loading acting on the rear of hard facing**



For hard facings, the design of the front face is to a uniform normal pressure of:

$$p_{hd} = \frac{T_{fd}}{S_v S_h} \sin\beta \sin(\beta + \varepsilon)$$

(or p_{hd} is the clay swelling pressure if greater).

The design value T_{fd} to be used for the design of the hard facing should correspond to the value assumed to be mobilized at the front face during the internal stability analyses (**4.2.1** and Figure 21), as illustrated in Figure 31.

NOTE The value of T_{fd} is typically taken to be between 60% and 100% of R_{nd}, depending on the nail spacing, where R_{nd} corresponds to the design tendon strength. (Clouterre [19]).

Punching resistance at the nail heads should also be checked and it is recommended that the design tendon strength R_{nd} of the nail be used as a worst case in this calculation, regardless of the value of T_{fd}.

Where hard facing systems are used, nominal nail plates may still be required in the short term in order to provide adequate face stability during construction, before installation of the hard facing.

4.7.6.4 Flexible facings

4.7.6.4.1 General

Flexible facing systems represent an intermediate solution, mid-way between soft and hard facings. Flexible facing systems perform a structural role, but some deformation (often considerable deformation) is likely to be required before their design strength can be mobilised. The design of these systems is consequently more problematic than for soft or hard facings and certain restrictions apply on when they can be used.

Flexible facing systems may be either "simple" or "complex".

4.7.6.4.2 Simple flexible facings

A simple flexible facing restrains the surface soil between the nail plate positions only. The nail plates continue to be designed according to Figure 30 for the assumed design strength of the nails at the front face, without any contribution from the flexible facing. But the surface soil between the nail plates is not self supporting and the facing must be designed to restrain it. A method for calculating the loading to be resisted by the facing based on a two-part wedge failure mechanism forming between two rows of nails is given in Figure 32.

The flexible facing may be designed to meet this loading by assuming that it acts as a tensioned catenary as shown in Figure 33.

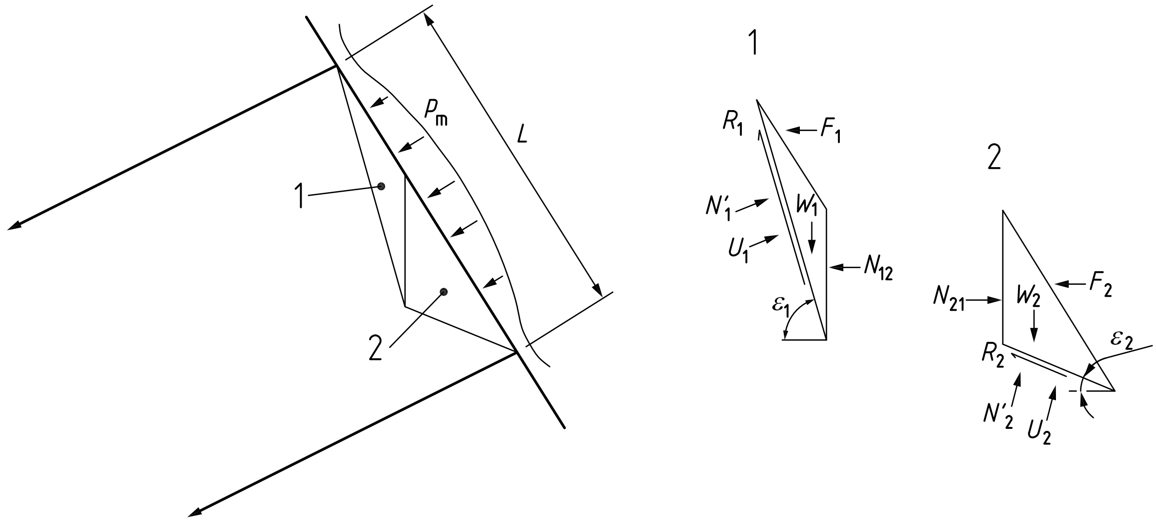
NOTE δ_v represents the inevitable vertical shortening of the slope which will occur during its design lifetime (say 0.1% L_0 to 0.2% L_0). This acts to compromise (i.e. reduce) the tension in the catenary.

An approximate allowance for the 3D nature of the nail head locations may be incorporated by defining the effective catenary span L_0 as $\sqrt{(S_v^2 + S_h^2)}$ instead of simply S_v.

The maximum horizontal deflection should be kept within serviceability limits. A maximum deflection of $0.02L_0$ is recommended.

The front facing should also be designed to resist punching shear around the perimeter of the nail plate.

Figure 32 **Calculation of design loading acting on a simple flexible facing**



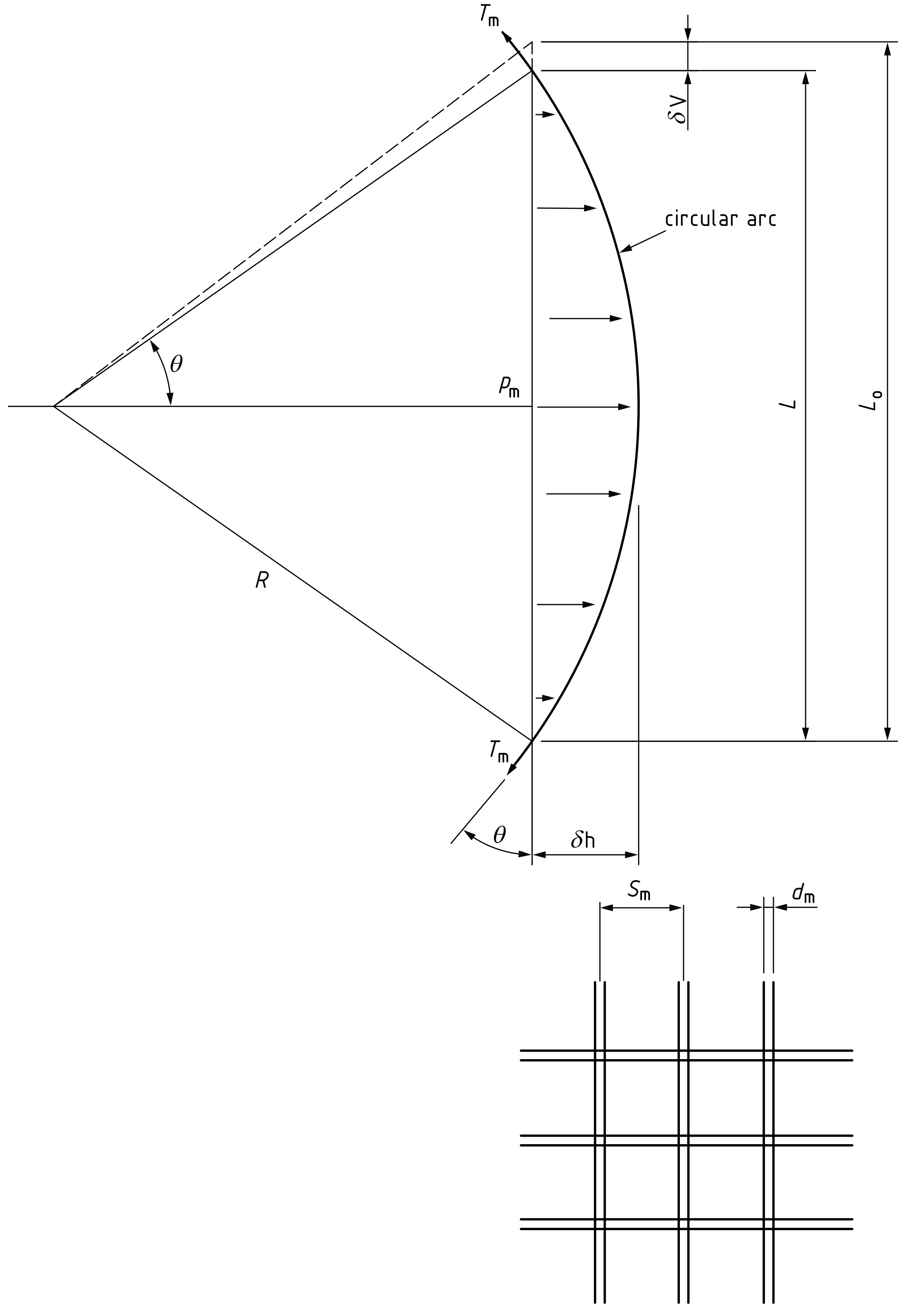
$$(F_1+F_2)_d = \frac{W_1(\tan\varepsilon_1 - \tan\varphi_d) + \dfrac{U_1\tan\varphi_d}{\cos\varepsilon_1}}{(1+\tan\varepsilon_1\tan\varphi_d)} + \frac{W_2(\tan\varepsilon_2 - \tan\varphi_d) + \dfrac{U_2\tan\varphi_d}{\cos\varepsilon_2}}{(1+\tan\varepsilon_2\tan\varphi_d)}$$

at the worst combination of ε_1 and ε_2

C1 $L = \sqrt{S_v^{\,2} + S_h^{\,2}}$ C1

C1 $\Rightarrow p_{md} = (F_1+F_2)/L$ C1

Figure 33 **Calculation of tension and deformation in flexible facing for a given design loading**



Method

Based on the properties of a circle.

Select mesh (d_m, S_m) such that required p_{md} (from Figure 32 or Figure 34) may be met at acceptable δh.

Figure 33 **Calculation of tension and deformation in flexible facing for a given design loading**

$$2R\delta h = \frac{L^2}{4}, \theta = \sin^{-1}\frac{L}{2R}$$

C1⟩ $$\varepsilon_m = \frac{2R\theta - L_0}{L_0}, T_{md} = \frac{\varepsilon_m E_m \frac{\pi d_m^2}{4}}{\gamma_s}$$ ⟨C1

C1⟩ *NOTE E_m is Young's modulus for the mesh material.* ⟨C1

$$p_{md} = \frac{2T_{md}\sin\theta}{LS_m}$$

where

C1⟩ $$L_0 = \sqrt{S_h^2 + S_v^2}$$ ⟨C1

$$L = L_0 - \delta v$$

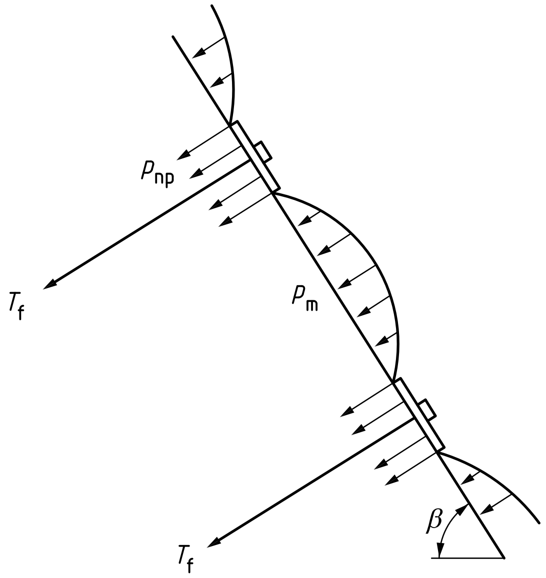
$\delta h \leq 0.02L_0$, say.

4.7.6.4.3 Complex flexible facings

Complex flexible facings are assumed not only to restrain the soil between nail positions, but also to contribute to the nail plate capacity. This option may be used when the nail plate size calculated according to Figure 30 would otherwise be excessively large. It is emphasized that this option will involve greater deformation of the front face than a simple flexible facing in order to mobilize the additional necessary catenary tension. If this deformation is unacceptable, then a hard facing should be chosen. Complex flexible facings should not be used for permanent slopes steeper than around 60°, or for permanent slopes where the maximum nail spacing (S_v or S_h) exceeds approximately 1 m without rigorous analysis to show that they will perform as intended.

Such a method for designing complex flexible facings is given in Figure 34, which provides a face pressure p_m such that the required value of T_f may be mobilized in the nails for internal stability.

Figure 34 **Requirements of a complex flexible facing**



$$T_{fd} = p_{npd}a^2 + \left(S_v S_h - a^2\right)p_{md}$$

where

T_{fd} is the value used in internal stability analysis;

p_{npd} = $a\eta_d$ from Figure 30;

p_{md} is given in Figure 33.

4.7.6.5 Swelling

Significant additional forces might be applied to the face due to swelling particularly in newly formed clay slopes where reduced horizontal constraint can lead to negative pore pressures at the face and consequent swelling; the effect of the assessed volume change on the proposed facing system should be considered (see Figure 31). Seasonal shrinking and swelling of clay slopes should also be considered particularly in high plasticity clays where a substantive facing system might be required to protect the slope from seasonal effects.

4.7.6.6 Self weight

NOTE The self weight of facing systems can be substantial and the "top down" construction method can result in facings "hanging" from nails in the short term.

The mass of the facing is beneficial in resisting soil pressures particularly on shallower slopes but high shear/tensile forces might be transmitted to the nails and bearing failure on the underside of nails should be considered particularly if the rear of the facing is smooth, steep and unsupported at the base.

4.7.7 Durability

NOTE With water, oxygen, organic matter and salt all likely to be in regular contact the conditions prevailing at the face of a soil nailed structure are likely to be significantly more onerous then those applicable to buried components.

Assessments may be made on the basis of the factors given in **4.6**. However facings are more readily inspected, maintained or replaced than other components and this should be considered when assessing any risk.

4.7.8 Structural elements

Structural elements of the facing should be assessed for the required design life of the structure.

4.8 Drainage

Groundwater represents a major hazard for soil-nailed slopes and walls, as for most earth-retaining structures; appropriate drainage should be installed to manage and control water flows (see Figure 35).

Consideration should be given to possible long-term increases in water pressure in clay cuttings due to recovery of suctions.

Suitable whole-life monitoring and maintenance should be carried out.

Longevity of the drainage system and the need for maintenance during the design life of drainage system need to be considered in the design and appropriate materials should be used. The design should also consider problems associated with replacing drainage during the design life.

Typical types of drainage that may be used include:

a) surface water interceptors (see Figure 36);

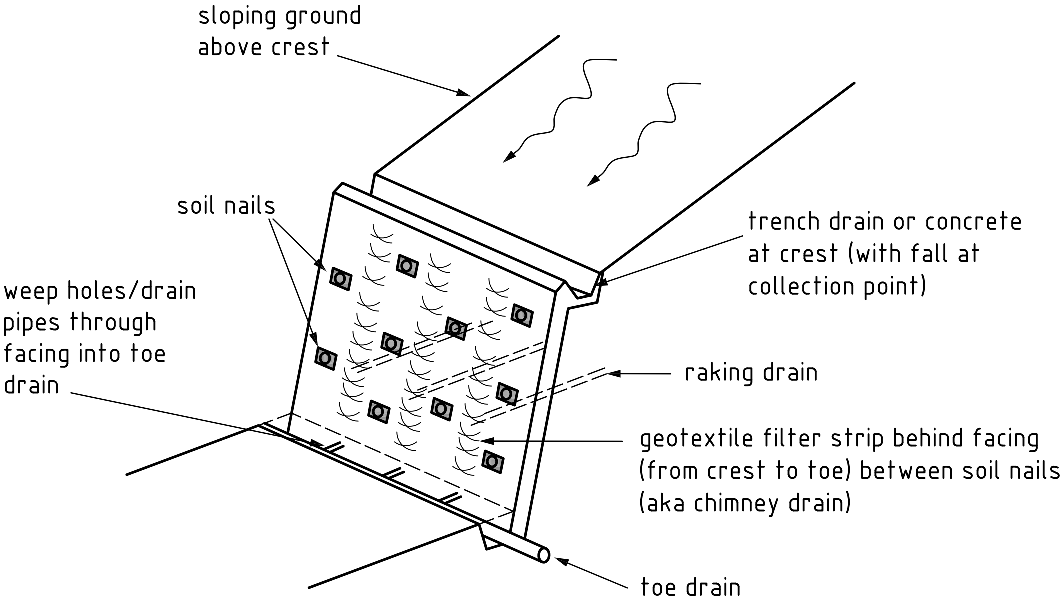
b) slope face drainage;

c) sub-surface (e.g. raking drains) drainage (see Figure 37);

d) weep holes through facings (see Figure 38).

NOTE Variants of item a) include lined drainage channels, French drains, bunds, sumps and appropriate pavement drainage systems.

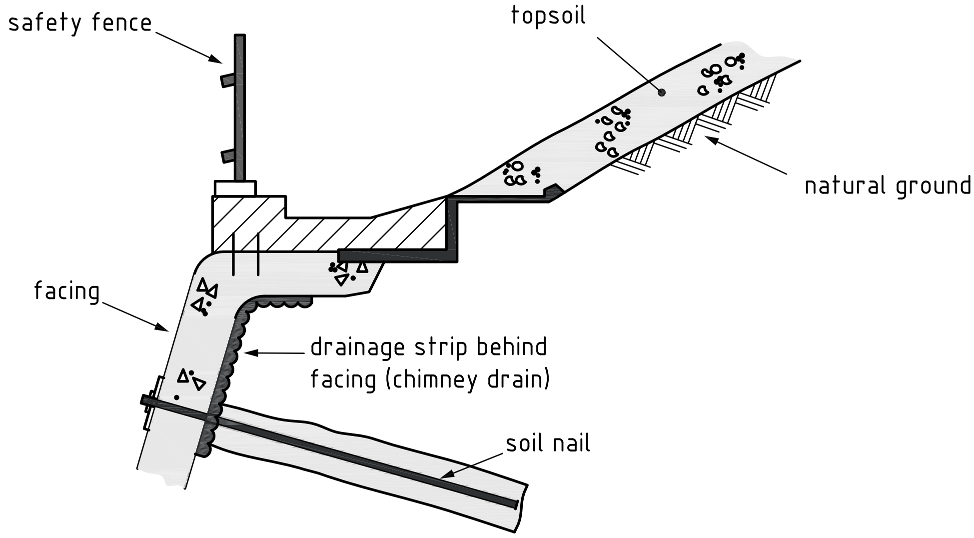
Figure 35 **Typical types of drainage for soil nailing**



Based on CIRIA C637 [1].

NOTE Drainage strips are shown but weepholes can be used as an alternative.

Figure 36 **Typical surface water interceptor detail above a steep soil-nailed slope**



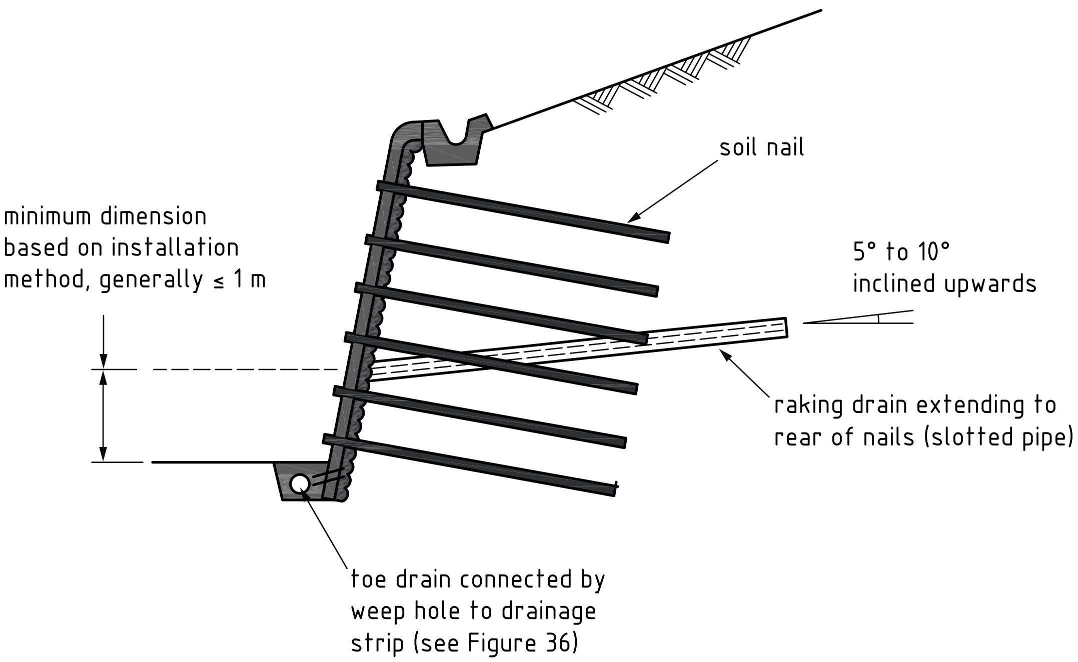
Based on CIRIA C637 [1].

NOTE 1 The concrete channel size has to match the design water flow.

NOTE 2 A crest drain ought to always be considered for permanent soil nail applications with a slope behind.

NOTE 3 For safety reasons, it is recommended that drainage and crest details are installed prior to excavation wherever possible.

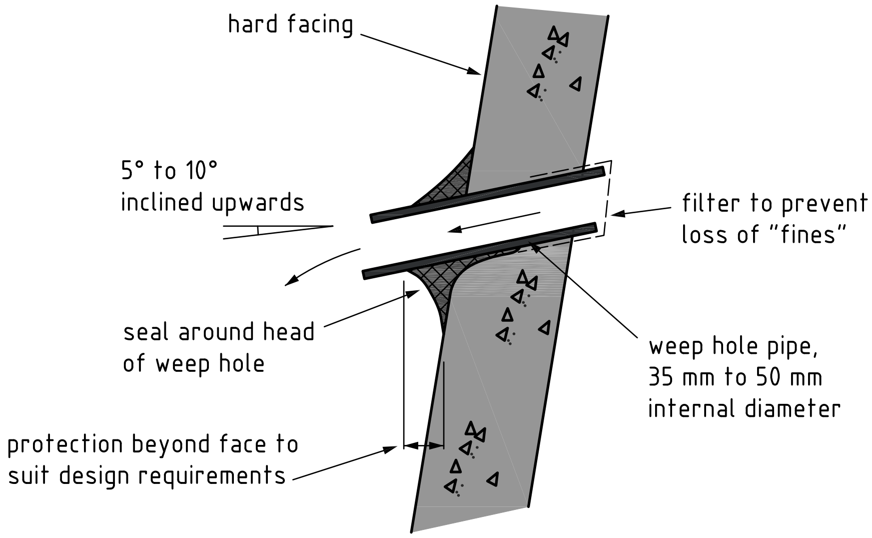
Figure 37 **Example of a raking drain in a steep soil-nailed slope**



Based on CIRIA C637 [1].

NOTE One or more levels of raking drains may be used.

Figure 38 **Typical detail for a weep hole in a steep soil-nailed wall**



NOTE Weepholes in hard facings are typically spaced at one per 2 m² to 10 m².

Section 5 : Serviceability and movements

5.1 Serviceability limit state

For each geotechnical design situation it should be verified that no serviceability limit state, as defined in BS EN 1990, is exceeded.

*NOTE 1 The serviceability limit states are defined as: "states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met" [C1) (BS EN 1990:2002, **1.5.2.14**). (C1]*

Serviceability limit states should be established prior to the design for the soil nailing being undertaken.

NOTE 2 Due to the complex interaction between the soil nails, the surrounding ground and the facing, which is three dimensional in nature, precise calculation of deflection is not straightforward and is rarely warranted except for particularly sensitive situations.

For steep faces estimation of likely deformation may most easily be achieved with reference to empirical relationships derived from field observations.

For less steep face angles tending to normal slope gradients prevention of the occurrence of SLS conditions may be achieved by limiting mobilized shear strengths.

For each design situation it should be verified that the serviceability limit state will not be exceeded for:

a) the soil nail structure or soil nail slope under consideration including all constituent elements, which can include:
    1) clearances of tolerances relating to the final position of the soil nail facing;
    2) crack control in the nail grout and/or concrete facing;
b) adjacent structures, services, etc., that might be influenced by movements generated by excavation or soil nailing construction.

The importance of maintaining the aesthetic appearance of the structure over the design life should also be considered.

Ground movements and element deflection should be considered, taking into account the range of variation in ground conditions that could be reasonably anticipated over the area of soil nailing under consideration:

a) during construction (particularly where top down construction is proposed);
b) for the design life following completion of construction; and
c) for transient or accidental conditions.

The influence of movement or deflection on the durability of elements should receive particular attention (e.g. cracking of grout and creep of polymeric materials).

Estimation of movement may be based on:

a) empirical relationships;
b) numerical modelling;
c) case studies.

5.2 Serviceability limit state analysis

For routine situations where limit equilibrium design methods are adopted direct prediction of deformations are not usually possible; in such situations deformations may be predicted approximately using empirical correlations.

Where critical situations require greater confidence in predicted deformations, a high level of analysis such as numerical analysis may be adopted. Where SLS analyses are undertaken characteristic values of parameters should be used (i.e. partial factors = 1.0).

5.3 Estimation of movement – General

Consideration should be given that actions causing movement can be due to the following combinations of factors:

a) self weight effects (particularly where top down construction is proposed);

b) application of external forces or loads (for example water pressures, other attached structures, surcharges, accidental impacts);

c) time dependent phenomena (for example swelling/heave in cohesive soils);

d) seismic forces;

e) sloughing of soil between nail heads.

To mobilize forces in passive soil nails and facings movement is required; pre-stressed soil nails may be effective where limiting movements is a prime objective.

Soil nailing is a three dimensional problem and this should be considered when estimating movements.

Estimation of movement may need to consider:

1) magnitude of movement;

2) rate of movement.

Consideration should also be given that movements depend on the following factors:

i) construction rate;

ii) nail spacing, length and inclination;

iii) excavation height;

iv) nail and soil stiffness;

v) facing type and stiffness;

vi) compressibility of foundation soils;

vii) magnitude and location of externally applied loads or forces.

5.4 Use of empirical relationships

*COMMENTARY ON **5.4***

Where the soil nailing has a steep or vertical face constructed in a top down sequence the soil nailed zone tends to rotate outwards and downwards at the top as construction and excavation proceeds. With each excavation stage the soil nails progressively strain and elongate as they mobilize load. In this situation the movements occurring during construction are usually the most significant.

Further time dependent deformation can take place following excavation to the lowest level as stress redistribution occurs within the nailed block, which can also be associated with stress relief induced swelling, heave or application of a surcharge.

Schlosser and Unterreiner [34] suggest for steep faces with hard facings, the magnitude of vertical and horizontal deflection are similar at the top of the wall and range between 0.1% and 0.3% of the wall height, as shown in Table 10.

From Schlosser and Unterreiner [34], horizontal displacement behind the nailed zone may be estimated by:

$$\delta_0 = k(1 - \tan\varphi)H$$

where values of *k* are given in Table 10.

NOTE The movements of soil nail structures are similar to other types of retaining structure.

Table 10 **Displacements at the top of steep soil nailed structures**

From Clouterre [19]

	Soil type		
	Weathered rock/stiff soils	**Sandy soils**	**Clayey soils (not high or very high plasticity clays)**
$\delta_h = \delta_v$	*H*/1000	2*H*/1000	3*H*/1000
Coefficient *k*	0.80	1.25	1.50

A further check on anticipated deflection may be made by considering the strain distribution and hence nail elongation at working loads. Where the workings strain in the nail is small (for example where the tendon bar diameter is large or the face angle is not steep) strain within the nail tendon may be assumed to be minimal and the limitations of this approach should be considered. Where nails are installed to improve the factor of safety against slope failure the force mobilized in the nails under serviceability conditions may be assumed to be very low and hence elongation of the soil nails at working loads might be very small.

In addition to nail elongation it should be remembered that deformation of the ground is still required before the nails mobilize load and this will contribute to overall deformation.

Because of these limitations simple calculations methods should be used with care when making predictions of deformation for existing slopes.

As with un-reinforced slopes the occurrence of the serviceability limit state may be avoided by:

a) limiting the mobilized shear strength;

b) observing movements and specifying corrective actions to reduce them.

Infrastructure embankments should be given special consideration, particularly those carrying railways where deformation control might be a critical requirement; in such cases, deformation affecting the embankment might be the result of movement within the body of the embankment or the result of progressive movement of surface material (termed ravelling). Careful investigation to find the cause of the apparent movement is essential before an evaluation to decide the suitability of soil nailing to address the deformation mechanisms is made.

The application of prior experience, careful detailing and reference to case studies should be used to provide the principal means of estimating likely performance.

5.5 Numerical modelling

For particularly complex problems or where the estimation of serviceability deformations is critical numerical modelling may be undertaken, for example using finite element or finite difference methods. See also **4.4**.

NOTE 1 The accuracy of numerical modelling depends on many factors including the constitutive framework for modelling the soil, the nails and the facing and the interaction between the various elements.

Where there is uncertainty over the best way to formulate numerical models careful calibration with other experience, supplemented with construction monitoring should be undertaken if deformations predicted by numerical analysis are to be relied on.

NOTE 2 It is beyond the scope of this standard to provide detailed advice on the essential requirements to be considered when undertaking numerical modelling.

5.6 Case studies

Case studies of measured behaviour may provide a useful guide to anticipated performance of new construction.

Historic information should be validated to ensure similarity of:

a) geometry;

b) nail type and installation method;

c) soil type;

d) external loading regime;

e) construction sequence;

f) facing type.

Section 6 : Design verification

6.1 Testing

6.1.1 Pullout testing

Selection of characteristic parameters for design should be recognized as critical to ensure adequate and safe structures and slopes. Critical parameters subject to high variability should be derived from site specific tests; in particular the pullout resistance between the soil nail and the ground is a fundamental parameter that strongly influences the design of soil nail applications. In most cases it is not possible to fully replicate the stress distribution that will occur in a production nail by undertaking pull-out tests, therefore when designing nail tests and analysing the results, aspects that might lead to differences in performance or behaviour of the production nails compared to the test nails should be considered, for example:

a) differences in the ambient soil stresses due to differences in location, orientation, depth of embedment and other boundary effects and the way these differences might affect the mobilized bond stress in the production nails;

b) variation in magnitude and direction of bond stresses mobilized in pull-out tests and in production nails;

c) variation in the soil strata between the test nail and production nail locations.

In most routine situations pull-out tests remain the only viable means to evaluating nail performance in spite of the limitations of the method. Due to the mobilization of the strain field in a test nail subject to a pull-out test being significantly different from production nails, pull-out tests on production nails should be avoided being confined to sacrificial nails only (see **4.3**).

Tests should be spaced to give representative spread across the soil nailed structure.

6.1.2 Types of soil nail test

NOTE BS EN 14490:2010 covers the pullout testing of soil nails.

Two types of soil nail test that may be used are defined in BS EN 14490:2010:

a) *Sacrificial test*

Sacrificial tests may be used to verify the ultimate bond resistance used in the design.

If these tests fail to provide verification of the relevant parameters adopted in the design a review of the soil nail installation and/or the soil nail length and layout should be carried out.

These tests may be done significantly in advance of the main construction works and are usually undertaken before the detailed design is completed. It is essential that such design investigation test nails are installed by the same construction methods and in similar ground conditions as the proposed production nails.

b) *Production test*

Production tests may be used to demonstrate satisfactory soil nail performance at the accepted load.

Soil nail tests should be selected using Table 11, which shows the purpose of each of the types of soil nail test.

Soil nail tests should be performed in accordance with Table 12, which shows:

1) type of test to be undertaken;

2) suggested test frequency.

Table 11 **Type of soil nail test (from BS EN 14490:2010)**

	Sacrificial nail test	Production nail test
Purpose of test	To verify the ultimate soil nail to ground bond resistance used in the design: a) the bond in the passive zone; b) the bond in the active zone; c) the bond along the entire length of the nail.	To demonstrate satisfactory soil nail performance at a load designated by the designer. The test is performed on the entire length of the nail.
When tested	Before, during or after production works.	During or on completion of production works.
Type of nail used	Sacrificial	Production
Action taken in case of non-compliant test result	Review soil nail installation method and/or consider alternative soil nail length and layout.	Consult designer for action to be taken and approval to continue.
Comments	If necessary at each different soil layer.	Caution should be exercised when testing production nails not to overstress the nail to grout bond or cause damage to corrosion protection. When a structural facing is used the test nail should be debonded within the zone of influence of the facing.

Table 12 **Recommended test frequency (from BS EN 14490:2010)**

Test type	**Suggested minimum frequency of load tests**	
	Sacrificial nail test	**Production nail test**
Geotechnical Category 1: negligible risk to property or life.	Optional	Optional
Geotechnical Category 2: no abnormal risk to property or life.	If no comparable experience of soil type: a minimum of three sacrificial nails with at least one sacrificial nail per soil type. Where direct experience exists then sacrificial nail tests are optional.	2%, minimum of three tests. *These criteria are subject to a minimum of one test per soil type and per excavation stage.*
Geotechnical Category 3: all other structures not in Category 1 or 2.	A minimum of five sacrificial nails with at least two sacrificial nails per soil type.	For number of nails: 3%, min. five tests. *These criteria are subject to a minimum of one test per soil type and per excavation stage.*

NOTE 1 Geotechnical Category of structure as defined in BS EN 1997.

NOTE 2 Test nails should be evenly distributed throughout the structure.

NOTE 3 The frequency of testing is a suggested minimum.

NOTE 4 Where sacrificial nail tests are carried out the number of production nail tests can be reduced on a pro-rata basis.

NOTE 5 For spacing, less then 0.8 m, a group test of four nails is recommended.

Any element or interface may be subject to testing, for example:

a) soil-nail bond stress;

b) head plate bearing capacity;

c) facing rupture strength;

d) reinforcement/grout bond;

e) grout/concrete strength.

6.2 Nail pullout resistance

6.2.1 Static nail test procedure

The soil nail bond stress is fundamental and should be verified to validate the design.

Generally static tests should be performed whereby the test nail is loaded and unloaded incrementally in cycles with the load held constant for each load increment. For each load increment displacement should be measured at various time increments, e.g. t = 0 min, 1 min, 2 min, 5 min, 10 min, 15 min, 20 min, 60 min, until the displacement is less than 0.5 mm between two consecutive readings. In addition the following minimum load hold periods should be adopted:

a) for the maximum test load P_{test}, 10 min;

b) for intermediate test loads, 1 min.

If the displacement rate of 0.5 mm between two consecutive readings cannot be achieved for a given test load no further load should be applied and the residual test load recorded. The ultimate load P_{ult}, should be the minimum of the residual load or the minimum test load at which the rate of displacement was less than 2 mm per log cycle of time:

$$\frac{s_2 - s_1}{\log(\frac{t_2}{t_1})} < 2 \text{ mm}$$

where s_1 and s_2 are the measured nail displacements at time t_1 and time t_2 respectively.

Table 13 gives recommendations for the static load testing of soil nails. The soil nail designer should, in addition, specify:

a) the number of load cycles;

b) the total test load;

c) the magnitude of the load increments.

NOTE These relatively fast rates of testing might not be appropriate for long-term performance of nails in stiff clay.

Table 13 **Criteria for static loading of soil nails**

Test type	**Sacrificial**	**Production**
Estimation of maximum test load	The value of P_{test} shall be based on the value of design bond resistence T_d (or working bond T_w), the partial factor γ_d (normally in the range 1.5 to 2.0) and an appropriate value for the factor ξ_γ	The value of P_{pr} shall be based on either the design bond resistance T_d or the working unit bond resistance T_w multiplied by a proof factor *k*, which normally lies in the range 1.1 to 1.5. The value *k* should never exceed the design partial factor γ_d to minimize the risk of overstressing the soil nail bond, or causing damage to the corrosion protection system.
Number of load cycles	A minimum of two cycles is recommended with the bond resistance in the first cycle not exceeding T_d.	A single cycle is normally satisfactory.
Number of load increments	The maximum increment size should be sufficient to define the shape of the load displacement graph and should not normally exceed 20% of the maximum cycle load.	The minimum number of load increments is 5.
Interpretation of results	A *sacrificial* test result is acceptable provided that at the maximum test load P_{test} the creep rate is less than 2 mm per log cycle of time, i.e. $(s_2 - s_1)/\log(t_2/t_1) < 2$ mm where s_1 and s_2 are the measured nail displacements at time t_1 and time t_2, respectively. *The measured extension at the head of the nail is not less than the theoretical extension of any debonded length of the test nail* L_{db}.	A *production* test result is acceptable provided that: at the maximum proof load P_{pr} the creep rate is less than 2 mm per log cycle of time, i.e. $(s_2 - s_1)/\log(t_2/t_1) < 2$ mm where s_1 and s_2 are the measured nail displacements at time t_1 and time t_2, respectively. *The measured extension at the head of the nail is not less than the theoretical extension of any debonded length of the test nail* L_{db}.

NOTE 1 Definition of symbols used in this table as per BS EN 14490:2010, not ***1.3.2*** *of this standard.*

NOTE 2 The value of P_{test} should not exceed 80% of the characteristic nail tendon strength R_{tk} for Sacrificial tests or the design tendon nail strength R_{td} for Production tests.

6.2.2 Test equipment

Testing equipment should include the following.

a) *Stressing jack*

The stressing jack should be capable of applying the required test loads and be of a total capacity that is significantly greater than the maximum test load.

b) *Load measurement*

Load measurement may be by either monitoring the hydraulic pressure in the jack or by the use of a load cell.

Load measurement devices should be calibrated to an accuracy of ±2% of the maximum test load.

c) *Reaction system*

The stressing jack should be supported by a suitably robust reaction frame. The reaction frame should hold the stressing jack in line with the axis of the soil nail being tested and be capable of safety distributing the reaction force into the face of the soil nail structure without exceeding the safe bearing capacity of the facing.

d) *Displacement measurement*

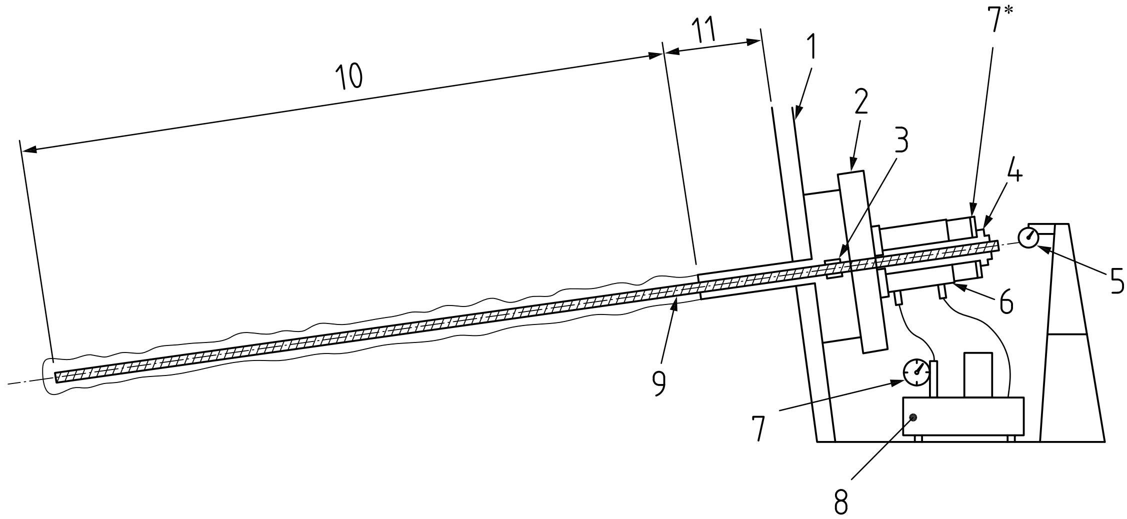
The displacement monitoring system should have an accuracy of ±0.1 mm.

The extension of the stressed part of the soil nail should be measured throughout the test without the need for reseating. The support of measurement gauges should be independent from the reaction system to enable absolute displacements to be measured.

The support system should be sufficiently robust so as not to be influenced by climatic effects or background vibrations.

A schematic soil nail testing system is shown in Figure 39 (from BS EN 14490:2010, Figure C.1).

Figure 39 **Schematic layout of the nail load test system**



Key

1	facing	7	calibrated pressure gauge to control jack force (load cell can be used as alternative, see 7* in the figure)
2	rigid reaction frame		
3	extension of soil nail for testing	8	pump
4	locking nut and plate at top of jack	9	soil nail
5	displacement gauge on independent support frame	10	bonded length
6	hydraulic jack for stressing	11	de-bonded length

6.2.3 Test procedures

It is recommended that only static maintained loading is adopted.

Static load testing involves incrementally loading the test nail and measuring the displacement of the nail at each increment. Long term creep characteristics may be investigated by maintaining a constant load over a period of time and measuring displacement. The number of loading cycles, maximum cycle load, minimum number of load increments and maximum test load may vary according to the purpose of the test.

6.2.4 Acceptance criteria

The purpose of most testing should be to either establish in the first instance the characteristic bond stress τ_k following design (sacrificial tests) or to demonstrate the satisfactory performance of the soil nail at an acceptance load (production tests).

The characteristic bond stress τ_k established from load tests is given by:

$$\tau_k = \frac{\tau_u}{\xi}$$

$$\tau_u = \frac{P_{ult}}{\pi d_{hole} L_b}$$

The value of ξ is based on the number of tests undertaken and values that may be used are given in Table 14 (from BS EN 14490:2010, Table C.1).

Table 14 **Values of correlation factor ξ**

Number of Load Tests	**1**	**2**	**>2**
ξ based on mean of test results	1.5	1.35	1.3
ξ based on lowest test result	1.5	1.25	1.1

6.2.5 Reporting

A test report including the following information should be produced:

a) installation records:

1) nail installation date;
2) location of test nail (reference layout drawing);
3) test nail position, length and inclination;
4) details of drilling method;
5) hole diameter, tendon type and size;
6) nail bonded and unbonded lengths;
7) nature of ground encountered;
8) groundwater encountered;
9) obstructions and delays;
10) grout take and loss;

b) testing records:

1) graphical plots showing applied load against;
2) displacement;
3) graphical plots showing applied load against time;
4) test data presented in tabular format;

c) design parameters:

1) maximum test load P_{test};
2) ultimate pull out force achieved in the test P_{ult}.

6.3 Materials testing

Materials testing should conform to the following standards, which are considered applicable to soil nailing:

a) grout – BS EN 196 (all parts);

b) sprayed concrete – BS EN 14487 (both parts);

c) concrete – BS EN 206-1;

d) reinforcing steel – BS EN 10080.

6.4 Other tests

Other tests such as face stability, head plate bearing capacity and non-destructive testing are out lined in BS EN 14490:2010 and should be specified as required by the designer.

6.5 Monitoring

Monitoring may be undertaken for a variety of reasons including:

a) confirmation that movements or forces do not exceed specified limits;

b) in conjunction with construction undertaking in accordance with the observational method (see Nicholson et al [35]);

c) to gather data for case history or research purposes.

Whilst often desirable, monitoring is by no means essential for the execution of successful soil nailing works.

Monitoring of soil nail structures may be undertaken:

1) during construction, especially where top down construction is envisaged;

2) post construction.

The type, extent and accuracy of monitoring should be clearly defined at the final design stage, in accordance with BS EN 1997-1:2004, **4.5**.

Monitoring should be designed to verify movements or forces adopted in the design, or to confirm pre-determined threshold parameters are not exceeded.

6.6 Monitoring during construction

Monitoring of the horizontal and vertical movement of the crest and facing may be performed at all stages of construction to confirm stability.

If movements approach or exceed threshold values then stabilization measures or a revised construction sequence should be implemented. The designer should review the design and the construction sequence in light of monitored behaviour of the structure making any necessary changes to the design.

6.7 Long-term or post-construction monitoring

Long-term and post-construction monitoring should be carried out following BS EN 14490:2010, **9.5**.

Section 7 : Maintenance

Soil-nailed structures should be designed to withstand the combined effect of all the deteriorating forces to which they might be reasonably expected to be exposed for a finite period – called the *design life*.

Regular inspection and maintenance are needed and should be implemented for such a design life to be achieved and to make sure that safety is not degraded.

The inspection and maintenance regime should be drawn up, depending on the risk category of the structure.

NOTE The CDM regulations require that a Health and Safety File is set up when a new structure is built. This has to cover maintenance requirements.

Facilities for maintenance (especially for safe and efficient access for maintenance) should be incorporated as part of the design.

Regular inspections/condition appraisal should check the following:

a) movement of facing and localized bulging;

b) movements in the soil-nailed ground and appearance of tension cracks at the crest;

c) condition of drainage (including moist areas on the facing and flow rates in the drains) and weep holes not being blocked or clogged;

d) condition of the nails and the facing and the amount of corrosion damage; the time intervals between such checks and their degree of rigour will depend on the aggressivity of the ground;

e) vegetation growth on the slope or wall; if trees have got too large they might blow over and cause damage – so they will need to be cut back or coppiced;

f) degradation of the structure caused by animal or human activity.

Records should be kept of the inspections and of any maintenance carried out.

The environmental value of slopes as habitats for animals and plants is increasingly being recognized; the times of year and procedures for vegetation maintenance should take this into account – to minimize disturbance to wildlife.

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