$$\left\{\frac{-\left[\sum\frac{T_{\mathrm{d1}}}{S_{\mathrm{h}}}\right]\left[\cos\left(\theta_1+\varepsilon\right)+\sin\left(\theta_1+\varepsilon\right)\tan\varphi'_{\mathrm{d1}}\right]+U_1\tan\varphi'_{\mathrm{d1}}-K_{\mathrm{d1}}}{\cos\theta_1+\sin\theta_1\tan\varphi'_{\mathrm{d1}}}\right\}$$

$$N_{12}=\left\{\frac{-\left(W_2\gamma_g+Q_{\mathrm{p2}}\gamma_{\mathrm{qp}}+Q_{\mathrm{v2}}\gamma_{\mathrm{qv}}\right)\left(\sin\theta_2-\cos\theta_2\tan\varphi'_{\mathrm{d2}}\right)}{\cos\theta_2+\sin\theta_2\tan\varphi'_{\mathrm{d2}}}\right\}+$$

$$\left\{\frac{\left[\sum\frac{T_{\mathrm{d2}}}{S_{\mathrm{h}}}\right]\left[\cos\left(\theta_2+\varepsilon\right)+\sin\left(\theta_2+\varepsilon\right)\tan\varphi'_{\mathrm{d2}}\right]+U_2\tan\varphi'_{\mathrm{d2}}-K_{\mathrm{d2}}}{\cos\theta_2+\sin\theta_2\tan\varphi'_{\mathrm{d2}}}\right\}$$

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 0.5φ' (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.

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 be 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*

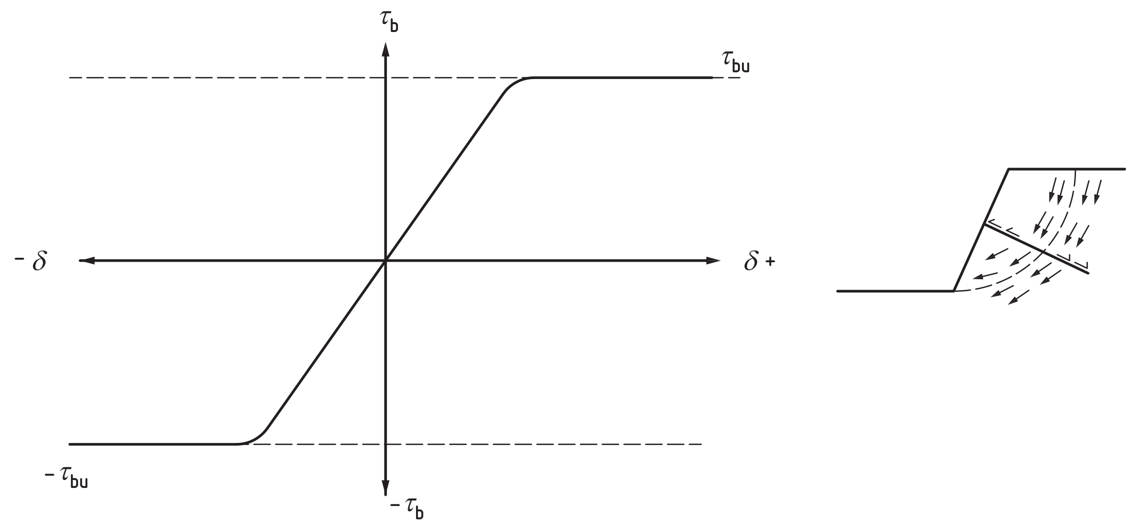
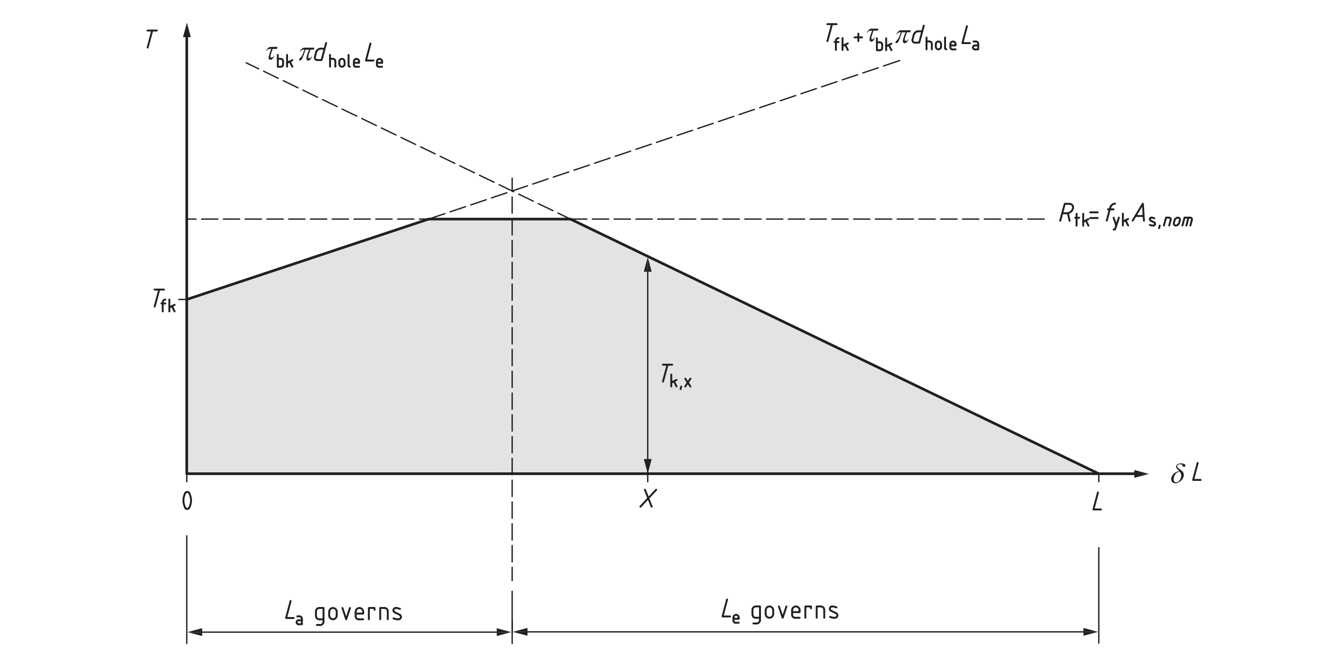


Figure 21 — *Limiting nail strength envelope*



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*

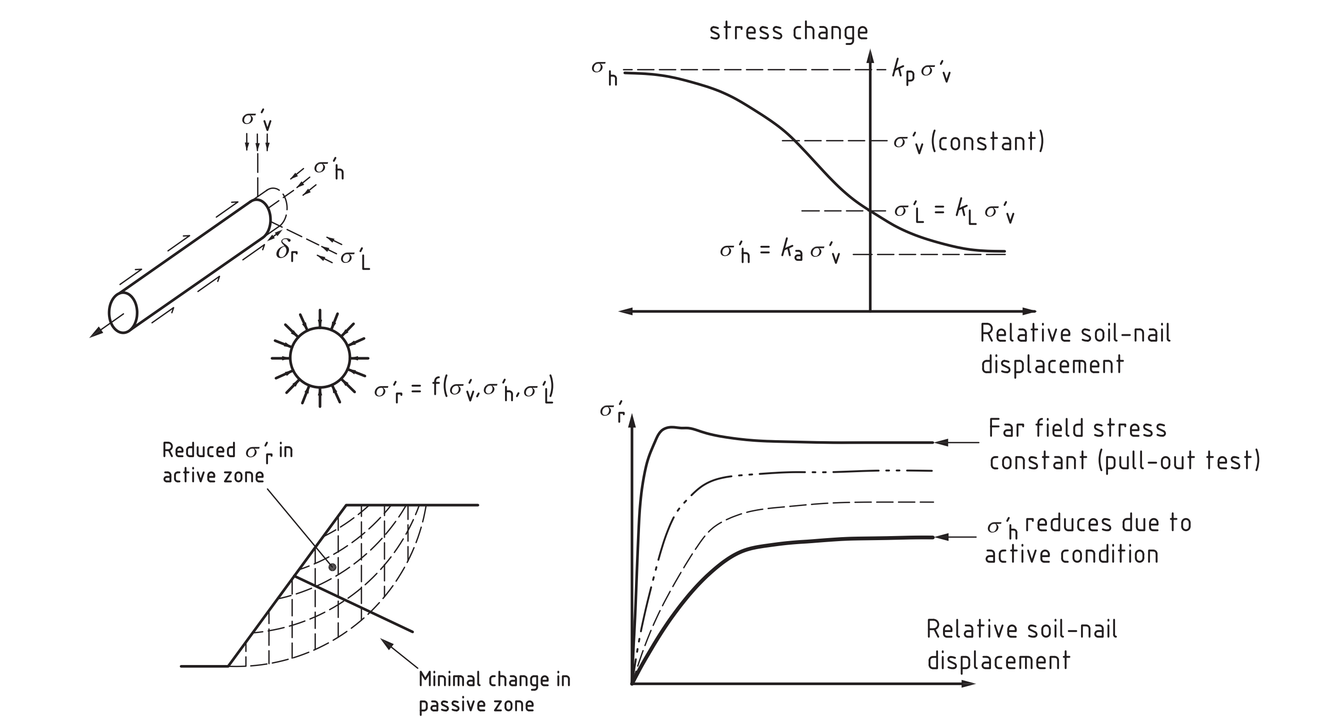


Figure 23 — *Modification of interface stresses due to far field stress changes*

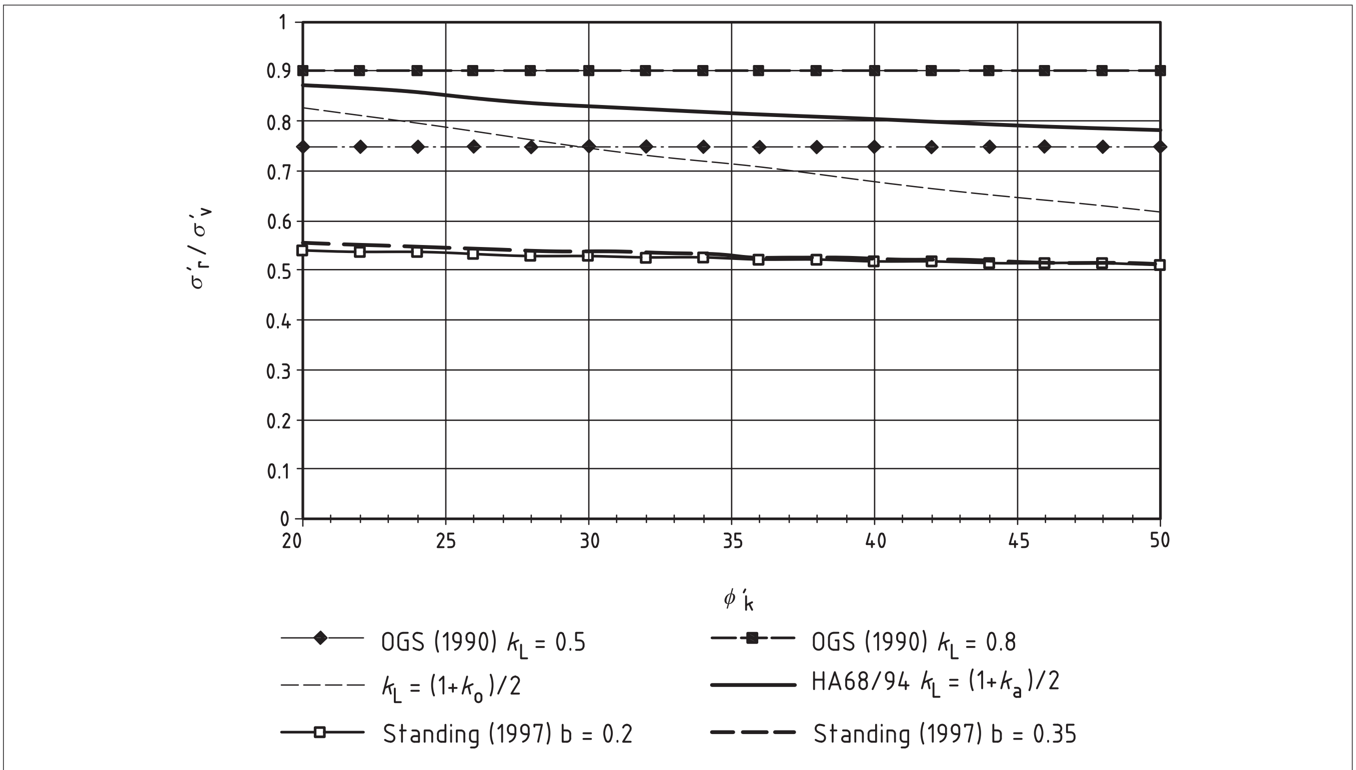
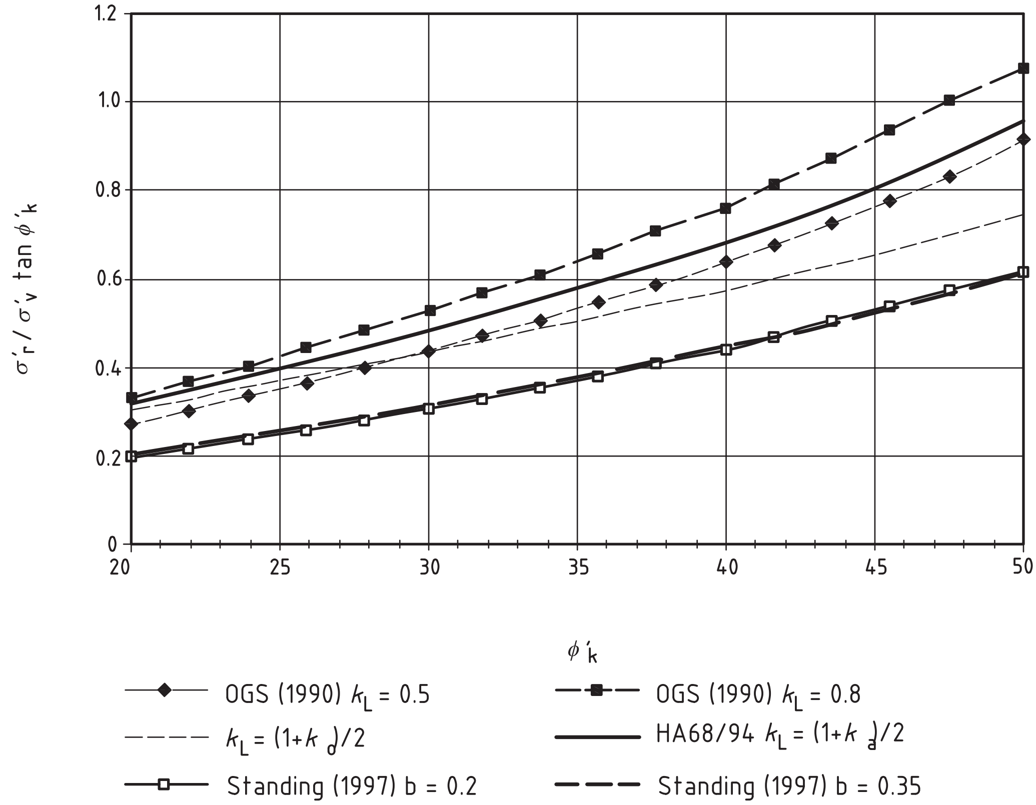


Figure 24 — *Relationships between radial friction normalized by vertical effective stress for a range of characteristic friction angle*



4.3.3 Effect of nail construction method

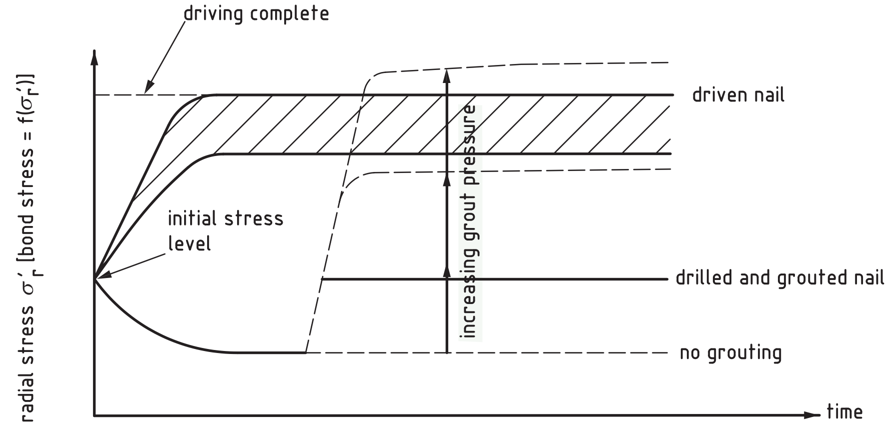
There are numerous methods of installing soil nails that should be considered, some of which will result in local stress increases and others in reductions in local stresses (see Figure 25). Generally methods that involve driving the nail into the ground may be expected to increase the stress locally 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.

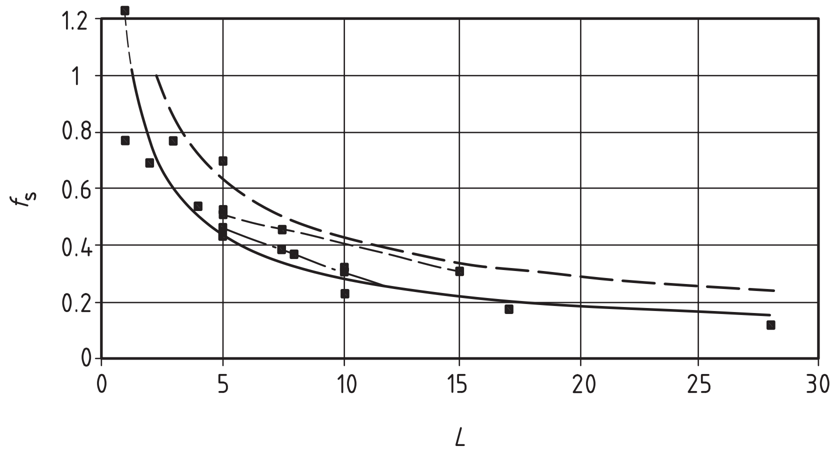
NOTE 1 This is an observation reported in BS 8081, after the work on ground anchors by Ostermeyer, and is considered to be due to different degrees of mobilization of bond stress along the test length. This effect is partly a result of non-uniform extension of the nail and is more prevalent in ground 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/m² 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:

$$\tau_{\mathrm{bu}} = \lambda_{\mathrm{f}} k_{\mathrm{r}} \sigma_{\mathrm{v}}' \tan \varphi_{\mathrm{k}}'$$

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.

$$\tau_{\mathrm{bu}} = \alpha c_{\mathrm{u}}$$

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 A1⟩ 1.5 and 1.3 should be applied to the average, or 1.5 and 1.1 to the lowest result, as per **6.2.4**, Table 14 ⟨A1.

*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.

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.