Soil nailing for slopes

Prepared for Civil Engineering, Highways Agency

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Executive Summary

The aim of this project is to encourage the use of soil nailing in the construction of new steepened slopes and the strengthening of existing earthworks where technical or economic benefits would result. Soil nailing is a relatively new technique and has considerable potential in both new construction and maintenance. The analysis and design of nailed slopes can be complex and although HA has produced an Advice Note and a British Standard has been published, a variety of approaches and assumptions are made by designers. The report identifies the important factors that need to be considered by clients and designers when soil nailing is proposed. Where published guidance is not available the authors have provided discussion and advice. Case histories and commentaries are provided on soil nailing schemes and it is hoped that these will be of value to geotechnical engineers when considering the use of soil nailing.

Soil nailing for slope stabilisation is a relatively new technique in the UK and no single, well-accepted design method is employed within the industry. In 1994 the Highways Agency published Advice Note HA 68 *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques* (DMRB 4.1). More recently BS8006:1995 *Code of practice for strengthened/reinforced soils and other fills* has been published. This document provides fairly comprehensive advice on reinforced earth structures but only limited guidance on the design and analysis of soil nails for slopes. It describes a number of design methods including the two-part wedge approach as given in HA 68. The document, however, recommends partial factors that are at variance with those adopted in HA 68. This can lead to problems with the design of soil nails.

Many designers favour a simple approach of analysing the unreinforced slope and calculating the total nail force required to improve stability. This may be acceptable in straightforward situations, but more rigorous analysis is generally needed. Such an analysis is provided by HA 68, but many designers find it difficult to use and consider the resulting designs to be conservative.

The design of soil nailing is critically dependent on the quality of the site investigation data available. Selecting design soil strengths and porewater pressures is difficult, as is the prediction of corrosivity. The technique is unlikely to be suitable in soft soils, or where obstructions such as cobbles are present. With the present level of experience of soil nailing, it is recommended that nails are not used in situations where large cyclic or dynamic loads might apply.

Some deformation of a soil nailed slope is required to mobilise tension in the nails (above any small tensions developed during construction) and to reach a state of equilibrium. Soil nailed slopes are not appropriate, therefore, in situations where some movement of the slope cannot be tolerated during the service life of the earthwork. The nail installation angle has a significant and complex effect on the performance of a nailed slope. For ease of grouting and speedy generation of tension with soil movement it is suggested that a value of between 10^o and 20^o to the horizontal be chosen.

Trial pull-out tests are commonly carried out as part of the nailing works and give important insight into potential nail performance, although interpretation of the results is not straightforward. Pull-out test results on the schemes studied in this report were best predicted using calculations based on the undrained shear strength and gave a mean value of the ratio P_{mes}/P_{calc} of about 1.9, using an adhesion factor of 0.45. While this suggests that the short-term test pull-out resistance may be better estimated using the undrained strength, drained parameters may be more appropriate for estimating long-term behaviour.

However, the relations derived between measured and calculated pull-out resistance may be used by the designer to check and adjust the design. Where early tests show pull-out results consistently and significantly higher than unfactored design values, the designer might consider increasing the design values. However, where they are lower the cause must be investigated.

Although the use of soil nailing has not been as widespread as anticipated, it has proved to be a good technique for the construction of new steep cuts or the strengthening of existing marginally stable slopes. Advantages include ease of construction, economic and environmental benefits. It is hoped that this report will provide additional guidance that will allow the technique to be applied more widely in the future.

1 Introduction

1.1 Background

Soil nailing is a useful, economic technique for the construction of new steep cuts or the strengthening of existing slopes. While the basic concept of reinforcing soil with tensile elements is reasonably straightforward, the exact mechanism by which nails strengthen and stabilise a slope cannot be modelled easily. A number of assumptions and simplifications must be made to define required properties of the nails and their spacing. The details of the installation of the nails can have a significant effect on their performance and design requires a good deal of information and experienced engineering judgement.

The use of soil nailing has increased rapidly in Europe and North America since the early 1970s. The first recorded application of soil nailing in Europe was in 1972 for an 18 m high 70° cutting as part of a railway widening project near Versailles, France. Schlosser *et al.* (1992) indicate that by the late 1980's some 80,000 m^2 per year of soil nailed slopes and structures was being constructed in France alone. Soil nailing has been used in the United States since the mid-1970s particularly for temporary excavation support (FHWA, 1996). Despite this trend the use and development of the technique in the UK has been relatively limited.

At the present time there is limited guidance available for evaluating the potential for using soil nails or for selecting the appropriate method of analysis. The Advice Note HA 68 *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques* (DMRB 4.1), published in 1994, gives guidance for the design of reinforcement, including soil nails, for strengthening highway earthworks. In addition, BS 8006:1995 *Code of practice for strengthened/reinforced soils and other fills* gives comprehensive advice on reinforced earth structures but only limited advice on the design and construction of soil nailed slopes. The document, however, recommends partial factors that are different from those adopted in HA 68 and this can lead to problems when designers try to combine parts of the two documents.

The situation is further complicated by the fact that the Highways Agency have implemented BS 8006:1995 through the publication of BD 70 *Strengthened/ reinforced soils and other fills for retaining walls and bridge abutments: Use of BS 8006:19*95 (DMRB 2.1). However, BD 70 does not cover the design of earthwork slopes and provides only general guidance on soil nailing for retaining walls.

Other documents which are sometimes referenced in design submissions are the French Clouterre Recommendations (1991) and the American FHWA Design Manual (1996). A CEN Execution Standard for Soil Nailing is currently being drafted.

Where soil nailing has been employed, UK design engineers have adapted some of the above documents and standards and others on soil reinforcement and ground anchorages. These documents might not be entirely appropriate to the design of soil nails for slope stabilisation, particularly for highway schemes where compliance with Departmental Standards and Advice Notes is required.

The lack of a proven and accepted design method may be discouraging more widespread use of soil nailing techniques. Also, different approaches to design, incorporating different assumptions, have been used by different design authorities. On larger schemes, where designers tend to be, or have access to, experienced geotechnical engineers then well founded assumptions are likely to be employed. But for smaller jobs, or where soil nailing is brought in as an alternative option within a contract, insufficient time or expertise may be available for a rigorous design to be developed: this may lead to a final design being either unconservative or overly conservative. Because of the uncertainties associated with the installation of reinforcement in natural ground, designs have tended towards being safe rather than optimising economy. This is likely to change, albeit slowly, as more experience and a better understanding of the technique are developed.

1.2 Objectives and scope

The objective of this report is to encourage the use of soil nailing for the construction of new slopes and the strengthening of existing ones, where technical or economic benefits would result. A new certification system, the *strengthened earthworks appraisal system* is being introduced on HA schemes. This report forms a companion volume to Johnson and Card (1998), which covers soil nailing for retaining walls.

Section 2 of this report compares soil nailing with other techniques for strengthening or stabilising slopes. Section 3 discusses the principles of soil nailing, while Section 4 covers design considerations and includes a summary of UK design documents. Section 5 provides a discussion on the interpretation of pull-out test results. Section 6 lists the case studies while Section 7 summarises the important points which emerged during the study. Appendices A to H describe strengthening schemes using soil nails and provide details of the design philosophy, selection of design parameters, method of analysis and results of pull-out tests. As these Appendices provide a critique of the schemes described, these have been identified by a reference number only. Appendix I contains summaries of other schemes to further illustrate the wide potential of soil nailing. Appendix J provides a list of all the organisations who provided information to TRL on the schemes described in this report.

1.3 Methodology

The report is based on a review of existing schemes where sloping ground has been strengthened or stabilised using soil nails. This included the study of design calculations and methods, check calculations and site pull-out tests. Comments and opinion were sought from designers regarding their design philosophy for soil nails and on practical aspects of design, construction and durability. As part of the project, British Standards and other documents were examined to identify those parts most useful to a designer of a nailing scheme.

As part of the project TRL commissioned the University of Wales, Cardiff to undertake a series of centrifuge tests to investigate the behaviour of soil nails installed in a slope and allow prediction of their long-term performance. The results of these tests have been reported (Jones, 1999) and the understanding gained from these tests incorporated in this report.

2 Slope strengthening techniques

2.1 General

Reinforcement of slopes is undertaken for two main reasons, i.e:

- the construction of new embankments and cutting slopes;
- the stabilisation of existing cuttings or embankments.

There are a number of techniques available for increasing the stability of a soil slope by the inclusion of reinforcements. Greater strength could be imparted through tension reinforcement, shear enhancement or a combination of the two. For new embankment slopes, the reinforcing elements are built into the structure as it is constructed from the bottom up. For new cuttings construction will be from the top downwards, but for existing slopes the reinforcements will be installed into existing material and the building sequence may be bottom up or top down.

Reinforcements installed in fill will normally be laid horizontally, and the surrounding fill compacted around them. Nails and other reinforcements installed in natural soil will normally be inclined to the horizontal, and may be grouted into pre-drilled holes or installed by a displacement method such as firing or percussion. The angle of inclination at which the reinforcement is installed is an important aspect of the design on which little published advice is available: some comment is provided in Section 3.5. Typically, nails are relatively long and thin and installed approximately horizontally as shown in Figure 1a. Should the active wedge of soil start to move, tension will quickly build up in the nails to resist further movement.

Alternatively the reinforcements may be shorter and thicker and installed approximately normal to the potential failure plane, as shown in Figure 1b. In this case movement of the soil wedge would tend to induce bending and shear in the reinforcements which basically act as dowels.

For a soil nail to develop a significant restoring force due to bending and shear resistance, in general a substantial soil displacement will be required (Jewell and Pedley, 1990) particularly in soft soils which would tend to flow around the reinforcement. Thus where the reinforcement is intended to work in axial tension it should be installed at an angle such that a small movement of the soil will quickly generate tension in the nail.

Whichever reinforcing technique is chosen, it is important to consider the porewater pressures which could develop during the service life of the earthwork. A reinforced earth embankment constructed from the bottom up using free-draining fill should enable the designer to control the build-up of porewater pressures. And detailing of drainage

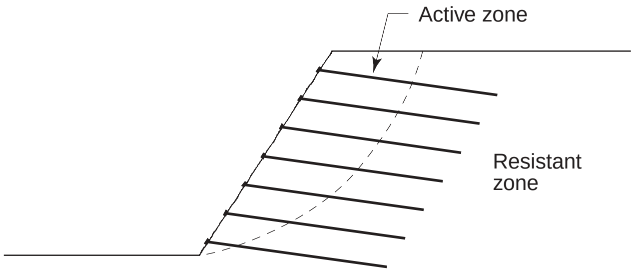

Figure 1a Soil nails

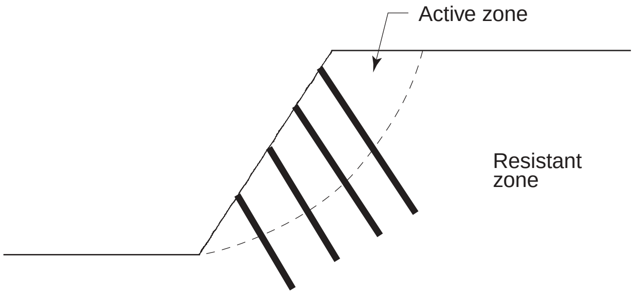

Figure 1b Soil dowels

measures should prevent excessive porewater pressures being generated during the service life of the slope.

However, it is generally more difficult to predict long-term pore pressures in natural soils. Furthermore, surface water infiltration on the slope face can result in high pore pressures at shallow depth resulting in potential shallow instability (Crabb, 1994; Fourie, 1996). The designer will need to consider whether the present or future porewater pressures generated in the soil are such that soil nailing is inappropriate. Drainage measures may often be required for nailed slopes or cuttings because of the importance of preventing the build-up of positive porewater pressures. The research undertaken at the University of Wales, Cardiff has identified the loading mechanisms of soil nails, due to staged construction and pore pressures generated from either groundwater movement or surface water infiltration (Jones, 1999).

Different reinforcing techniques improve stability in different ways and it is important that the designer considers the correct mechanisms and behaviour for the chosen technique. A major consideration is the slope angle and the consequential need for structural facing or erosion protection. For shallow slopes, typically less than 30^{o}, it is unlikely that any facing or cover system is required to prevent localised failure or to control erosion. For steeper slopes, however, some form of structural facing is likely to be needed together with erosion protection. For some techniques, such as soil nails, a facing element will generally be required to allow tension to develop in the nail.

A brief summary of some of the systems is given below; the first three are *in situ* techniques for natural ground whilst the fourth is for construction using imported fill.

2.2 Soil nails

Soil nails involve the insertion, either by boring or driving, of tensile elements into otherwise undisturbed soil or fill. To provide a reinforcing effect the nails must cross the potential slip planes within the soil mass. When inserted into bored holes, nails should be grouted to ensure intimate contact with the soil. They are installed typically at a declination of 10° to 20° to the horizontal primarily to aid the grouting process. They are essentially passive elements and do not normally generate any restoring force until there are movements within the soil mass, but some pre-load may be generated during the construction process.

As opposed to complete stabilisation, soil nails have also been used to control the rate and magnitude of movement. In such cases nailing is undertaken in conjunction with monitoring of slope movement to determine the optimum arrangement of nails. The technique has been used to minimise differential movement beneath a 500 m length of highway that traverses an existing landslide in the Severn Gorge at Ironbridge, Shropshire (Anon, 1996). The soil nails were designed to minimise differential soil movement beneath the carriageway. This was achieved by installing nails below the level of the carriageway and reconstructing the sub-base and pavement (reinforced with high strength geotextiles).

Case studies of slope stabilisation or strengthening using soil nails are given by Whyley (1996), Barley (1993) and Pedley and Pugh (1995). Ortigao *et al.* (1995) provide data on some 20 soil nailing schemes undertaken in Brazil.

2.3 Ground anchorages

Ground anchorages provide a stabilising force from a grouted length of tendon behind the potential failure plane: this is transferred along a debonded length of shaft to a surface bearing plate (BS 8081:1989). The bonded length of the anchorage must lie behind the potential failure plane to generate the required stabilising force. Ground anchorages are active devices and the unbonded length is prestressed against a surface bearing plate. Thus stabilising forces are generated without the need for any soil movement within the slope. Ground anchorages are sometimes installed at approximately right angles to the worst potential failure plane and in this case their effect is mainly one of increasing the frictional resistance along the plane by increasing the normal force. On other occasions anchorages are installed such that their tensile capacity directly opposes the likely movement of the ground. Local factors such as stratigraphy and site boundaries also influence anchor location and orientation.

An important consideration is the design of the facing plate which should be of sufficient size to ensure that local bearing capacity failure does not occur. Typically ground anchorages are used in conjunction with a vertical wall to stabilise cuttings in soil and rock. Littlejohn (1990) has reviewed the design and construction of ground anchorages.

2.4 Soil dowels

Soil dowels are usually large diameter concrete piles installed approximately at right angles to the potential failure plane to provide enhanced shearing resistance. They are generally used to reduce or halt downslope movements on well defined shear surfaces. Gudehus and Schwarz (1985) have shown that the most efficient way to mechanically increase the shearing resistance on a weakened shear surface through a soil is to use relatively large diameter piles which combine a large surface area with high bending stiffness. Thus the diameter of a soil dowel is generally much greater than that of a soil nail.

There is, however, no accepted standard design method for calculating the stabilising force which dowels generate. Commonly design methods consider the enhanced shear strength on the failure surface due to the inclusion of the dowels. Relatively simple methods of analysis, however, are available for calculating the stabilising force generated for slopes and landslides reinforced with dowels (De Beer and Wallays, 1970; Ito *et al.,* 1981; Hassiotis *et al.,* 1997). The use of vertical piles to stabilise slopes has recently been reviewed by Carder and Temporal (2000).

2.5 Reinforced and anchored soil

Commonly, reinforced soil (and occasionally anchored soil) are used to form steepened embankments and retaining walls. The techniques associated with reinforced and anchored soil are applicable mainly to new construction since the tensile elements are incorporated into the structure using layers of selected fill. Thus much better control of the fill properties and drainage conditions is possible compared to the other techniques described above. With Reinforced Earth, where commonly metal strips are attached to the rear of the facing, no systematic pre-tensioning of the reinforcement is possible. However, a load is induced in the strips during the construction process through the placing and compacting of the fill.

Similarly with anchored earth, where typically the threaded end of the anchor passes through the facing, some pre-load is induced during construction and there is also an opportunity to tighten the facing nut, but it is difficult to predict the effect that different tightening torques would have on the long-term performance of the structure. Any in-service movement of the soil will tend to increase the tensions in the reinforcing strips or anchors.

3 Principles of soil nailing

3.1 Nail behaviour

A soil slope can be formed in one of two ways:

- by natural geological and geomorphological changes, which often take place over a considerable time period;
- by man made excavation over a relatively short period of time.

In a natural slope geological processes may cause the soil mass to become unstable and begin to move. In a man made excavation the depth and inclination of the slope may be such that the soil cannot support its own mass. In such cases, modifications to the ground are needed to maintain stability. The fundamental mechanism of a soil nailed slope is the development of tensile force in the resistant zone. The nail elements interact with the ground to support the stresses and strains that would otherwise cause the unreinforced ground to fail.

The stability of a slope can be reduced by excavation (reduction of lateral stress), or through a reduction in soil strength (reduction in effective stress, through for example, inundation). Since the soil mass in a slope is restrained on three sides, the soil particles can only move down and outwards as resistance is reduced. The form of movement is dependent on the nature of the soil. A granular material will tend to move as a translational block parallel to the slope surface. A more cohesive material may move as blocks form along zones of weakness within the mass. This movement of the soil induces load into the embedded reinforcements. The level of load imposed on the nail and the mechanisms set up to redistribute that load are dependent on the response of the soil nail system. The initial movement is small and minimised by the axial stiffness of the nail. A nail is relatively stiff compared to the soil structure and hence its bending stiffness may also be mobilised. Moderate soil movements, however, result in a small contribution from bending stiffness. Thus for the most part, the load induced by the unstable mass is transferred along the nail in axial tension and redistributed to the stable mass.

With increasing ground movements, the tensile load in a nail increases. If the movements are very large then, depending on nail properties and geometry, the bending resistance of the nail may provide some small resistance in addition to the tensile component. Depending on the soil, geometry of the slope and nail inclination, this may induce a cantilever effect at the front of the nail, or possibly a deformed S shape within the zone of high strain (i.e. a failure plane or zone). This complex loading condition occurs at large displacements, usually well beyond serviceability limits for the slope. As failure approaches the distortion of the nail increases. Ultimate failure occurs when the nail itself ruptures, the soil moves around the nail, or the nail is 'pulled out'.

An existing slope showing signs of instability (i.e. tension cracks at the crest, small movements and the start of bulging at the toe) can be strengthened by soil nails. In the short-term the nails will not be loaded until ground movements occur. It may take a number of months or even years before any significant load is induced in the nails. In contrast, nails used to maintain the stability of a man made steepened slope will tend to achieve a significant proportion of their loading during construction. For example a steep excavation, typically greater than 45°, would remain stable for a short period of time depending on the soil and groundwater regime. As excavation proceeds, however, the distribution of loading on the slope is radically modified and the nails are rapidly loaded due to the significant reduction of stability in a short time. After construction, further loading similar to that described for the strengthening of an existing slope can occur.

3.2 Nail resistance

It is generally accepted that the axial resistance of the nail inclusion is the major component in maintaining stability of a soil nailed slope (Pedley, 1990; Davies, 1996; Bridle and Davies, 1997). The contribution from bending stiffness is small unless the nails are oriented approximately normal to the failure plane; their stiffness is similar to that of the surrounding ground; a narrow, well-defined shear band forms; and significant soil movement occurs. A number of published documents describe methods of analysis of soil nailed systems. These include:

- BS 8006:1995 *Code of practice for strengthened/ reinforced soils and other fills.*
- HA 68 *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques* (DMRB 4.1).
- French National Research Project Clouterre (1991). *Soil nailing recommendations.*
- Federal Highway Administration (1996). *Manual for design and construction monitoring of soil nail walls.*

As shown below, the general consensus amongst these documents is that bending resistance does not contribute significantly to the strength of a nailed slope.

In considering the contribution of the shear and bending resistance of the nail, Section 2.4 of HA 68 states that the contribution of soil nails is assumed to be purely axial. This is echoed in Section 2 of Clouterre (1991) which concludes that the most important interaction is the shear stress (skin friction) applied by the soil along the nail length, which induces tension in the nail. A second, less important interaction is the passive pressure of the earth along the nail during the displacement of the soil. The passive earth pressure mobilised makes it possible for bending moment and shear force to be mobilised in the nails; this mobilisation occurs only if a shear zone develops in the nailed mass. Both Clouterre (1991) and FHWA (1996) consider that the contribution of shear and bending resistance is relatively small and only develops near failure. Pedley (1990) reports that the beneficial effects arising from bending stiffness are a post serviceability phenomenon and therefore should not be relied upon in design. This is echoed in FHWA (1996), which states that the contribution to stability by bending resistance of the nail is typically an order of magnitude less than the contribution by axial resistance. Also the bending contribution is not achieved until displacements have taken place that are at least an order of magnitude greater than those required to generate maximum axial resistance. Thus, the only contribution from bending resistance is at ultimate limit state, where the nails may provide shear or bending resistance at large displacements.

These reinforcing mechanisms have been confirmed by the centrifuge tests undertaken by the University of Wales, Cardiff where both cohesive and non-cohesive soil slopes of 70° were stabilised using soil nails (Jones, 1999). Under serviceability limit state conditions, these instrumented nails maintained stability almost solely by axial resistance. Bending resistance was found to be negligible.

The displacements required to generate bending resistance were also investigated by TRL through lateral load trials, see Appendix A. Whilst the actual distribution of load in the nail was uncertain, significant lateral displacement of the nails was required to develop any noticeable lateral resistance. Experiments undertaken in France (Clouterre, 1991) and in Germany (Gassler, 1988) as well as shear box tests undertaken at University of Wales, Cardiff also demonstrate that the only significant loading mechanism that occurs at small displacements is tensile resistance (Bridle and Davies, 1997).

For ultimate limit state conditions, failure of a nail may involve a complex mechanism of axial, shear and bending resistance that is difficult to solve. Therefore, it is common practice to adopt reasonably conservative simplifications. One common assumption is that shear or bending resistance of the nail make no significant contribution to the stability of the slope at the working condition, or at the ultimate limit state.

3.3 Internal stability

The most widely accepted method of quantifying the stability of a nailed slope involves the use of a limit equilibrium method of analysis to determine the distribution of forces. It is common practice to assess stability using a slip surface analysis: this involves defining a plane that divides the slope into active and passive zones. An equilibrium analysis can be performed by resolving forces, taking moments, or both. A number of simplifying assumptions may be involved to determine the factor of safety against shearing of the soil along a potential failure plane. The process is repeated for a number of planes until the lowest factor of safety is identified, the critical slip plane.

A number of forms have been used to model a slip surface. The most common are shown in Figure 2 and described as follows:

- *Translational:* Essentially, a slip surface running parallel to the slope surface.
- *Single wedge:* A straight line extending from the toe and exiting some point behind the crest.
- *Two-part wedge:* A bi-linear slip plane comprising a line extending from the toe, and a steeper line extending to the top of the slope. Changing the angle of inclination and the length of the two lines varies the position of the slip surface.
- *Circular:* Circles of varying radii and position: the circles may or may not pass through the toe of the slope.
- *Log-spiral:* A log-spiral running from the toe of the slope.
- *Parabolic:* A parabola running from the toe of the slope.
- *Others:* Slip lines of no fixed shape, found by iteration.

Because of the simplifying assumptions made to facilitate computation, such as dividing the soil into blocks for equilibrium analysis, none of the above necessarily model *in situ* behaviour exactly. Selection of the method depends largely on the tools available to undertake the calculations. It is important to note that the slope angle influences the trajectory of the critical failure plane.

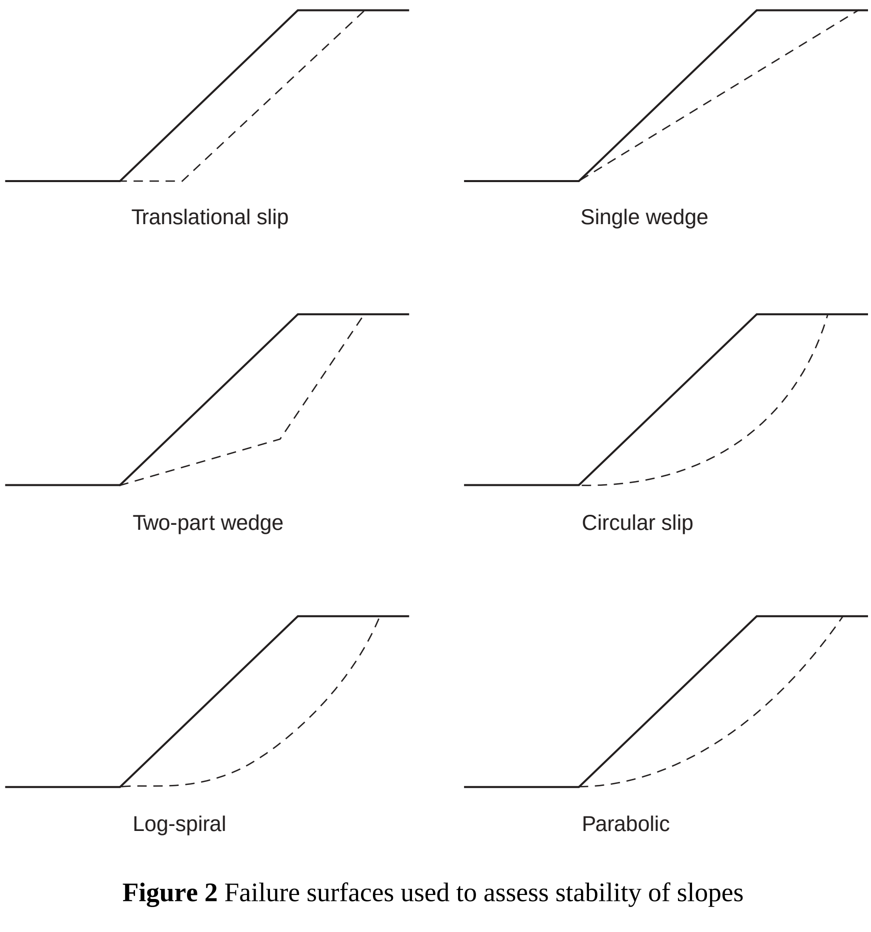

Figure 2 Failure surfaces used to assess stability of slopes

Comparisons of the above have been made by many researchers, see for example Love (1993) and Ortigao *et al.* (1995). Love (1993) undertook comparative studies using single wedge, two-part wedge and log-spiral mechanisms for reinforced slopes with slope angles in the range 60° to 90°. He found that, in general, the two-part wedge was the most conservative whilst the single wedge was the least conservative. It is not clear, however, whether the single wedge would give an unsafe design. Published design methods use a variety of potential slip surfaces and some of these are described below.

Clouterre (1991) recommends the use of standard methods, such as Bishop's method of slices (circular) or the perturbation method (non-circular), because these have been in widespread use for over 30 years. It advises against planar potential failure surfaces, particularly for cohesive soils. It notes that the critical failure plane for a reinforced slope may be different from that of the unreinforced slope.

The FHWA (1996) design manual is less specific and suggests that all potential slip surfaces must be examined to ensure that design is complete. Slip surfaces other than simple planes (e.g. circles, log-spirals, bilinear wedges, etc.) are preferred because (i) they generally provide lower calculated factors of safety and (ii) planar slip surfaces can be closely approximated by these more general shapes.

BS 8006:1995 provides similar advice to FHWA (1996) in that it suggests that the method of analysis selected should ensure that the most critical failure surface is determined. It recommends the two-part wedge and the log-spiral method. Where soil nailing is being used as a remedial measure in an unstable slope, it states that attention should be paid to existing failure surfaces. BS 8006:1995 also states that in some cases, particularly for steep slopes and near vertical walls, the most critical failure condition corresponds to a single planar surface, i.e. a single wedge.

HA 68 recommends the use of a two-part wedge mechanism: this is preferred because it provides a relatively simple method for obtaining a safe solution for steep slope angles between 50° and 70° and is particularly suitable to reinforced soil and soil nailing geometry.

The methods that are generally favoured appear to be those which can either be undertaken easily by hand, or by proprietary computer program. It also appears that the two-part wedge, log-spiral and circular slips are widely accepted for modelling potential failure surfaces, but the single wedge much less so.

For most methods of analysis, the basic approach is to identify the failure plane which generates the largest out-of-balance force (T_{max}). For rotational slip planes the out of balance moment (M_{max}) is more appropriate. A trial nail array is then assumed and is checked to ensure it can develop sufficient restoring force or moment to maintain stability with an adequate factor of safety.

There are innumerable possible failure planes, but analysis should identify that which requires the greatest restoring force (T_{max} or M_{max}) to maintain stability. It is then assumed that the resistant zone and failure wedge behave as rigid bodies but, as mentioned above, movement may occur within a 'failure zone' rather than along a discrete failure plane. Because the rear edge of such a failure zone may be located further back than the 'design' failure plane, the effective length of the nail (L_e) resisting pull-out may be rather less than that assumed in design. Where calculations indicate that the initial nail layout generates substantially more restoring force than required, consideration should be given to increasing the spacing or decreasing the diameter of the nails rather than decreasing their lengths. Assessing the relative economy of a layout is complicated where nails of a constant length are used rather than nails of different lengths.

For a marginally stable or failed slope where a pre-existing failure zone exists, an alternative (and possibly more logical) design approach is to determine the out of balance force or moment of the existing un-nailed slope. The required additional restoring force or moment to provide an adequate factor of safety is then determined. The required force or moment is provided by a nail array. This approach is generally consistent with that adopted in BS 8081:1989 for ground anchorages.

The above discussion relates to a slope composed of a reasonably uniform soil. Variations in the soil strata or the presence of relic slip surfaces have a major effect on the location and shape of the probable failure surface.

Because there is no single, universally accepted design method, in certain circumstances it may be appropriate to use two or more independent methods to determine the nail layout.

3.4 External and overall stability

For all slopes, checks must be made for gross sliding and deep seated failure. On occasions, some nails may need to be lengthened to ensure that one or other of these external failure modes does not occur.

BS 8006:1995 states that a stability check should be made on the block of reinforced soil (as if it were a gravity wall). For new construction, in particular steepened embankment and cutting slopes, where the nails are installed on a fairly close spacing, this seems a reasonable approach. However on marginally stable steep slopes, which have been assessed and found to require only a few widely spaced nails, the concept of the whole soil block acting as a monolith is less appropriate.

Conventional methods of slope stability analysis are considered appropriate for assessing overall stability. However judgement is required in the selection of soil strength parameters and suitable factors of safety: these vary according to the approach adopted.

3.5 Nail orientation

The addition of soil nails to an existing slope requires a decision on the orientation of the nail: for economy the most effective and practical angle of installation should be used.

Clouterre (1991) suggests that nails should be placed as horizontally as possible to limit deformations in the upper part of the wall. But for ease of installation, the nails are slightly inclined downward from the horizontal. While the inclination can depend on the technology available and the working conditions under which the nails have to function, in practice angles of 10° to 20° are common. If there is a need to steepen the nail inclination (e.g. to avoid shallow utilities), the local stability in the area of these more steeply inclined nails must be carefully considered because reinforcement efficiency decreases significantly with increased inclination.

HA 68 and the FHWA (1996) design manual give similar advice to Clouterre (1991). Thus it would appear from the literature that nails should be installed as close to the horizontal as practical to be most effective. This can be tested using the two-part wedge analysis given in HA 68. The two resisting components that vary with inclination are the direct tensile resistance and the improved shear resistance on the failure plane provided by the tensile resistance. This combination is a function of nail inclination through the factor:

$$\xi = \cos(\theta_1 - \phi') / \cos(\theta_1 - \phi' + \delta)$$

where θ_1 is the inclination of failure plane from the horizontal

ϕ' is the angle of friction of the soil

δ is the nail inclination downward from the horizontal.

Inclination can be plotted against ξ to determine its optimum value. Take a nail intersecting a failure plane, inclined at 60º to the horizontal. For a ϕ' value of 25º, the resulting plot of ξ versus δ is shown in Figure 3a. From this the most efficient nail inclination is –35º (or 35º above the horizontal). This can then be plotted as a function of nail effectiveness, see Figure 3b.

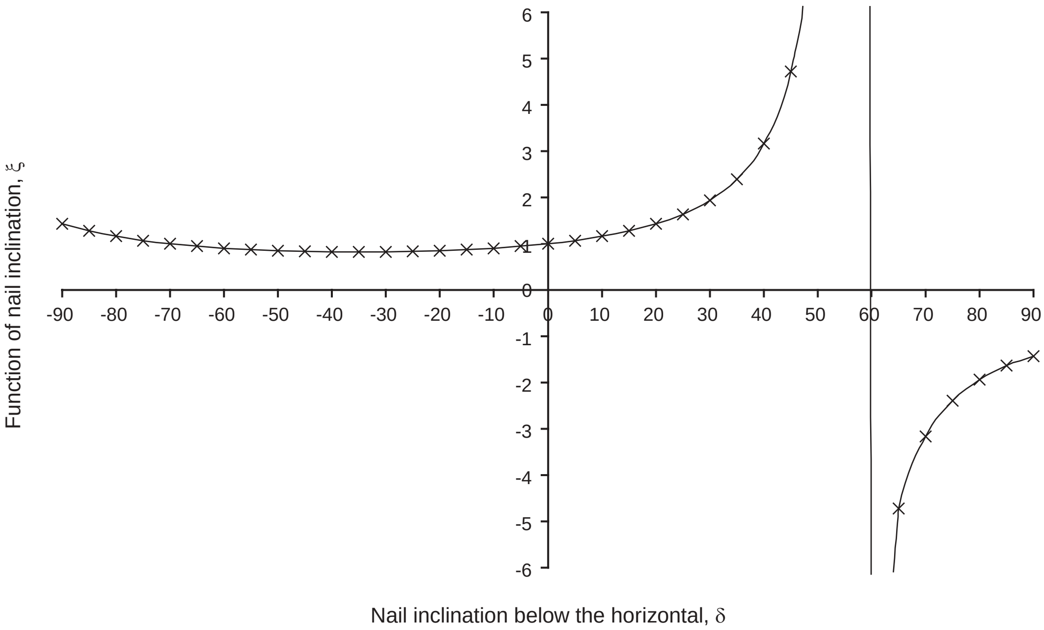

Figure 3a Variation of a function of tensile resistance of the nail with inclination

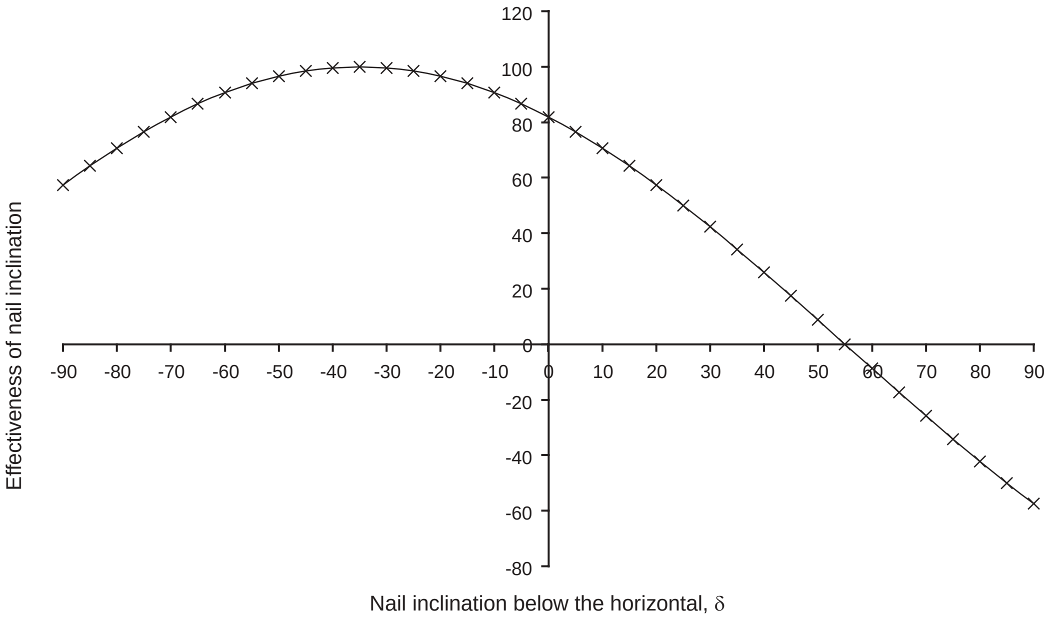

Figure 3b Effectiveness of nail inclination

This plot also shows that effectiveness is reduced to 82% for horizontal installation, and to 71% at an inclination of 10º below the horizontal. Effectiveness continues to decrease until a nail inclination of 55º is reached at which point the nail has no effect. These situations are shown in Figure 4. On this basis it would appear sensible to install nails inclined at an angle above the horizontal to maximise tensile resistance. However when other components of the nail design and the practicalities of construction are taken into account, an inclination just below the horizontal is generally the most favourable.

Figure 5 shows the above analysis applied to a 6 m high slope reinforced with a single 6 m long nail. The nail installed at the optimum angle has a short (2.3 m) length in the resistant zone and a little (1.2 m) depth of overburden. Nails installed horizontally or inclined downwards have a length in the resistant zone of between 4.2 m and 4.4 m, and an average overburden of 4 m to 5.9 m. Although the nail installed at 15º below the horizontal has an efficiency of 64% of the nail installed at the optimum angle, it has twice the length in the failure zone, and four times the overburden. Based on the length in the resistant zone alone, the nail inclined slightly downwards is actually more effective. If pull-out is considered to be a function of overburden it is four times more effective. However as the nail angle gets steeper than 15º the efficiency rapidly decreases without any increase in pull-out length or significant increase in overburden.

For practical reasons, drilled and grouted nails need to be inclined below the horizontal to facilitate grout flow, and fired nails tend to be installed perpendicular to the slope surface. Based on theoretical analysis and the practicalities of installation, it would appear the optimum design angle would be between about 10º and 15º below the horizontal.

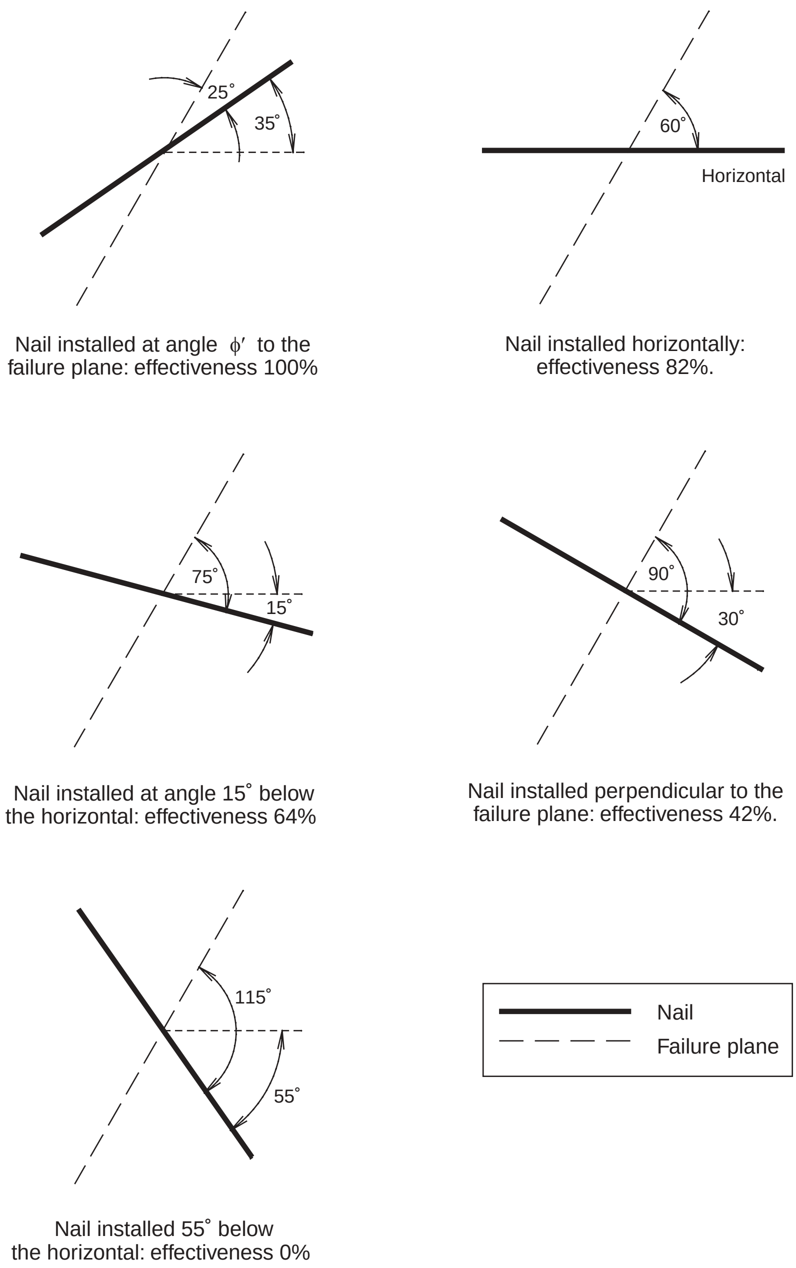

Figure 4 Theoretical tensile efficiency of nails placed at various inclinations

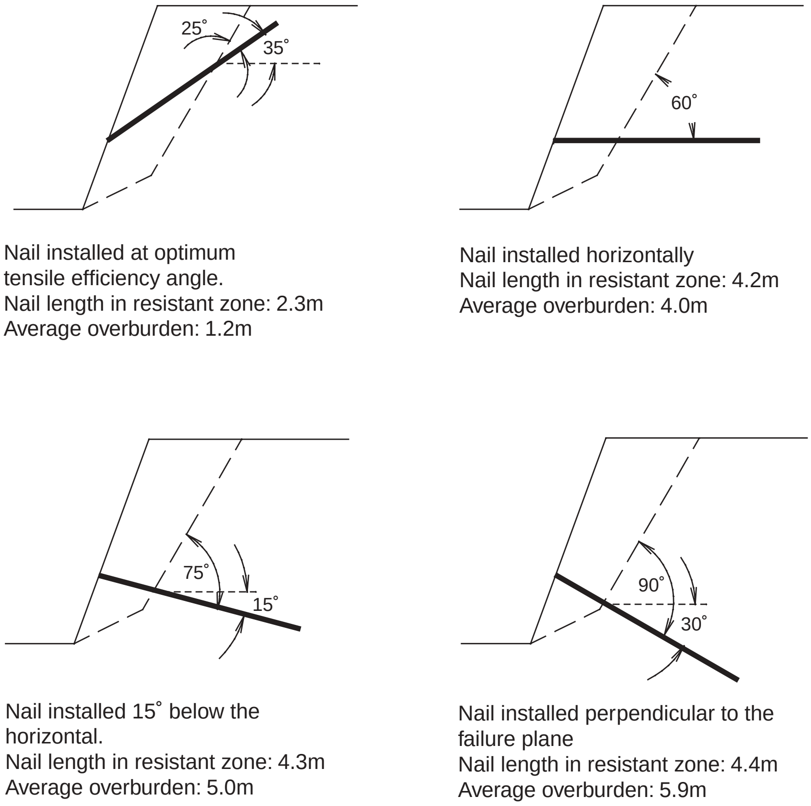

Figure 5 Nails installed at different efficiency levels: showing corresponding overburden and length in the resistant zone

4 Design considerations

This Section considers the elements of the design of a nailed slope and the factors that are important in the selection of the technique.

4.1 Professional roles

For highway schemes undertaken in accordance with HD 22 (DMRB 4.1.2), the design of a soil nailed slope requires Geotechnical Certification by the design engineer. A new procedure, *the strengthened earthworks appraisal system* is being introduced by HA.

A soil nailing system may be put forward as an alternative design by the contractor after the award of a contract. The soil nailing project may be designed and constructed by a specialised sub-contractor: the values of the design variables may be dependent on the method of construction. In such situations it is imperative that the specialised sub-contractor is aware of the client's requirements for nailed slopes and takes account of constraints from other parts of the scheme such as loading, and the presence of adjoining structures, underground services and earthworks.

The designer must also be aware of the existence of site investigation reports, and the values of the variables that are recommended in the interpretative geotechnical report. Otherwise the design could be based on a limited knowledge of the overall scheme requirements and unrealistic values for the input variables. This could result in a nail layout that either has a lower factor of safety than required or is less economic than the optimum. This is particularly likely where relatively inexperienced designers and soil nailing contractors are carrying out the design. A better solution is more likely to be produced where the designer, client and contractor co-operate and contribute their experience and expertise to the design.

Even where clear, well accepted design methods are available the designers must apply their skill and expertise to the specific case before them.

4.2 Site constraints

For each slope the designer should consider the suitability of various options that satisfy both technical and economic criteria. These options should be considered in the framework of existing documentation for the design of highway earthworks. For any slope the selection of soil strength properties and porewater pressure regime are of prime importance, as is, for a nailed structure, the corrosivity of the soil. With an *in situ* technique such as soil nails, there will be little or no control over the nature of the soil and the presence of cobbles and boulders or buried obstructions which may preclude the use of soil nails.

A general requirement for steepening existing slopes is that the soil must be sufficiently self supporting to permit the construction of benches (typically 1 m deep) while the facing and nails are installed. A soil nailed slope will be constructed from the top downwards and so adequate room is required for construction plant to form benches, to remove excavated material and to install the nails. In certain situations, the installation of soil nails may be precluded by wayleaves imposed to protect underground services and pipelines or sub-structures and foundations.

The method of nailing failed or marginally stable slopes will depend on several factors such as the type of slope, the extent and form of (potential) failure surfaces, ease of access (particularly where nails might extend under adjacent land), long-term changes in ground conditions and proximity of structures and buried services. In some circumstances, care will need to be taken to ensure that nail installation techniques do not further destabilise a slope which is already in a marginally stable state.

Care must be taken with both drilling and grouting processes. Cased holes will cost more than uncased ones but they may be essential in certain ground conditions. In general, grouting pressures are kept as low as possible commensurate with the use of fairly narrow bore tremie tubes (typically 10 to 15 mm internal diameter). However, some systems employ high grout pressures to enhance grout penetration and hence pull-out resistance.

It is important that a reliable assessment is made of the likely porewater pressures and the drainage measures necessary to keep them at an acceptable level. Further advice on drainage is given by Murray (1992).

4.3 UK design documents

There is no universally accepted document which provides definitive guidance on the detailing of soil nails together with a full design methodology. Furthermore, there is limited information to allow the evaluation of the technique compared to other methods, and on the selection of appropriate design values. There are, however, a number of UK documents which provide direction and advice:

- HA 68. *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques*. This provides a single unified design approach for all types of reinforced highway earthworks, including soil nails, with slope angles to the horizontal in the range 10° to 70° and soil types in the strength range $\phi' = 15°$ to 50°. Some guidance is given on the selection of design parameters, and on the detailing for and design of facing cover systems.
- BS 8006:1995 *Code of practice for strengthened/ reinforced soils and other fills*. This gives guidelines and recommendations for the application of reinforcement techniques to soils. Most of the document relates to reinforced earth techniques rather than soil nailing.
- BS 8081:1989 *Code of practice for ground anchorages*. Ground anchorages differ from soil nails in that they are active, pre-tensioned reinforcements. The document provides guidance on various methods of analysis. For steep and near-vertical walls, a single wedge analysis is recommended with slip circles for shallow slopes.

Table 1 provides a summary of the published methods and a comparison of the key factors which influence design. BS 8081:1989 uses a global factor of safety approach while the other methods use partial factors. As different approaches and assumptions are made in the various documents, care should be taken when comparing designs.

There are a number of common design principles in the above documents:

- All methods adopt a limit equilibrium design approach and equate a set of maximum driving forces to a set of minimum resisting forces.
- All methods only consider axial tensile forces in design, i.e. shear and bending resistance of the nails is ignored.
- Overall equilibrium in terms of slope stability, sliding and bearing capacity are considered.
- Partial factors are applied to derive a permissible stress from the ultimate strength of the materials.

Notwithstanding the above, there are significant variations between the documents:

- There is no agreement on the shape of the failure surface - this is left to the judgement of the designer.
- Partial load and material factors vary.

For ultimate limit states, overall stability, sliding and bearing capacity are important in the design of the slope as well as bond failure of the nail. For serviceability limit states, deformation limits of the slope and post-construction strain in the reinforcement would appear to be the only factors to be considered. These aspects are dealt with in BS 8006:1995 and HA 68, although the latter document does not address post-construction strain in the nail. The design philosophies of these three documents are outlined below and their applicability to the design of soil nailed slopes is discussed.

4.3.1 HA 68

While there are possible advantages in having a single approach for reinforced soil and soil nails there are also disadvantages. The design method appears to have been developed primarily for reinforced soil and then modified to cover soil nails. With full width geotextile reinforcements, a shorter length will be required behind the potential slip plane than for a soil nail. Should the soil be weaker than assumed in design it is relatively straightforward to increase the length of geotextile reinforcement to compensate.

A limit equilibrium approach is adopted based on a two-part wedge mechanism (Jewell *et al.,* 1984). For the limit equilibrium calculation, it is assumed that a set of driving forces is in equilibrium with a set of resisting forces. The driving forces are a function of the self weight of the soil plus any surcharge load and unfactored values are used. The resisting forces are represented by the shear strength of the soil and the reinforcement force for which factored design values are used. For horizontal reinforcement a unique critical bi-planar slip surface and unique out-of-balance force, T_{max} is calculated. However, for inclined reinforcement such as nails both the calculated slip surface and T_{max} vary as the nail inclination varies. Also, in order to solve the equation to give a value for T_{max} all the nail force must be assumed to act in either wedge 1 or wedge 2. This assumption is generally invalid but is necessary to simplify the mathematics. Depending on which of the two assumptions is made the shape of the failure plane and T_{max} can change significantly.

Minimum conceivable values of soil strength are used: these are supposed to reflect long-term conditions. These are represented by critical state parameters or factored peak

Table 1 Comparison of design methods and their partial factors

	HA 68	*BS 8006:1995*		*BS 8081:1989*
Design approach	Limit state	Limit state		Limit state
Analysis	Limiting equilibrium	Limiting equilibrium		Limiting equilibrium
Shape of failure surface	Two-part wedge (applies to <70° slopes)	Two-part wedge or log-spiral (applies between 10° and 70° slopes)		Single and multiple wedge for steep slopes, circular for shallow slopes <27°
Representative soil parameters	Minimum conceivable	Cautious estimate/ worst credible		No specific guidance
Water regime	Conservative values	No specific guidance		No specific guidance
Material factor on tanϕ'	Varies: 1.0 - 1.5 on residual, critical state or peak	1.0		n/a
Material factor on c'	1.0 - 1.5 on peak strength (5 kN/m^2 max.)	1.6		n/a
Material factor on Cu	Not used	Not used		Adhesion factor = 0.3 to 0.35 on Cu
Minimum surcharge	Not given	Not given		Not given
Load factors		ULS	SLS	
Vertical soil loads	1.0	1.5	1.0	Overall FS = 1.5
Vertical dead loads		1.2	1.0	
Vertical surcharge loads	1.0	1.3	1.0	Overall FS = 1.5
Non-vertical soil loads	As vertical	As vertical		As vertical
Non-vertical surcharge loads	As vertical	As vertical		As vertical
Pull-out capacity	Interface sliding factor based on tests or residual strength	1.3		FS = 3 on ultimate load to derive design load
Base sliding	Depends on interface sliding factor	1.2		Not covered explicitly

strength parameters. A factor of safety on peak strength parameters, ϕ'_{peak} and c'_{peak}, ranging between 1.3 and 1.5 is recommended. A partial factor is applied to the yield strength of a nail, and a pull-out factor is also applied to the soil/grout or nail bond strength. The latter is equivalent to an adhesion factor applied to skin frictional effects to calculate the shaft resistance of piles or ground anchorages.

The two-part wedge is a reasonable, albeit possibly slightly conservative, approach for the analysis of slopes typically steeper than 60° (Love, 1993). It can, however, be overly conservative for the analysis of shallow slopes, less than say 27°. For shallow slopes a circular failure surface might be more applicable whilst for a shallow slide a simple ‘infinite slope analysis’ may be adequate.

HA 68 does not provides guidance on serviceability limit states.

4.3.2 BS 8006:1995

This is applicable to the use of soil reinforcement techniques for both walls and slopes. Section 7.5.2 of the document gives specific, but limited, guidance for soil nailed slopes. The design philosophy is based on limit state design principles to assess external and internal stability. Partial factors of safety are adopted for ultimate and serviceability limit state criteria.

For slope stability reference is made to BS 6031:1981 for guidance on factors of safety. This document recommends a factor of safety against slope instability of 1.5 for long-term permanent works.

As with HA 68, limit equilibrium methods are used for the design of nailed slopes. Axial tensile forces are considered to be the predominant stabilising effect although Section 7.5.5.4 does mention the possibility of calculating shear effects. For the purposes of the limit equilibrium calculation, it is assumed that a set of driving forces is in equilibrium with a set of resisting forces. In particular two methods of analysis are described in detail for assessing internal stability: the log-spiral method and the two-part wedge analysis. The two-part wedge analysis is recommended for slopes because of its relative simplicity although as mentioned in Section 4.3.1 it may be over-conservative for shallow slopes.

As shown in Table 1 the soil material factors differ from those used in HA 68. Partial factors are also applied when assessing external stability and for pull-out capacity (f_s = 1.3). Following the recommendations of CIRIA Report 65

(Hanna, 1980) a partial factor (f_n) can also be applied to either the load or material partial factors to take account of the consequence of failure. Table 3 of BS 8006:1995 gives values for f_n ranging up to 1.1. This partial factor is not applicable for slopes less than 2m in height and where damage would be minimal, and a value of unity is given where failure of an embankment slope would result in moderate damage and loss of services.

4.3.3 BS 8081:1989

This provides recommendations and guidance for soil and rock anchorages. Soil nailing is specifically excluded from the standard, but, as evidenced by this report, in a number of cases its recommendations have been adopted. There are fundamental differences between ground anchorages and soil nails. Anchorages are normally widely-spaced, relatively deep and have a high pull-out capacity. They require some form of facing plate and have an unbonded length, but most importantly, they are active, pre-loaded devices unlike passive nails which do not develop any tension until ground movement occurs.

Type A anchorages most closely resemble a soil nail. A lumped factor of safety is applied to determine the ultimate pull-out capacity. The design requires consideration of the following:

- overall stability;
- depth of embedment;
- group effects;
- fixed anchor dimensions.

As shown in Table 1, lumped factors of safety are adopted for overall stability. A basic assumption is that the anchorage prestress increases the shear strength of the soil sufficiently to displace the potential failure plane beyond the fixed anchor length. The required ultimate load capacity is determined by assuming that the ground has failed along a shear surface, postulating a failure mechanism and then examining the relevant forces in a stability analysis. The load required is assumed to be transferred by end bearing and side shear. The ultimate pull-out capacity of the anchorage is based on undrained soil strength parameters for cohesive soils and drained parameters for cohesionless soils. For permanent anchorages the following minimum factors of safety are recommended:

- Design strength of tendon = 2.0
- Ground/grout interface friction = 3.0
- Grout/tendon or grout encapsulation interface = 3.0

The fundamental difference between this document and the other two is in the approach to the determination of pull-out resistance. BS 8081:1989 gives calculated pull-out resistances which are essentially independent of effective stress and are thus independent of overburden. Both HA 68 and BS 8006:1995 calculate pull-out capacity from the frictional characteristics of the soil (typically ϕ'_{crit}) and the normal effective stress acting on the nail: in such cases therefore overburden pressure has a major effect on the calculated pull-out capacity of the nail.

4.4 Typical nail geometry and layout

Bruce and Jewell (1986) reviewed a number of case studies of nailed slopes and derived a number of characteristics that provide a useful measure of the layout and performance of nails. These include:

- length ratio = maximum nail length / excavation height = L / H
- bond ratio = hole diameter x nail length / vertical face area supported by a nail = D x L / A
- strength ratio = (nail diameter)^2 / vertical face area supported by a nail = D^2 / A

Table 2 summarises these parameters for a number of UK schemes that used either drilled and grouted (D&G) nails or driven (D) nails: the table also includes the original data produced by Bruce and Jewell (1986 and 1987).

As can be seen there is a wide variation in the length ratio with respect to slope angle: there is no discernible relationship between nail length, retained height and slope angle. HA 68 appears to produce designs with long nails compared to other design methods. In terms of economics, a scheme with more, shorter nails might be cheaper than one with fewer, longer nails, especially where cased holes are required: however, rig mobilisation costs will be higher. The diameter of the nail and hole can also be an important economic factor. It would normally be assumed that a greater surface area, and thus larger borehole, provides a greater pull-out resistance. As the diameter of the borehole increases (say, above 300 mm) the potential contribution to pull-out resistance from shear and bending moment increases. Thus larger diameter nails are more able to provide a dowelling action and this aspect may need to be considered in design.

In developing a nail layout, vertical spacing is generally determined by the stability of the benches for installation (often 1 m). The critical height for stability, h_c, of a vertical bench is given by the equation:

$$h_c = 2c' / \gamma (K_a)^{0.5}$$

where: K_a = coefficient of active earth pressure
γ = unit weight of soil
c' = soil cohesion

The critical height is dependent on the soil cohesion available during the construction period. For the short-term (say one day) a value based on the undrained shear strength (Cu) rather than c′ would be more appropriate.

The necessary restoring force is usually calculated for a unit horizontal length of slope or wall, and various horizontal spacing and nail lengths are tried until the layout provides sufficient additional restoring force to stabilise the structure. On nearly all schemes examined to date (see Appendices A to I) a constant nail length was used throughout the works, primarily to simplify installation operations. There is, however, no technical reason for this and different nail lengths may be used.

Typical horizontal and vertical spacings are of the order of 1 to 2 m. HA 68 recommends a maximum horizontal and vertical spacing of 2 m. For steepened slopes, the designer must judge the most suitable layout bearing in

Table 2 Soil nail stabilisation parameters for slopes

Reference	*Nail type*	*Length ratio*	*Bond ratio*	*Strength ratio x 10^{-3}*	*Remarks*
Bruce & Jewell (1986 & 1987)	D&G	0.28-0.35	0.82-1.22	0.4	70° slopes in granular soils.
Bruce & Jewell (1986 & 1987)	D&G	0.5-1.0	0.16-0.18	0.1-0.27	80° slopes in glacial till / mudstone.
Bruce & Jewell (1986 & 1987)	D	1.0	0.92	1.39	80° slopes in glacial till / mudstone.
Barley (1993)	D&G	0.42-1.0	0.15-0.36	0.2-0.28	Steep slopes >45° in cohesive soils.
Pedley & Pugh (1995)	D&G	0.63-1.1	0.3-0.4	0.2-0.33	70° cutting in silty clay and clayey sand.
Whyley (1996)	D&G	1.1-1.6	0.4-0.5	0.7	40° slope in silty sand and clay.
Scheme 1 Appendix A	D	0.625	0.22	1.7	45° slope in London Clay - temporary works.
Scheme 2 Appendix B	D	2	0.175	1.4	56° slope in clayey sand.
Scheme 3 Appendix C	D&G	2.2	0.055	0.24	68° slope in weathered mudstone fill.
Scheme 4 Appendix D	D&G	1.3	0.195	0.3125	Strengthening of existing 22° slope in London Clay.
Scheme 5 Appendix E	D&G	1.38	0.67	0.26	24° cutting slope in London Clay.
Scheme 6 Appendix F	D&G	1-3	0.037-0.2	0.4	68° steepened cutting slope.
Winter and Smith (1995)	D&G	0.75	0.6	0.625	55° steepened slope in Glacial Till.

mind the stable height of construction benches, the restoring force required, and the strength of the facing to support point loads applied at the nail head.

In theory, there are many possible combinations of nail spacing and length which satisfy the requirement for internal stability, namely that the sum of nail pull-out in the resistant zone and the sum of nail strengths are each greater than the required restoring force for the critical failure surface. Reinforcing systems have scope for redistributing the load between elements. But to limit excessive movement and to prevent overstress of a layer of reinforcement, which could lead to progressive failure, local balance between restoring and disturbing forces should be considered:

- BS 8006:1995 states that the adherence capacity of each layer of reinforcement should be compared with the local force to be resisted. However, this appears to relate primarily to the use of horizontal reinforcement in fills attached to small facing units.
- HA 68 provides rules for 'optimising' the vertical spacing of nails in slopes by varying the spacing of the nails throughout the slope.

All methods of designing stable slopes require checks on external stability (sliding or rotating on a deeper failure surface) and overall slope stability. Longer nails (at the top or bottom of the nailed slope) may be required to satisfy external and overall stability than are required to satisfy internal stability. The T_{ob} mechanism given in HA 68 is a useful means of checking the basal sliding of the reinforced block.

HA 68 recommends the checking of potential mechanisms beyond the assumed 'critical' failure plane, since these may require anchorage lengths beyond that required for the 'critical' mechanism. While a check of alternative failure planes for internal stability is not advocated in BS 8006:1995 a designer might wish to do so, particularly if the original design minimised costs by reducing the nail lengths to the minimum.

4.5 Design parameters

4.5.1 Soil parameters

The selection of the values for the soil variables for the design of a nailed slope requires careful judgement by a geotechnical engineer experienced in the interpretation of site investigation data. The nail layout can be very sensitive to variation of these values and their selection is therefore critical if safe and economic designs are to be developed.

The selection of soil strength parameters for design requires an understanding of what geotechnical processes are involved and what might influence the measured values. It is important to select parameters that reflect the long-term soil behaviour. This can be a difficult process since the operating strength is a function of the soil stress state, porewater pressure and overburden pressure. It is well established that the stress-strain behaviour of most soils is highly non-linear over the normal range of strains of interest in the design of slope stabilisation works. The peak effective angle of friction is not a material constant and varies with for example density, over consolidation ratio and the effective normal stress. The critical state friction angle, ϕ'_{crit}, is a material constant and is, therefore, a more reliable measure to use in design.

BS 8006:1995 recommends the use of design strengths based on peak strength parameters (c'_{peak} and ϕ'_{peak}). However, Section 2.5 suggests the use of characteristic values based on a cautious estimate of soil strength while Section 5.3.4 recommends design values as being the worst credible value divided by a partial factor f_{ms}. As the value of f_{ms} is generally unity (Table 26 of BS 8006:1995) the design soil strength is not reduced below the worst credible value. The use of peak values with a partial factor of unity could be considered unconservative to the point of being unsafe but the package of partial factors given in BS 8006 is intended to provide an overall factor of safety similar to those inbuilt into to earlier design methods. Farrar and Murray (1993) suggested that the mobilised shear strength (ϕ'_{mob}) at K_o conditions would be equal to peak shear strength (ϕ'_{peak}) unless the soil has been previously subjected to significant strains.

The approach in HA 68 is to use minimum conceivable values for design represented by factored peak strength values where:

$$\tan \phi'_{des} = \tan \phi'_{peak} / f_s \qquad c'_{des} = c'_{peak} / f_s$$

or,

$$\tan \phi'_{des} = \tan \phi'_{crit} \qquad c'_{des} = c'_{crit} = 0$$

In slopes with pre-existing shear surfaces it will be necessary to use residual shear strength parameters (c'_{res} and ϕ'_{res}). Indeed for some slopes residual strength may develop over the design life due to natural weathering or progressive movement of the soil.

The adoption of undrained soil parameters together with a lumped factor of safety (the approach adopted in BS 8081:1989) has the advantage of simplicity but is not recommended for nailed structures, because of the difficulty in evaluating undrained strength, particularly locally to the ground/grout interface.

Crabb (1994) concluded that the widespread problem of shallow failures in highway earthworks, particularly in over consolidated clay, is caused by a combination of two effects. The first is the swelling and softening of the near surface material, which under the influence of shear strains in the slope, reduces its strength towards the critical state. The second is the equilibrium of infiltration of rainwater into the slope with groundwater flow through the slope and evapo-transpiration at the surface. Crabb (1994) found that the depth of the failure surface was controlled by the resulting distribution of porewater pressure. Deeper failure surfaces are unlikely to develop because porewater pressure reduces with depth. It was concluded that there was little evidence that this regime was likely to change in the long-term.

4.5.2 Loading

Some designers have found BS 8006:1995 difficult to interpret regarding partial load factors. Section 2.4 of BS 8006:1995 advises that dead loads should be calculated using the unfactored self weight of the soil. However, Tables 26 and 17 of BS 8006, indicate that the factor f_{fs} = 1.5 should be applied to the soil mass when calculating 'disturbing forces'. There have been different interpretations of whether the 1.5 value should be applied when calculating the pull-out resistance of the nail. Applying the 1.5 factor will result in higher vertical stresses and higher design pull-out resistances than are theoretically justifiable. However, this may be acceptable because the partial factors given in BS 8006 are intended to be used as a package. Usually the use of peak strength values with a partial factor of unity would be considered unsafe for long-term design (Table 26).

HA 68 does not provide any specific guidance on partial load factors to be used for ultimate and serviceability limit state conditions. This is because partial factors are only applied to the soil strength, see Section 4.5.1.

4.5.3 Groundwater

Porewater pressures can substantially affect the stability of a nailed slope. Higher porewater pressures require a higher restoring force to maintain the stability of a potential failure zone, and they also reduce the effective stress acting on the nail/ground interface along which pull-out resistance is generated: both increase the nail length required to maintain stability.

A design based on effective stresses requires a knowledge or estimate of the likely porewater pressure regime in the ground both at construction and in the longer-term as steady state seepage and infiltration conditions develop. Often only limited information is available to the designer regarding the existing porewater pressures and for estimating long-term conditions. It is important that as much information as possible is obtained during the site investigation for the works.

Studies by Crabb and Hiller (1993) and Crabb (1994) on shallow failures in highway slopes in over consolidated clay have shown that surface water infiltration can result in high seasonal porewater pressures at shallow depth which reduce with depth. They concluded that these effects are likely to be more apparent on shallow slopes with little or no vegetation than on steep slopes which allow rapid run-off of surface water. It is therefore necessary when considering porewater pressure distribution to consider the effects of surface water infiltration and groundwater regime.

The methods for including porewater pressures in an analysis are rather imprecise. An appropriate value for the porewater pressure parameter (r_u) may be estimated and included in an analysis but this cannot readily account for high porewater pressure from surface infiltration. Positive porewater pressures will reduce the effective stress giving a lower resisting force along any potential failure surface and a lower pull-out resistance for any particular nail. Alternatively a groundwater profile or flow net may be assumed. If a potential failure plane and the layout of the nails is superimposed onto the groundwater profile or flow net, the designer can determine the out-of-balance force and nail pull-out resistance can be determined by estimating the porewater pressure, and hence the effective stress, at various locations. The results obtained through such an approach should again be regarded as only an approximation to the likely in-service condition.

Drainage measures will generally be cost-effective in helping to stabilise all slopes including nailed slopes. Carefully installed cut-off drains behind the crest of a slope will help minimise surface water entering the slope. Care should be taken generally in detailing the drainage to minimise the surface water entering the slope. The drainage systems employed will need to be robust, long-lived and capable of inspection and maintenance during the life of the structure. Further advice on drainage is given in Murray (1992).

4.6 Soil/nail interaction

The ability of a nail to generate sufficient pull-out resistance is of fundamental importance to the stability of a nailed slope. For reinforced earth BS 8006:1995 and the earlier BE 3 (DMRB 2.1), require the pull-out resistance to be determined from the surface area of a reinforcing strip, the vertical effective stress and the coefficient of friction between the soil and strip. For straight, flat strips which are placed and subsequently covered by a frictional fill this approach is satisfactory. However, even for this relatively straightforward case it is difficult to predict the ultimate pull-out resistance accurately. One major complicating factor is an effect sometimes termed ‘constrained dilation’ where the soil dilates to accommodate the movement of the strip through the ground. This movement is prevented by the surrounding soil (providing it is not in a loose condition) and an increasing force can be applied to the strip until some of the soil grains start to crush or passive failure occurs in the surrounding soil permitting the strip to move. Because of this the measured pull-out values for strips are almost always substantially greater than calculated.

When nails are installed in natural ground there are additional complicating factors. Nails are more likely to be used in clayey soils and thus the estimate of porewater pressures is more likely to be a problem. Where nails are installed by a displacement technique, such as firing, this will tend to increase the normal stress in the soil surrounding the nail, thus increasing pull-out resistance, at least in the short-term. If the borehole for a grouted nail is not straight or if the sides of the hole are rough, the nail is likely to generate a higher pull-out resistance. Where grout enters fissures or adheres to cobbles adjacent to the borehole, higher pull-out capacities are again likely.

The most appropriate method of calculating pull-out resistance appears to be that given in Appendix D of HA 68. Normally, pull-out tests are carried out at the start of the works to confirm that measured pull-out values match or exceed the expected values. It is recommended that this approach is maintained to help provide confidence in the works. However, where a large number of pull-out tests on a scheme consistently give significantly higher values than those calculated using HA 68, an experienced designer may wish to re-examine the analysis and up-rate the pull-out resistances of the nails. However, up-rating should be done with caution: the long-term performance of the nails must be carefully assessed. Further discussion on the interpretation of pull-out tests is given in Section 5.

4.7 Partial factors

In a limit state approach to design, partial factors should be related to the level of uncertainty associated with a variable or method of analysis. Thus for a material property, a large partial factor value would be applied where there was a high level of uncertainty, but a smaller value would be applicable where the range of values was small and clearly defined. However, the approach taken in BS 8006:1995 does not follow this philosophy. The values of the partial factors are based on a calibration exercise which was adjusted to give similar designs to those obtained using earlier methods. The calibration exercise was based on reinforced fills and did not include *in situ* techniques such as soil nails. Thus the partial factor values given in BS 8006 might not be the same as those derived through engineering judgement and experience.

Values of the partial factors given in current design codes are summarised in Table 1. The following points should be noted:

- In BS 8006:1995 lower values of the partial factors for external loads are given in Table 26 (relating to slopes) than in Tables 17 and 18 (relating to walls). This might reflect the greater consequences of failure for a wall than a slope, but it might not be true for all highway slopes.
- There have been different interpretations of the requirements in BS 8006:1995 regarding the factor f_{fs} applied to the weight of the soil. One interpretation is that it should be applied to all calculations (both disturbing and restoring). Another is that it is inappropriate to apply f_{fs} to the calculation of pull-out resistance. This would imply a greater normal force on the nail than one could reasonably expect (see Section 4.5.2).
- A lower partial factor (of unity) for the weight of soil is applied in HA 68 than given in Table 26 of BS 8006:1995, but a higher factor is applied to the peak angle of shearing resistance ϕ'_{peak} (both relating to slopes). Using a partial factor of unity on ϕ'_{peak} as in BS8006 would normally be considered unsafe for long-term design.
- The partial factor value for pull-out of 1.3 given in BS 8006:1995 is considerably lower than the global factor of safety on pull-out of 3 defined in BS 8081:1989.

For nailed walls, where large movements are not expected, it would appear more reasonable to base the design soil strength on peak values. For consistency with BS 8006:1995, the factor f_{ms} should be applied (generally unity to ϕ'_{peak} and 1.6 to c'_{peak}). For slopes, larger movements can be tolerated and it is unreasonable to assume that ϕ'_{peak} will operate in the long-term. However, if a lower partial factor for soil strength is adopted as in HA 68, it will be necessary to assess the values of the other partial factors in Table 26 since, as mentioned previously, these values are a ‘package’ meant to be used in combination.

4.8 Displacements

Deformation (either during or after construction) is required in a soil nailed slope to mobilise tension in the nails and reach a state of equilibrium. Soil nailed slopes should not therefore be used where significant movements cannot be tolerated during the service life of the structure. The use of pre-tensioned ground anchorages is likely to be a suitable technique for controlling movement. Some schemes have used a combination of nails and anchors (Clouterre, 1991, Figures 31 to 33) to try to obtain the benefits of both techniques.

A nailed slope should be sufficiently flexible to allow the slope to deform and mobilise tension in all nails, but sufficiently rigid to permit load sharing between the nails. The normal method of construction for steepened slopes, from the top down in benches, will tend to permit greater movement at the top of the slope. This, in turn, will tend to generate higher mobilised tensions in the upper nails than in the lower ones. Thus, while calculations may indicate a greater pull-out capacity towards the base of a slope (greater effective length and overburden) in practice larger tensions may be generated in the upper layers.

Woodward (1991) suggested that to minimise the deformation in excavations stabilised by soil nailing the cut depth at each stage of excavation should be kept small, say 1 to 1.5 m, and the facing and nailing completed during a working shift. Consideration should also be given to excavation in panels or additional temporary support such as external propping.

Field data from nailed structures show that deformation can continue after the end of construction (Juran and Elias, 1991; Kakurai and Hori, 1991; Woodward, 1991) even in cohesionless soils. Centrifuge model testing of a nailed clay slope indicated that changes in nail load occurred during installation and excavation of the cut slope (Jones, 1999). These changes continued for a significant time after installation due to changes in porewater pressure. Deformation can also be particularly sensitive to climate (Juran and Elias, 1991) and effects such as freezing should be taken into account in design. In certain circumstances, post-construction monitoring may be useful.

4.9 Facing

The surface of a nailed slope may require a facing to:

- redistribute pull-out forces between nails as movement of the slope takes place;
- provide a reaction for individual nails to mobilise tensile forces;
- prevent or reduce surface water uptake and local increases in porewater pressure at shallow depth;
- prevent localised failure between nail heads.

Typically a facing may be formed using a geosynthetic or steel mesh, which is fixed to the nail heads by a plate held in place by a nut. For steep slopes the mesh may not be sufficient to maintain local stability and a more ‘rigid’ panel may be used e.g. constructed out of steel mesh and filled with soil or rock. Where desirable, or for structural purposes, concrete panels or block walls may be installed.

BS 8006:1995 makes a distinction between ‘steep slopes’ (slope angles > 45^{o}) and ‘shallow slopes’ (slope angles ≤ 45^{o}). BS 8006 advises that it is usually necessary to provide some form of facing for steep slopes to provide anchorage of the reinforcement in the active zone and to provide erosion protection. For ‘soft’ facings, it is difficult to establish permanent vegetation to cover the exposed face of steep slopes. This is easier for shallower slopes, but it may take some time for vegetation to become established. It is therefore essential to provide short term protection against erosion. Snowdon (1997) states that vegetation can play an important role in preventing shallow slip failures by the removal of the excess moisture in the slope and providing tensile reinforcement through the roots. To provide an environment in which vegetation can become established, he suggests the following:

- provide an adequate depth of topsoil to reduce desiccation, particularly on south facing slopes;
- include water-retaining polymers and slow release fertilisers;
- incorporate seeds within the topsoil;
- use a geogrid for the facing, possible backed with an open textured biodegradable fabric;
- plant rambling and climbing evergreens at the top and on the slope face;
- include some form of mini-benching to provide a catchment area to retain run-off;
- incorporate a hard, steep lower face, whilst reducing the angle of the upper vegetated slope;
- time the planting in relation to construction activities and plant welfare;
- consider maintenance operations including safety, access and timing.

4.10 Durability

In common with other highway earthworks in the UK, the required service life of a nailed slope is normally 60 years. For all works, a corrosivity assessment should be made of the soil or fill to determine the suitability of the nails. There is little advice on durability in HA 68, and while BS 8006:1995 does cover durability it is written mainly for walls rather than earthworks. Table 4 of BS 8006:1995 gives limits on suitable fill to be used in reinforced earth construction (and is thus applicable to structures rather than *in situ* slopes). This table has been amended by BD 70 (DMRB 2.1) to make it applicable to natural soil as well as fill, and also to delete the option of using ungalvanised steel for reinforcement for structures. Where soil nailing is proposed for highway slopes there is no requirement to adopt the recommendation in BD 70 or Table 4 of BS 80006:1995. A separate evaluation of soil aggressivity, using an approach such as that given by Murray (1992), is recommended.

The first difficulty which a designer may encounter is that unless soil nailing was considered at an early stage (perhaps at the desk study) the site investigation will probably not have included the tests required by Murray (1992) or BS 8006:1995. Thus the designer will be unable to assess whether the soil falls within the corrosivity limits. If sufficient time is available it may be possible to arrange a supplementary site investigation to assess the corrosion potential of the soil (and possibly better define the strength of the soil and porewater pressures). In practice it is much more difficult to provide a comprehensive assessment of natural soils than excavated fills.

The aggressivity assessments given in Table 4 of BS 8006:1995 and Tables 3 and 4 given by Murray (1992), require similar sets of tests. Where the BS 8006 approach is followed and the soil is less aggressive than the limits set in Table 4 of BS 8006, galvanised steel or stainless steel nails may be used and sacrificial thicknesses for a 60 year design life in these conditions are given in Table 7 of BS 8006. Examples of suitable materials are given in Table 6 of BS 8006. For HA structures, BD 70, Section 3.2.1 states that the steels must comply with the British Standards listed in Table 6 of BS 8006 or have a current BBA certificate. While certificates have been issued for strips and anchors for reinforced earth construction, at present no such certificates have been issued for soil nails.

Steels other than those listed in BS 8006 Table 6 might also be suitable for soil nails for slope strengthening or steepening. Soil nails are not normally highly stressed and steel of a relatively low or medium tensile strength is usually satisfactorily (e.g. Grade 250 or 460 to BS 4449:1988). However, designers or contractors may specify higher strength steels, in bar or tube form, such as used in rock bolting or pre-stressing applications. High tensile steels can be weakened by hydrogen embrittlement, sometimes associated with the acid pickling process prior to galvanising. Generally such problems relate only to extremely high tensile steels, with ultimate tensile strengths greater than, say, 1000 N/mm^2.

Where the advice in BS 8006 is followed, steel for galvanising should have a silicon content which readily permits a zinc coating weight of not less than 1000 g/m^2 (Section 3.2.2.1 of BS 8006 and Section 3.2.2 of BD 70). Alternatively, a high coating thickness may be achieved by grit blasting and pickling prior to galvanising; or grit blasting may be sufficient on its own. Experience has shown that austenitic types of stainless steel (as given in Table 6 of BS 8006:1995) are suitable but ferritic stainless steel is unsuitable, because of its tendency to pit in the presence of chloride ions.

The approach given by Murray (1992) divides soils into four categories; non-aggressive, mildly aggressive, aggressive and highly aggressive, and is based on the data given in Eyre and Lewis (1987). It recommends that permanent nailed structures should not be constructed in highly aggressive soils. Annual rates of galvanising loss are provided for the three remaining categories. For an initial coating weight of 1000 g/m^2 (140 microns) these equate to galvanising 'lives' of 35 years in non-aggressive conditions, 18 years in mildly aggressive conditions and 10 years in aggressive conditions. In practice the corrosion of both the galvanising and the underlying steel is unlikely to be uniform but the guidance given by Murray (1992) is probably the best currently available. To comply with BD 70, the substitution of the galvanising by an additional sacrificial thickness of steel is not permitted (amendment to Table 7 of BS 8006:1995). The corrosion resistance of materials other than galvanised steel is not covered by Murray (1992).

An alternative approach permitted in Section 3.2.2.2 of BS 8006:1995 is for nails to be protected in accordance with the recommendations for corrosion protection in BS 8081:1989. This states that 'double protection' is required to reduce the possibility of corrosion to a negligible level. For permanent work this can be achieved using two concentric sheaths filled with grout (Figures 19 and 20 of BS 8081:1989).

Glass reinforced plastic (GRP) tube typically with an outside diameter of 22 mm and an inside diameter of 12 mm, has also been used for nails. While GRP does not corrode its strength can be significantly reduced primarily through the mechanism of stress corrosion. Mallinder (1979) and Greene and Brady(1994) recommend that the 120 year working strength of a GRP reinforcement should be taken as about 10% of its short-term ultimate tensile strength. A rather higher strength could be assumed for a 60 year design life.

Some general advice on durability and corrosion resistance will be given in the CEN Execution Standard for Soil Nailing currently being drafted.

5 Interpretation of pull-out tests

The relation between the short-term pull-out resistance of a nail and the long-term restoring force available from that nail is complex. No pull-out test can replicate the situation when the active block of soil starts to move, and because the stress regime across the potential failure surface is likely to be different in the two cases, the stresses on the resistant part of the nail will be different. Also, in a pull-out test the loading is axial whereas at the actual failure condition there is likely to be a combination of bending and tension forces acting near to the failure plane. For simplicity (and consistency with the design assumptions) it is assumed that soil movement occurs along a defined failure surface rather than being spread over a wide failure zone although the latter may be more realistic in some cases.

Generally a designer will be interested primarily with the resistance to pull-out generated behind the potential failure surface. This can be estimated by undertaking a pull-out test on a nail grouted along its full length, and calculating the 'useful' pull-out from the ratio of the effective length of nail (in the resistant zone) to the total nail length. This fairly straightforward approach could be considered to give a reasonably conservative result because soil strength tends to increase with distance from the face so that the bottom half of the nail should generate more pull-out resistance than the top half. Conversely it could be argued that if loose soils or fills are present just behind the face then greater grout penetration would occur and a greater contribution to pull-out would be provided by the upper portion of the nail. Test loads are normally applied by means of a hollow hydraulic jack on a reaction plate. It is important, but especially so with the test technique described above, that a reaction frame is employed during loading. This should be designed such that the reaction force is applied to the soil some distance from the nail to minimise additional normal stress on the nail and hence pull-out resistance due to the application of the test load. Reaction frames are typically about two metres wide and thus load the ground about one metre from the nail.

Another approach is to sleeve that part of the test nail passing through the active wedge enabling measurement to be made of the pull-out generated in the resistant zone alone. Possible disadvantages of this, apparently more realistic method, are that the stress regime in the soil is not as it would be on the point of failure, and also the grout around the sleeved length of nail could provide some additional pull-out resistance. It may be possible to fit borehole packers around the nails to ensure that only the resistant zone is grouted and thereby provide a more realistic test.

An effect observed in granular soils, especially with rough or ribbed reinforcement, is that of constrained dilation (Schlosser, 1979; BS 8006:1995). As a nail or other reinforcing strip starts to pull-out, adjacent soil particles have to slide or roll over one another. They are prevented from moving readily by the constraint provided by the surrounding soil, and thus higher than calculated pull-out resistances are generated. In BS 8006:1995 this effect is discussed in terms of μ^*, the apparent coefficient of friction. Work by Schlosser and others has indicated that this is a marked effect at low cover depths - say less than 3m - but there is a significant reduction in the constrained dilatancy effect as cover depth increases beyond this. There are other physical factors which can produce higher pull-out test results than calculated, and these may be considered either as features of constrained dilation or as separate mechanisms. These include non-straight boreholes (either curved or dog-leg) and the presence of fissured or non-homogeneous ground.

Such effects may also be present but to a lesser extent in cohesive soils. However, the short-term pull-out resistance of a nail installed in clay may be higher than attainable in the long-term since porewater pressures during construction (and testing) could be lower than those encountered during the service life of the structure. This approach would be consistent with the concept of using undrained shear strength (Cu) for short-term soil behaviour (say the excavation of benches) but effective stress parameters, ϕ' and c', for the long-term condition. It is also possible that the movement and stresses generated during testing could produce temporary porewater suctions locally, leading to higher effective stresses on the nail and enhanced pull-out resistances.

The results of pull-out tests undertaken on all the schemes reviewed in Appendices A to H produced higher values than those calculated using the design equations given in HA 68. The engineer must know whether unfactored ‘best estimate’ values have been employed or factored values relating to the long-term condition (and including allowances for uncertainties) when making such calculations. It would be unrealistic to expect fully factored ‘design’ pull-out values to be close to measured site values. In practice, the actual pull-out resistances should always exceed the design values. On one scheme, (not in the appendices) low pull-out resistances were attributed to the tremie tube not reaching to the bottom of the boreholes. It is conceivable that a combination of adverse factors could lead to pull-out resistances lower than those calculated. For example, consider a smooth, straight borehole drilled in a material such as chalk: if water was employed during boring then a layer of smeared, weak material could be left on the side of the borehole. If the grout had a high water/cement ratio, this could further wet and weaken the chalk. The grout might also weaken through segregation and by shrinking away from the side of the borehole.

The method given in Section 2.27 of HA 68 for calculating pull-out resistance assumes that it is directly proportional to the effective length (L_e) of the nail, and implies that it will be uniform along the length of the nail (for a constant cover depth). This is unlikely to be the case, but it may be approximately correct at the onset of grout/ground failure. The development of pull-out resistance appears to be as follows. During a test the load applied at the nail head is shed quickly into the surrounding soil; the distribution is largely dependent on the relative stiffness of the nail and soil. The grout/ground interface nearest the nail head will be subject to the highest shear stress. With increasing pull-out load, the interface stress near the nail head exceeds some threshold value (which will be influenced by all the factors discussed above) and the contribution to resistance at that point may fall from some value related to ϕ'_{peak} through one related to ϕ'_{crit} or even $\phi'_{residual}$ should sufficient movement occur. A greater contribution to pull-out is then required of the next section of nail until sufficient relative movement occurs to reduce the available resistance from this section from ϕ'_{peak} to ϕ'_{crit} or $\phi'_{residual}$. Thus as long as the rupture strength of the nail is not exceeded, a progressively greater contribution to pull-out is provided by the deeper parts of the nail until eventually the maximum pull-out capacity is exceeded.

This progressive development of pull-out resistance is also likely to occur in an in-service nail except that the maximum load will be developed at the failure plane. Under working condition, it is unlikely that there will be a uniform stress distribution along a nail and thus the design assumption of a uniform pull-out resistance being developed along the nail is unlikely to be correct. This progressive development of pull-out resistance may provide a more accurate model of nail behaviour at failure but it cannot at present be readily employed in the design process.

6 Case studies

Appendices A to H present case studies where data have been gathered on the design and, where applicable, construction of soil nailed slopes. They highlight the areas of the design where assumptions have been made or there are inconsistencies in the approach compared with other case studies. Details of pull-out tests performed as part of the works, or by TRL as part of this project, are also included. The data sheets for some further case studies are given in Appendix I. The amount of information available for each scheme varies considerably. For some schemes, data were extracted from published articles and papers: for the remainder, information was supplied by parties involved in the scheme.

At Scheme 8 (see Appendix H) long term tests were undertaken on one 5.5 m long driven, and one 5.5 m long drilled and grouted nail. These tests were undertaken to ascertain whether the pull-out load at failure varied with the rate of pull-out. In the long term tests, the load on each nail was increased in increments, allowing three to four weeks between each increase in load. During this period, the jack was locked off and a series of compressible spherical washers was incorporated in the system to reduce the loss of load. In practice, despite these precautions, the applied load reduced by between 10 and 40% during the periods between the application of the increments of load, the percentage drop reducing as the load increased. The fall in load was thought to be mainly due to the bedding in of the reaction beams into the slope.

In both cases, the pull-out load at failure was less than that obtained for the rapidly tested nails. The displacement of the nail before the onset of failure was also smaller than that obtained in the rapid tests. The peak loads obtained from the rapid tests were 70 kN (percussive) and 66 kN (drilled and grouted) whereas the long-term tests gave loads of 60.5 and 59 kN (i.e. about 14 and 13% lower).

Similar long term tests were carried out on Scheme 5 (Appendix E) where three rapid tests were compared with three slow tests (over 20 months). The test nails on this scheme were drilled and grouted and were 6m long. The opposite result was obtained here in that long term test pull-out (average 214 kN) was greater than the rapid pull-out (average 193 kN) i.e. about 11% higher.

For most of the case studies presented in Appendices A to H, the measured pull-out resistance of the trial nails, P_{mes}, has been compared to the calculated pull-out resistance, $P_{calc.}$ The pull-out resistance of the trial nails was calculated using the method in HA 68 and based on the total installed nail length (L_t) rather than the effective length (L_e) of nail lying behind the potential failure plane. In some cases the calculated pull-out resistance is derived from both the design effective strength values, c'_{des} and ϕ'_{des}, and the peak strength values, c'_{peak} and ϕ'_{peak}. For comparison, the pull-out resistance has also been calculated using undrained parameters taking a mean Cu for the soil from site investigation data and a value of adhesion (α) of 0.45. This follows the practice for calculating the skin friction developed on piles (Tomlinson, 1995; Skempton, 1959).

From back analysis of all the pull-out test data tabulated in Appendices A to H, the following values of P_{mes}/P_{calc} have been obtained. Using design effective strength parameters, the mean value of P_{mes}/P_{calc} is 4.6. Using an estimated value of Cu and α of 0.45, the mean value of P_{mes}/P_{calc} is 1.9. The average value of the pull-out factor (λ) defined in HA 68 Section 2.23 is 1.1.

7 Discussion and conclusions

7.1 General considerations

At present, there is no single, universally accepted document or computer package which contains a detailed, comprehensive design method for soil nails. However, the advice given in a number of documents, allied to good geotechnical input, will result in satisfactory designs. It is inevitable that as this is a relatively new and complex technique there are significant uncertainties in design.

The basic method of analysis is to identify the failure plane which generates the largest out-of-balance force (T_{max} or M_{max}) for the completed earthwork. The slope angle will influence the choice of the method of analysis. For minor works or where the consequences of failure are low, simple methods of analysis for design, based on conventional slope stability analysis of the unreinforced slope, are valid. For more critical, complex situations, a more detailed analysis will be required perhaps with the design checked against an independent method.

Some designers are reluctant to adopt HA 68 because they find it difficult to use and consider the resulting designs to be overly conservative. The limitations and difficulties implicit within the calculation procedure mean that designers and checkers find it difficult to develop an intuitive feel for the design. The limitations include the restriction to a single layer of soil, and the difficulties include assessment of the effects of the orientation of the nails and the distribution of pull-out forces between the two wedges. The approach adopted in HA 68 is to analyse the reinforced condition, whereas some in-house designs are based on an initial analysis of the unreinforced condition, with nails designed to counter the calculated out-of-balance force. Some designers consider this to be a more logical approach than that given in HA 68.

The design of a soil nailed slope requires a high level of geotechnical expertise. This input can generally be ensured for schemes undertaken for major clients where a well-established consulting engineer is likely to be the designer and a second well-established consulting engineer acts as a checker.

New works, i.e. steepened cutting or embankment slopes, are feasible only where a stable temporary soil face can be excavated and reasonably high pull-out resistances can be achieved. Soil nailing is, therefore, unlikely to be suitable in soft clays, peat, loose granular deposits with little fines content or where cobbles, boulders or other obstructions preclude the installation of soil nails. The suitability is also influenced by the general topography, available land, ownership and ease of access. With the present level of experience of soil nailing, it is recommended that nails are not used in situations where large cyclic or dynamic loads might apply.

7.2 Detailed design

A general indication of an approximate layout of a soil nail array may be obtained from Table 2. Often the properties of natural soils and *in situ* porewater pressures will mean that nails will need to be longer than for other reinforced soil techniques. Uniform spacing and nail length have normally been used, but these might vary according to economy of construction and the extra cost of more complicated site installation practice.

The dimensions and spacings of soil nails are very sensitive to the strength assumed for the soil. The determination of realistic long-term soil strength parameters c'_{des} and ϕ'_{des} is therefore critical if a safe and economic design is to be achieved. A small cohesive strength is beneficial to the stability of a slope but it is difficult to predict with confidence. Designers have tended to take c' as zero and it is recommended that this is generally followed unless there is a clear reason for using a higher value.

Porewater pressure is an important factor affecting the stability. An appropriate value for the porewater pressure parameter (r_u) may be estimated and included in the design. Alternatively the designer may calculate an assumed groundwater profile or use a flow net and determine the out-of-balance force and nail pull-out resistance by estimating the appropriate porewater pressure at various locations for a given nail layout. In practice it appears that slope failures are caused primarily by infiltration of rainwater into the slope softening the top metre of material resulting in a shallow slip. Where high positive porewater pressures are anticipated a technique other than soil nailing may be more appropriate. Generally drainage measures should be included or at least considered for works incorporating nails.

The nail installation angle (δ) has a significant and complex effect on the performance of a nailed slope. It affects:

- the relative contributions from axial load and bending;
- the angle of the critical failure wedge;
- the maximum force required for stability ($T_{max\delta}$);
- the average overburden acting on a nail and hence its pull-out resistance;
- the length of nail in the resistance zone and hence its pull-out resistance.

For ease of grouting and speedy generation of tension with soil movement it is suggested that a value of δ of between 10º and 20º be chosen unless there are any over-riding considerations.

In theory, there are many possible combinations of nail spacing and length which satisfy the requirement for internal stability. A simple analysis could be undertaken, assuming that a uniform force is required from all the nails. This is a reasonable assumption where the slope or facing would redistribute any locally high loads. For steep slopes the observed deflections (Clouterre, 1991; Farrar and Murray, 1993) show that greater deflections (and possibly greater tensions) are developed in the upper nails. The simple approach of ensuring that the total nail force exceeds the total out-of-balance force appears reasonable.

It is recommended that nail lengths are rounded up rather than down to ensure sufficient length is provided in the resistant zone. Similarly, when preliminary nail layouts provide significantly more pull-out than is required it is suggested that nail spacing be increased (if appropriate) rather than the nail lengths reduced.

Some deformation of a soil nailed slope is required to mobilise tension in the nails (above any small tensions developed during construction) and to reach a state of equilibrium. Soil nailed slopes are not appropriate, therefore, in situations where some movement of the slope cannot be tolerated during the service life of the earthwork.

The required service life of the soil mass is 60 years. For both new and strengthening works, corrosivity assessment must be made of the soil to determine its suitability for the nails. But inevitably, not all the necessary information will be available and assumptions and simplifications might have to be made to finalise the design.

7.3 Pull-out tests

Trial pull-out tests are commonly carried out prior to, at the beginning of and during the nailing works. These give important insight into potential nail performance, but interpretation of the results is not straightforward. Where a reasonable specification for the working method and suitable site supervision are employed, the pull-out results should exceed the calculated values. This should be the case even when 'best estimate' values are used rather than 'safe' factored values. Where early tests shows pull-out results consistently and significantly higher than unfactored design values, the designer might consider increasing the design values. However, where they are lower the cause must be investigated.

The measured pull-out resistance, P_{mes}, on the schemes described in Appendices A to H is consistently higher than the calculated value, P_{calc}, based on factored drained soil strength, in accordance with HA 68. The mean value of the ratio P_{mes}/P_{calc} is about 4.6.

Pull-out test results on these schemes were best predicted using calculations based on the undrained shear strength and gave a mean value of the ratio P_{mes}/P_{calc} of about 1.9, using an adhesion factor of 0.45. While this suggests that the short-term test pull-out resistance may be better estimated using the undrained strength it does not necessarily mean that this is the case for long-term behaviour.

Recommendations for pull-out test procedures are given in the CEN Execution Standard for Soil Nailing currently being drafted.

7.4 Summary

There is currently no single detailed, comprehensive design method for soil nails. Care therefore needs to be exercised to ensure that a high level of geotechnical expertise is available to ensure that the advice and guidance published in various documents is used to produce a safe and economic design.

Many designers favour a simple approach of analysing the unreinforced slope and calculating the total nail force required to improve stability. This may be acceptable in straightforward situations, but on occasion, a more rigorous analysis may be needed. One such method is provided by HA 68, but many designers find it difficult to use and consider the resulting designs to be conservative.

The design of soil nailing is critically dependent on the quality of the site investigation data available. Selecting design soil strengths and porewater pressures is difficult, as is the prediction of corrosivity. The technique is unlikely to be suitable in soft soils, or where obstructions such as cobbles are present. With the present level of experience of soil nailing, it is recommended that nails are not used in situations where large cyclic or dynamic loads might apply.

Some deformation of a soil nailed slope is required to mobilise tension in the nails (above any small tensions developed during construction) and to reach a state of equilibrium. Soil nailed slopes are not appropriate, therefore, in situations where some movement of the slope cannot be tolerated during the service life of the earthwork. The nail installation angle has a significant and complex effect on the performance of a nailed slope. For ease of grouting and speedy generation of tension with soil movement it is suggested that a value of between 10º and 20º be chosen.

Trial pull-out tests are commonly carried out as part of the nailing works. These give important insight into potential nail performance, but interpretation of the results is not straightforward. Pull-out test results on the schemes studied in this report were best predicted using calculations based on the undrained shear strength and gave a mean value of the ratio P_{mes}/P_{calc} of about 1.9, using an adhesion factor of 0.45. While this suggests that the short-term test pull-out resistance may be better estimated using the undrained strength it does not necessarily mean that this is the case for long-term behaviour.

However, the relations derived between measured and calculated pull-out resistance may be used by the designer to check and adjust the design. Where early tests show pull-out results consistently and significantly higher than unfactored design values, the designer might consider increasing the design values. However, where they are lower the cause must be investigated.

Although the use of soil nailing has not been as widespread as anticipated, it has proved to be a good technique for the construction of new steep cuts or the strengthening of existing marginally stable slopes. Advantages include ease of construction, economic and environmental benefits. It is hoped that this report will provide additional guidance that will allow the technique to be applied more widely in the future.

8 References

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HA 43	Geotechnical considerations and techniques for widening highway earthworks (DMRB 4.1)
HA 68	Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques (DMRB 4.1)
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Appendix A: Scheme 1 – Temporary works

A.1 General requirements

An 8m high 1 in 1 cutting slope required stabilisation as temporary works for the construction of a box culvert. As shown in Figure A1, the slope supports the existing road and was formed in London Clay with the top 2m in the overlying Clay with Flints. Pre-existing shear surfaces in the London Clay were identified during the site investigation for the project.

Fired soil nails were used to provide stability to the slope for the duration of the temporary works with the design and installation carried out by a specialised contractor. The programmed design life of the temporary works was 4 months but because of problems elsewhere on the scheme it was, in practice, needed for 9 months and performed satisfactorily for this period.

A.2 Design parameters

In the original design both the Clay with Flints and London Clay were assumed to be fully drained with a pore pressure coefficient, $r_u = 0$. The medium term effective strength peak parameters for the Clay with Flints and London Clay were assumed to be $c' = 20$ kN/m^2 and $\phi' = 20^o$ for both materials. These cohesion values may be rather optimistic as they represent the unweathered material. Slope stability calculations based on the peak values show that the slope is stable.

A uniform highway surcharge of 10 kN/m^2 was incorporated as temporary loading due to construction traffic. This is consistent with Clause 5.8.2.1 of BD 37.

A.3 Design methodology

A rather unusual soil model was used to determine the required pull-out force. The model considers the stability of a soil wedge with a K_o earth pressure distribution acting on a vertical failure plane. This suggested a vertical slip plane and resulted in a fairly short length of nail in the resistant zone. The nails were installed at an inclination of 45^o to the horizontal. At this fairly steep angle, the nails would not have applied much (tension-derived) restoring force to a potential slip and the contribution to stability would have tended to come from some nail shear or dowel effect in the fairly stiff clay.

The original design by the specialised contractor did not provide a calculation for the pull-out resistance for the nails but merely stated that the available pull-out was 30kN per m width of slope. This was based on pull-out resistances measured on other clay slopes. Although this is not a rigorous method of determining design pull-out values, the performance of the nails was confirmed by 12 trial nails in the slope prior to the start of the works. In addition the specialised contractor did not give a guaranteed installed nail length but argued that should nails not penetrate the full design depth this would indicate a greater soil strength than assumed which would still provide a satisfactory factor of safety.

A.4 Design check

The design engineer for the highway scheme checked the design using parts of HA 68. The assumptions made in the check differed from the original design as follows.

1 The most likely failure plane was based partly on an observed slip adjacent to the nailed slope. This was about 1m deep and extended from the crest to about 4m down the slope.

2 The effective strength design parameters for the Clay with Flints and London Clay were $c' = 0$ kN/m^2 and $\phi' = 30^o$, and $c' = 20$ kN/m^2 and $\phi' = 20^o$.

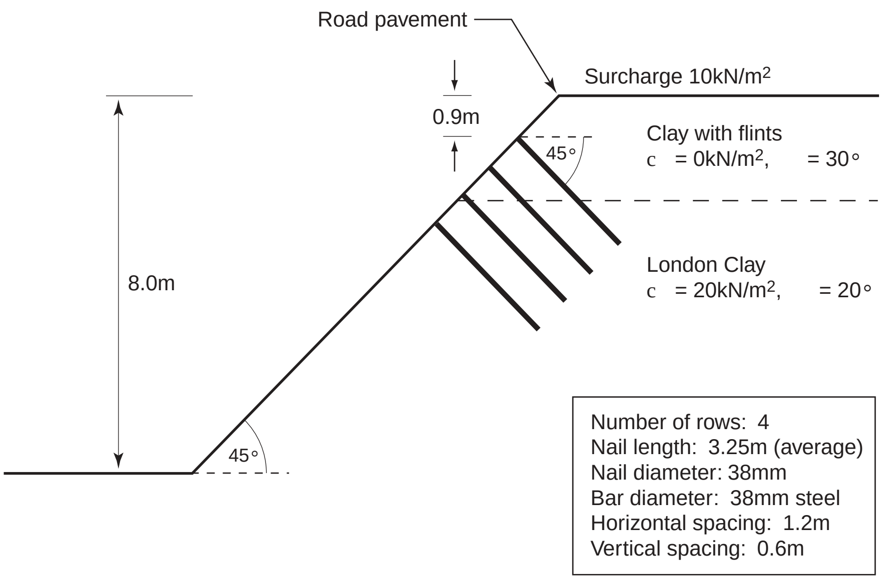


Figure A1 Scheme details

3 No factor of safety was applied to the soil parameters to derive design values in accordance with Clauses 2.13 and 2.14 of HA 68. Because of this, the pull-out resistance of each nail was about twice that obtained from using factored design values.

4 A uniform surcharge of 10 kN/m^2 plus an allowance for a 40 tonne vehicle on the supported road were used.

5 An r_u value of 0.5 and bulk density γ_b = 18.5 kN/m^3 were adopted to represent fully saturated ground but these were only used in checking the possibility of a shallow slip.

The design check derived an ultimate pull-out resistance of P_{calc} = 15.8 kN per nail, assuming:

Nail length in ground = 5 m

Nail effective length L_e behind failure plane = 4 m

Nail diameter = 0.038 m

Mean cover depth to nail =3.5 m

Interface sliding factor nail/soil, α =1

A.5 Pull-out tests

Twelve trial nails (SN1A - SN4C) with an average embedment depth of 3.5 m were installed by the sub-contractor adjacent to the works. They gave a measured mean pull-out value of P_{mes} = 43 kN, which was taken to be sufficient to validate the design. However, because no slip plane was identified in the original design, the length of nail providing useful pull-out resistance is not known. Thus the measured pull-out values cannot be directly compared to the design values.

Twenty five test nails (A4 to D13) were tested to failure by TRL when the temporary works were no longer required. The pull-out resistance of the nails was calculated essentially using HA 68 but using a variety of assumptions on soil strength. For example, for nail A5 the calculated values are shown below. For these tests the effective length, L_e , is the total nail length in the ground (3.34 m) and the average overburden was 1.96 m.

i P_{calc} (for ϕ'=20^o c$'$=20 kN/m^2) is 11.6 kN

ii P_{calc} (for ϕ'=20^o c$'$=0 kN/m^2) is 4.5 kN

iii P_{calc} (for ϕ'=0^o Cu=100 kN/m^2) is 36 kN

iv P max_{mes} is 37.9 kN

v P $residual_{mes}$ is 11.5 kN

Similar results for (i), (iv) and (v) for the other nails are shown in Table A1. Case (i) employs the same soil properties as used in the original design and for checking (for temporary works). Case (ii) uses the much lower critical state values which might be appropriate for the long term properties of London Clay. Case (iii) uses the undrained shear strength, Cu, typically 100 to 150 kN/m^2 for firm London Clay, as confirmed at this site by shear vane testing. The similarity between the measured values at (iv) and those at (iii) would suggest that the undrained shear strength, Cu, is the dominant factor in pull-out tests in cohesive soils.

Most of these nails were loaded in a standard ‘quick test’, taking typically one to two hours. Tests C6, C7, C8, C10 and C13 were loaded over typically three or four days with two increments per day being applied. The TRL equipment included a ‘bridge’ to react against the soil about 0.75 m away from the nail being tested while the sub-contractor’s equipment for the tests described earlier reacted against a plate around and close to the nail. However, there is little difference in the results from the different types of test on this site.

A lateral loading test was undertaken in which the head of a nail was loaded laterally with the reaction being shared by three or four other nails. These test results, presented in Table A2, show that much larger deflections are generated by a load applied laterally than by one applied axially. Generally the axial tests provided a maximum pull-out resistance of 30 to 40 kN at about 2 mm movement while the lateral tests generated about 10 to 15 kN resistance at deflections of 200 mm to 300 mm.

Table A1 Axial pull-out tests on selected nails

Nail number	P_{max} measured (kN)	$P_{residual}$ measured (kN)	Effective length L_e (m)	Mean cover depth (m)	P_{calc} [1] (kN)	P_{max}/L_e measured (kN/m)
A4	52.9	17	3.39	1.59	10.9	15.5
A5	39.9	11	3.34	1.96	11.6	11.3
A7	33.3	10	3.31	1.96	11.5	10.6
A9	38.9	11	3.34	2.02	11.8	12.0
A10	36.3	22	3.24	1.96	11.3	11.2
A11	30.0	21	3.24	1.96	11.3	9.3
B1	35.7	10	3.24	2.02	11.4	11.0
B2	38.1	28	3.53	1.97	12.3	10.8
B10	38.0	13	3.24	1.94	11.3	11.7
B11	38.5	16	3.24	1.94	11.3	11.9
C4	35.6	11	2.95	1.95	10.3	12.1
C6	40.0	22	3.31	2.14	11.9	12.1
C7	42.2	26	3.44	2.15	12.4	12.3
C8	32.6	18	3.07	2.25	11.3	10.6
C10	38.4	20	3.24	2.25	11.9	11.9
C11	31.8	-	3.24	2.25	11.9	9.8
C12	32.6	14	3.24	2.22	11.9	10.1
C13	44.8	24	3.24	2.25	11.9	13.8
D1	34.8	11	3.24	2.15	11.7	10.7
D2	27.4	11	3.24	2.29	12.0	8.5
D3	35.3	14	3.44	2.22	12.6	10.3
D5	25.8	11	3.24	2.28	12.0	8.0
D9	29.2	12	3.24	2.29	12.0	9.0
D12	32.3	12	3.24	2.3	12.0	10.1
D13	33.5	11	3.24	2.3	12.0	10.3
SN1A	49	23	3.5	1.96?	12.2	14.0
SN2A	42	24	3.65	1.96?	12.7	11.5
SN3A	43	22.5	3.25	1.96?	11.3	13.2
SN4A	42	-	3.6	1.96?	12.6	11.7
SN1B	43	23	3.7	1.96?	12.9	11.6
SN2B	41	22	3.2	1.96?	11.2	12.8
SN3B	42	21	3.65	1.96?	12.7	11.5
SN4B	39	22.5	3.75	1.96?	13.1	10.4
SN1C	43	22.5	3.5	2.18?	12.7	12.3
SN2C	43	26	3.5	2.18?	12.7	12.3
SN3C	43	23	3.5	2.18?	13.1	11.9
SN4C	47	25.5	3.65	2.18?	13.3	12.9

Nails A4 to D13 were tested by TRL, whilst nails SN1A to SN4C were tested by the sub-contractor.
[1] *Calculated using $c'=20$ kN/m^2, $\phi'=20^o$*

Table A2 Results of lateral load tests

Nail number	Max lateral load (kN)	Max deflection (mm)
A2	9.9	256
A8	11.1	191
A12	15.0	327
B5	12.2	211
B9	12.4	174
C6	12.2	108
D11	10.2	266

Appendix B: Scheme 2 – Steepened noise bund

B.1 General requirements

In order to widen an existing highway, fired soil nails were required over a 100 m length of an existing noise bund to create a 1 horizontal to 1.5 vertical (56°) steepened slope. The noise bund is 2.3 m in height above carriageway level, and 5.3 m above the level of the adjacent field and about 2m wide at its crest. A timber noise barrier 2 m in height was to be mounted on top of the bund. This was to be fixed to steel I-beam columns at 5 m centres which were founded within circular casings bored into the bund. Depending on the depth of these concrete-filled casings they could have a stabilising or de-stabilising effect on the soil bund but this was not covered in the design or check. The layout is shown in Figure B1.

B.2 Design parameters

The noise bund embankment is composed of compacted Bagshot Beds placed as part of the original road construction. It comprises medium dense silty clayey fine to medium sands with some gravel. The effective stress soil parameters quoted by the sub-contractor to be used in the design are summarised in Table B1. There is no indication in the scheme documents of whether these parameters are peak or critical state values and, hence, if they are factored as suggested in HA 68. The groundwater level was assumed to be below the base of the steepened slope. Nail lengths and spacing are not given within the scheme documents but it appears that the method uses the shear component in the nails as well as the tension.

Table B1 Soil design parameters

	Bulk density	*Soil strength*	
Stratum	*γ_b (kN/m^3)*	*c'_{peak} (kN/m^2)*	*ϕ'_{peak} (degrees)*
Alluvium	19	0	24
Bagshot Beds: clay	20	5	28
Bagshot Beds: sand	20	0	35

A value of soil/nail adhesion of 100 kN/m^2 was adopted by the sub-contractor for the purposes of design. This value was derived from back analysis of pull-out tests from previous jobs including Scheme 1 (see Appendix A). These tests, however, were performed on fired nails in a variety of soils. The validity of adopting an adhesion value derived from a different project in different ground conditions is somewhat questionable, although confirmatory pull-out tests were planned.

B.3 Design methodology

The original soil nail design was undertaken by the sub-contractor using the computer program, Talren. This program determines the stability of a soil structure with, or without, reinforcements and is based on Bishop's 'method of slices'. Talren calculates both shear and tensile forces in the soil nails but it is understood that only tensile forces were considered in the design check.

A site visit was undertaken in January 1995 during the installation of the soil nails. Approximately 30 m of noise

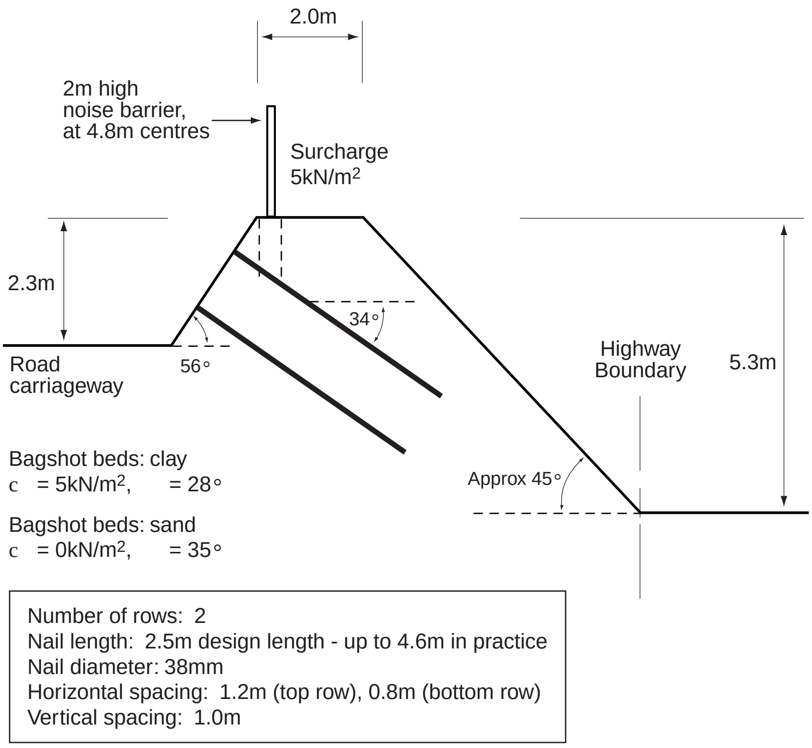


Figure B1 Scheme details

bund had been treated using some 59 nails arranged in two rows. Most of the nails were fired at angles of between 25° to 35° to the horizontal (approximately 90° to the slope) and at 1.2 m centres in the top row and 0.8 m centres at the bottom. This differs a little from the spacing given in the design check. Measurements of the exposed length of nails indicated that the nails had been installed to depths of up to 4.6 m.

Steel mesh reinforcement (200 mm by 100 mm by 8 mm diameter) and a geogrid had been laid over the treated cut slope and steel plates fixed to some nail heads to hold the surface layers in place and allow vegetation to become established. The slope of the noise bund had previously been over-excavated by the main contractor. To attain the correct slope profile, the as dug soil had been replaced and partially recompacted. Rainfall had caused slumping of this recompacted material, and localised failure of the surface material had formed gullies up to 250 mm deep. Slumping of material had caused bulging of the mesh and geogrid at or about carriageway level. At this point, the soil nailing operation was aborted and the slope stabilised using a geotextile reinforced soil technique.

B.4 Design check

The soil nail design by the sub-contractor was checked using the same computer program, Talren. The soil parameters used by the checker appear to be broadly similar to those used by the sub-contractor. It would seem that the values of the soil variables input into Talren were unfactored. The checker, however, factored the effective angle of friction of the Bagshot Beds to take account of an increase in porewater pressure equivalent to an r_u value of 0.05, i.e:

$$\phi'_{peak} = \tan^{-1} (\tan 30/1.05) = 28.8^{o}$$

The value assumed for r_u is small and this factoring approach is not generally employed when dealing with porewater pressure effects. The checker also increased the height of the slope by 1m to allow for future excavation of drainage trenches in front of the bund.

The requirements from the design performed by the checker were two rows of nails at 0.96 m centres for the upper row and 0.8 m centres for the lower row. All nails were assumed to be 2.5 m in length. It appears that this information is calculated or estimated separately and then input into the Talren program.

Specific comments on the program are:

1 The design soil parameters are not factored in accordance with HA 68.

2 A partial factor of safety of 2 is applied to the adhesion value of 100 kN/m^2: no other partial factors are applied to loads or materials.

3 There is no indication over what length of nail the adhesion applies.

4 A uniform surcharge loading of 5 kN/m^2 is applied to the crest and non-highway facing slope.

5 There is no indication of how the length, layout or inclination of the nails are computed.

B.5 Pull-out tests

Although provision was made for pull-out tests in the scheme documents, TRL understands that none were carried out because of the problems caused by the over-excavation of the slope.

Appendix C: Scheme 3 – Widened cutting for lay-bys

C.1 General requirements

This highway scheme included a proposal to install 18 additional lay-bys. At two locations, soil nailing techniques were employed to steepen existing cutting slopes to accommodate an additional 3.5 m wide area for the lay-bys. This was achieved by steepening the bottom of the slopes from the existing 27^o to 68^o over a vertical height of some 2.2 m. One cutting is in Keuper Marl and the other is a ‘false cutting’ where the road passes through former open cast coal workings backfilled with mudstone spoil.

A sub-contractor prepared the design for the slope stabilisation based on using bored and grouted glass reinforced plastic (GRP) soil nails. The layout is shown in Figure C1.

C.2 Design parameters

Effective stress soil parameters were adopted for the soil nail design. An angle of internal friction of $\phi'_{peak} = 30^o$ was used for both the Keuper Marl and mudstone spoil. No *in situ* or laboratory soil tests are available to verify this value, or indicate the degree of conservatism and its suitability for long-term design. A factor of safety of 1.4 has been applied to the friction angle: this is in accordance with BS 8081:1989, although HA 68 recommends a value of 1.5 on peak soil strength for permanent works. In this connection the factor of safety was applied to ϕ'_{peak} directly rather than to tan ϕ'_{peak} resulting in a slightly more conservative design value of 21.4^o being used instead of 22.4^o. No account was taken of the groundwater conditions or any likely changes in porewater pressure for long-term design.

C.3 Design methodology

The sub-contractor’s design is based on BS 8081:1989, Clause 6.2.2.2 for Type A anchorages. This assumes that pull-out capacity is dictated by the nail geometry and the transfer of stresses from the nail to the surrounding ground in shear at the soil/grout interface. Only tensile forces in the nail are considered for design: i.e. bending or shear resistance of the nails is neglected, as in HA 68. The design assumes a block failure mode (single wedge failure plane) which is generally in accordance with BS 8081:1989 Figure 51. However the angle of the failure plane was assumed without any justification of whether this represented the critical case in terms of instability and maximum out of balance force.

An ultimate bond strength for the soil/grout interface of 30 kN/m was assumed for design although laboratory pull-out tests by a university indicated a bond strength of only 21 kN/m. From BS 8081:1989 Appendix F2, the design value appears reasonable in view of the load factor of safety of 3 as used by the sub-contractor, i.e. a design bond strength of 10 kN/m. In addition a factor of safety of 1.5 was applied to the effective length of the nail. The overall effect of these partial factors is to provide a conservative design with an 8 m long single nail providing an ultimate pull-out capacity of 53 kN.

It was assumed, however, that the design pull-out resistance of a single nail was equivalent to the installation of two 5 m long nails. These shorter length nails were then adopted for design. This assumption of equivalent lengths is incorrect because the pull-out capacity of a nail relies on the length beyond the potential failure plane and the average

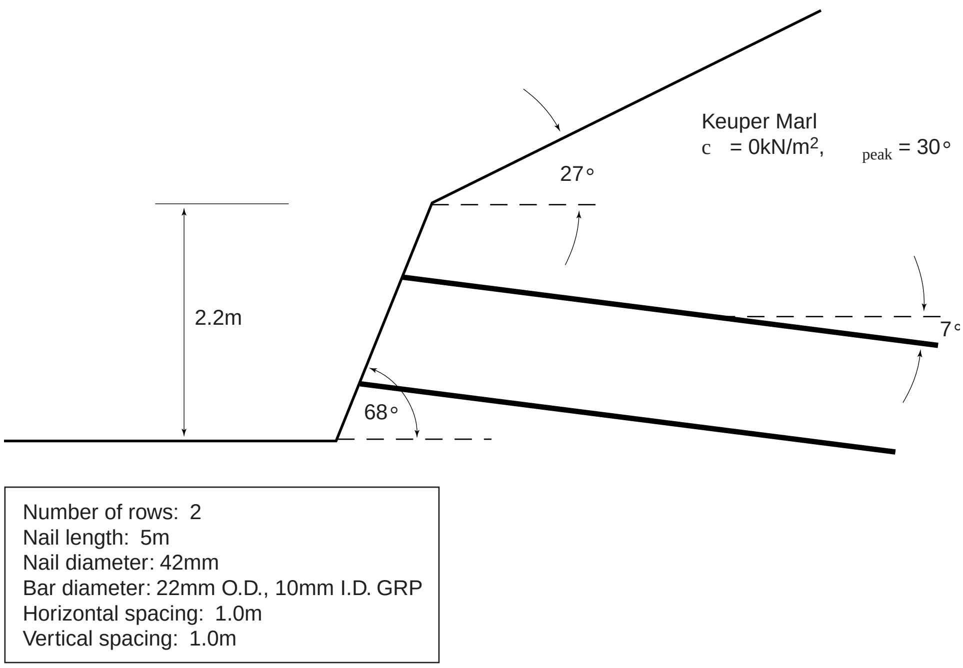


Figure C1 Scheme details

vertical effective stress along the nail length. The two 5 m long nails had a combined calculated pull-out capacity less than a single 8 m long nail. Furthermore, the design assumed that the full length of the nail would provide pull-out resistance as in the case of an anchorage designed in accordance with BS 8081:1989 where the anchorage prestress increases the shear strength of the soil such that the potential failure plane is beyond the fixed anchor length. This is not the case for soil nails where there is little or no prestress. This is also at variance with the design methodology in HA 68, which is based on the effective length of the nail behind the potential failure plane.

No calculations were given on the design of the soil nail layout. In this connection there appears to be no design for spreader plates at the end of the nails to ensure adequate load distribution against bearing capacity failure.

C.4 Design check

No formal design check was undertaken by the design engineer for the project. Instead the sub-contractor was asked to perform pull-out tests on six nails to confirm the 10 kN/m bond strength assumed in the design. A site meeting was held before the works commenced between the designer, client, sub-contractor and TRL where some improvements to the design were developed.

C.5 Pull-out tests

Short-term pull-out tests were undertaken on six trial nails. The maximum measured pull-out loads ranged between P_{mes} of 60 and 110 kN which exceeded the required design capacity of 53 kN per nail.

Appendix D: Scheme 4 – Slope strengthening

D.1 General requirements

Evidence of tension cracks in the carriageway of a highway gave rise to concern regarding the stability of the supporting embankment formed from recompacted London Clay. Slope stability analysis (considering circular slips) using critical state strength parameters identified two potential slip surfaces. These were a shallow failure surface, within the embankment (factor of safety = 0.538), which was considered to have caused the cracking and a deep seated rotational failure (factor of safety = 0.956), through the underlying *in situ* London Clay.

The brief from the client was for the factor of safety of the embankment to be increased to 1.3 over a minimum 40 year design life of the works. It was also a requirement that the vegetation on the embankment should not be disturbed and that there was unrestricted access for routine maintenance. Bored-and-grouted soil nails were installed to satisfy these requirements.

The design of the soil nails for the stabilisation of the embankment is given in a submission to the client from the design engineer.

D.2 Design parameters

The embankment comprises firm becoming stiff reworked and compacted London Clay overlying *in situ* firm becoming stiff London Clay. Probing through the embankment indicated the presence of soft or loose material.

Stabilisation of this embankment had been under consideration for some time and the client had gathered a large quantity of site investigation data and interpretative reports. This was made available to the designer and would have been of great value in making an informed decision on appropriate design parameters. Table D1 summarises the values of the soil variables used in the design.

An r_u value of 0.1 was used in the design: this seems reasonable given the measured groundwater levels. A uniform surcharge of 20 kN/m^2 was adopted to represent the highway live load beyond the crest of the slope (this is equivalent to 45 units of HB loading in accordance with BD 37 Clause 5.8.2.1).

D.3 Design methodology

There are two parts to the design:

1 Determining overall stability of the existing unreinforced slope and the out of balance forces.

2 Designing the soil nails to achieve a factor of safety of 1.3 on the restoring force/moment based on limit equilibrium principles.

From the design methodology, critical state strength parameters were adopted to assess the long-term slope stability of the embankment. This approach and the values used for the London Clay appear reasonable. The derived factor of safety against slope instability is quoted as 1.014 indicating that the slope is, at best, marginally stable.

The soil nail design, described by the designer, is based on BS 8081:1989 Clause 6.2.5.2 for fixed anchors in cohesive soils (Type A anchorages). This method is for tremie or gravity grouted straight shafted anchorages similar to those developed for bored piles, and is based on the use of undrained shear strength of the soil, Cu. Details of the layout are given in Figure D1.

There appears to be some inconsistency in the overall design approach, with mixed short and long-term soil strength values used for each part of the analysis. It would have been consistent to have used the same critical state soil strength values for both overall and internal stability. In addition the ultimate pull-out resistance of an individual nail was not calculated. This requires an assessment of the interface sliding force between the soil and grout. Instead the design assumed a nail length of 6.5 m and calculated the required ultimate resisting load for each nail (40.5 kN) from the geometry of the potential slip failure and equilibrium of forces (with the required factor of safety of 1.3). The required mean undrained shear strength of the grout/ground interface (Cu of 94.5 kN/m^2) was then derived. This undrained shear strength was then checked against the actual soil strength (Cu=75 to 150 kN/m^2) and considered to be adequate. Clearly, however, local soil strengths will be less than this mean value and will reduce the ultimate resisting force of some nails.

In accordance with HA 68, the soil adhesion should only be applied to the length of nail below the potential slip. The designer calculates the mean length of nail to be 3.4 m below the potential slip surface. It appears, therefore, that

Table D1 Soil design parameters

			Effective soil strength			
Stratum	Bulk density γ_b (kN/m^3)	Undrained cohesion Cu (kN/m^2)	c'_{peak} (c'_{res}) (kN/m^2)	ϕ'_{peak} (ϕ'_{res}) (degrees)	c'_{crit} (kN/m^2)	ϕ'_{crit} (degrees)
London Clay embankment	18	75 locally 25 - 50	5 (0)	23 (11)	0	21
London Clay *in situ*	20	75 - 150	25	28 (14 - 16)	0	23

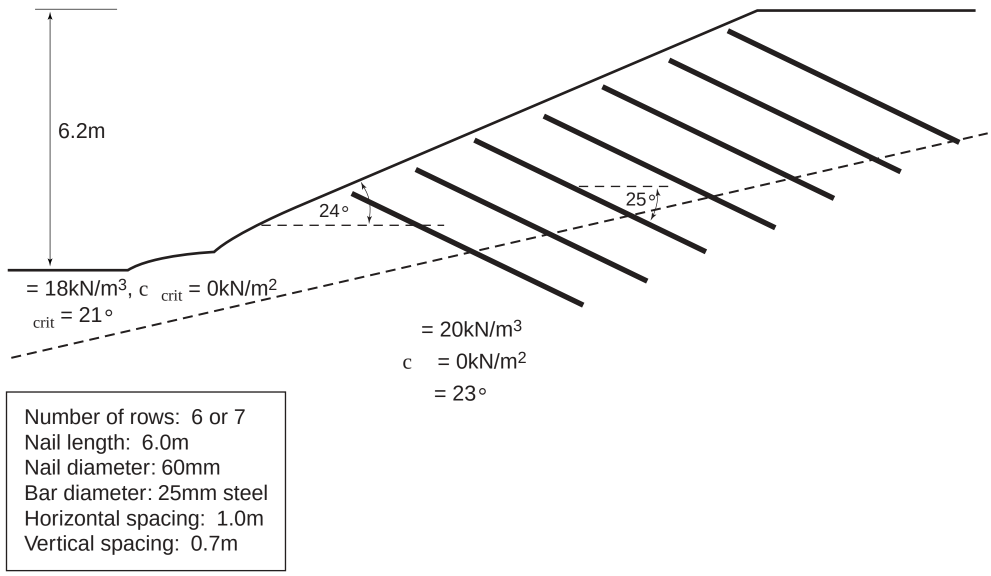


Figure D1 Scheme details

to provide sufficient adhesion the mean undrained strength of the clay over the final 3.4 m of nail should be about 191.5 kN/m^2. This is greater than the assumed design parameters for the intact London Clay suggesting that the nails have inadequate capacity for the length installed below the potential slip plane. Should activation of a slip occur it is uncertain whether the majority of nails will have adequate capacity to resist failure.

There was no design detail given for the facing plates to the soil nails. It is understood that this omission may have been a requirement of the client to allow unhindered maintenance to the grassed embankment. Facing plates are normally required to mobilise the resisting force in the nails. The only means of this occurring with this design is via mobilisation of the soil/grout bond above the potential slip plane. This contribution is usually ignored and has not been used elsewhere in the design calculations.

D.4 Design check

There was no separate calculated design check for this scheme but the client commented on the details of the design submission.

D.5 Pull-out tests

Six short term pull-out tests (T1 to T6) were undertaken during construction to validate the ultimate design load. The measured pull-out capacity of the nails is shown in Table D2. The calculated pull-out resistance has been determined using typical peak effective stress parameters applicable to the short-term strength of the material.

The results showed that the 6.5m long nails achieved an ultimate load capacity of some 105 kN. The design load is based on the proportion of the ultimate capacity of the full length of the nail carried by that length of the nail behind the slip plane i.e. 105 kN x 3.4 m/6.5 m = 54.9 kN which exceeds the required design ultimate load of 40.5 kN.

From a comparison of pull-out test results for 5.5 m and 6.5 m long nails, the designer stated that the load test results demonstrate that the final 1 m length of nail (5.5 m to 6.5 m) generates about 30 kN of the mobilised resisting force. For this to be correct, however, the undrained shear strength of the London Clay would have to be about 450 kN/m^2. This is unrealistic and suggests that the assumption of load distribution and nail bond is incorrect.

Table D2 Axial pull-out tests

Nail No	*P_{max} measured (kN)*	*Effective length L_e (m)*	*Mean cover depth (m)*	*P_{calc}[1] (kN)*	*P_{max}/L_e measured (kN/m)*	*P_{calc}[2] (kN)*
T1	105	6	3	25.1	15.5	59.4
T2	105	6	3	25.1	11.3	59.4
T3	55	5	3	20.9	10.6	49.5
T4	60	5	3	20.9	12.0	49.5
T5	115	6	3	25.1	11.2	59.4
T6	105	6	3	25.1	9.3	59.4

[1] *Calculated using c'_{peak}=5 kN/m^2,ϕ'_{peak}=23^o*

[2] *Calculated using Cu=150 kN/m^2*

As can be seen from Table D2 the measured pull-out resistance, P_{mes}, is between 2.6 and 4.6 times greater than the calculated pull-out resistance based on HA 68 methodology and using short-term peak strength parameters (c'_{peak}=5 kN/m^2, ϕ'_{peak}=23^o). For comparison an analysis of pull-out resistance has also been undertaken using undrained soil strength, Cu = 150 kN/m^2, which is considered a typical mean value from the ground investigation data. The pull-out resistance based on undrained strength, $P_{calc(u)}$, is presented in Table D2: the measured pull-out resistance is between 1.1 and 1.9 times greater than the calculated value. Although the undrained pull-out resistance is higher than that derived from an effective strength calculation it is still only approximately 50% of the measured pull-out resistance.

Appendix E: Scheme 5 – Slope strengthening

E.1 General requirements

An unstable slope along a railway cutting has been stabilised using bored-and-grouted soil nails. The section of cutting is some 400 m long and 4.8 m to 6.7 m high. It slopes at angles ranging between 18^o and 24^o. The brief from the client was for the factor of safety of the slope to be increased to a minimum of 1.3. It is understood that the required service life of the strengthened slope was 120 years rather than 60 years commonly quoted for highway earthworks. Existing vegetation on the slope was to be retained as far as possible to maintain stability of near surface soils. In addition, the remedial solution was intended to minimise the need to remove large quantities of spoil and minimise disruption to residential properties located just beyond the crest of the slope. The soil design and specification for the remedial works is given in a report prepared by the designer.

E.2 Design parameters

The findings of the ground investigations indicate that the cutting is in London Clay overlain by a thin mantle of topsoil/made ground. Relic shear surfaces are commonplace within the London Clay at depths of between 1 m and 2 m. Best estimate soil strength parameters adopted by the designer from the ground investigation data are shown in Table E1. Three zones of the London Clay are identified for the purposes of ground characterisation and the choice of design parameters.

Table E1 Soil design parameters

	Best estimate peak strength		*Design strength*	
Stratum	c'_{peak} *(kN/m²)*	ϕ'_{peak} *(degrees)*	c'_{des} *(kN/m²)*	ϕ'_{des} *(degrees)*
Weathered London Clay	14	20	1.5	20
Weathered London Clay - relic shear surfaces			0	13
Unweathered London Clay	18	24	5	24

As shown in Table E1, the design values were based on the minimum conceivable field values. This means that where relic shear surfaces exist in the upper surface of the London Clay, residual shear strength values are used, but critical state parameters c'_{crit} and ϕ'_{crit} are adopted for the remaining soil mass. This is generally in accordance with guidance given in HA 68. In this connection the Advice Note recommends the use of $c'_{crit} = 0$ whereas the designer has adopted less conservative values of 1.5 kN/m² and 5 kN/m² for the weathered and unweathered London Clay respectively. According to HA 68 Appendix I.4, relaxation of the general design method is allowed provided that site load trials are conducted to demonstrate that the design pull-out forces can be achieved for a derived nail length. Site trials were specified by the designer.

The long-term groundwater levels in Zones 2 and 3 were assessed, and an r_u value of 0.1 was assumed in design: this seems a reasonable value for the likely long-term condition. For Zone 1, an r_u value of 0.2 was assumed which again seems reasonable for the near surface material. These values of r_u imply that the infiltration of rainwater through the slope surface is more significant than the percolation of groundwater through the soil. This is particularly likely for shallower slopes and is consistent with porewater pressure measurements taken by TRL at other sites (Crabb and Hiller, 1993; Crabb, 1994).

E.3 Design methodology

The design methodology was based partly on HA 68. The following assumptions have been made.

1 The critical failure plane has been inferred from observation of previous instability and is believed to be at a depth of 2 m parallel to the cutting slope.

2 A limit equilibrium analysis was used. Design values for soil properties were factored in accordance with HA 68. No other factors are applied except ones to cover material characteristics, damage and corrosion effects in the nail.

3 The resisting force in the nail is assumed to be purely tensile; i.e. the effects of bending stiffness and shear are ignored.

Only an outline of the design and a summary of the soil nail specification is included in the design report. For this reason it is not possible to comment on the choice of design values nor the program used to undertake the analysis. Ultimate and serviceability limit states have been considered for all elements of the design in accordance with Draft BS 8006:1991 (current at the time of the design). Specific comments are:

a It is not clear whether the stability of the existing slope was initially assessed using the design values for the soil properties and what the factor of safety might have been.

b It is not clear what the overall stability of the stabilised slope would be. No calculations are offered and although it is stated that the overall stability of the slope is greater than 1.3, no information is provided on soil data, geometry or groundwater parameters used for this check.

In summary the design requires 8 m long 25 mm diameter steel nails at a horizontal and vertical spacing of 2 m, and 1.2 m to 1.6 m respectively. The nails are grouted into 200 mm diameter holes. According to the specification the grout should be a pumpable mortar with a compressive strength of 40 kN/m² at 28 days: this is likely to be a typographic error because the recommended grout strength in BS 8081: 1989 is a minimum of 40 MN/m². Each nail is tied to a 800 mm x 800 mm concrete spreader plate. Details of the layout are shown in Figure E1.

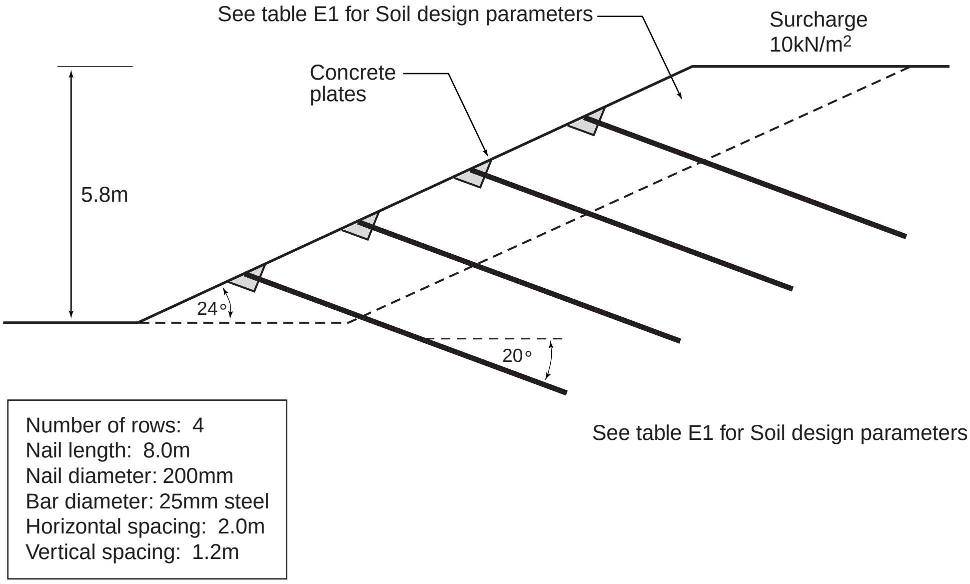


Figure E1 Scheme details

E.4 Design check

The designer appears to have undertaken an in-house design check for the scheme, i.e. no independent organisation was appointed by the client to undertake this function.

E.5 Pull-out tests

Eight short-term pull-out tests (Nos. 1 to 8) were carried out by the contractor as part of the works and the results are given in Table E2. It should be noted that the maximum pull-out capacity of the nails was not recorded in any of these tests because the tests were abandoned at some point due to excessive settlement of the reaction frame. The tests were stopped before grout to ground bond failure occurred but, referring to the results from the shorter nails, it seems reasonable to assume that the nails would have generated a mean pull-out resistance of around 200 to 220 kN.

A further nine short-term pull-out tests were undertaken by TRL on 2 m, 4 m and 6 m long nails. The results from these tests are given in Table E3.

The calculated pull-out resistance of the trial nails can be determined using the method in HA 68 but based on the total grouted nail length, L_t, and not the effective length, L_e, of the nail behind the potential failure plane.

E.5.1 Short-term pull-out tests

The rate of loading of the nails was approximately 50 kN per hour. It can be seen from Table E2 that the ratio, $P_{mes}/P_{calc}{}^1$, for the 8 m nails ranges from 1.5 to 3.1. The difference between the measured and $P_{calc}{}^1$ values is thought to be largely due to the fact that the design strength parameters adopted, i.e. $c' = 1.5$ kN/m^2 , $\phi' = 20^o$ and $r_u = 0.1$, were chosen to model long-term conditions and thus do not necessarily reflect the conditions current at the time of the test.

Pull-out resistances were also calculated using the best estimates of soil strength as given in the design report, i.e. $c' = 14$ kN/m^2 , $\phi' = 20^o$ and $r_u = 0.1$. These calculated pull-out strengths, $P_{calc}{}^2$, are also presented in Table E2. However, as shown in the table some of the values of P_{mes} were still nearly double the value of $P_{calc}{}^2$.

Table E2 Short-term axial pull-out tests

Nail No	*P_{mes} (kN)*	*Length L_e (m)*	*P_{mes}/L_e (kN/m)*	*Mean cover depth (m)*	*$P_{calc}{}^1$ (kN)*	*$P_{calc}{}^2$ (kN)*	*$P_{calc}{}^3$ (kN)*	*$P_{mes}/P_{calc}{}^1$*	*$P_{mes}/P_{calc}{}^2$*	*$P_{mes}/P_{calc}{}^3$*
1	192	8	24.0	3.25	78	127	226	2.5	1.5	0.8
2	120	8	15.0	3.25	78	127	226	1.5	0.9	0.5
3	168	8	21.0	3.25	78	127	226	2.2	1.3	0.7
4	240	8	30.0	3.25	78	127	226	3.1	1.9	1.1
5	168	8	21.0	3.25	78	127	226	2.2	1.3	0.7
6	216	8	27.0	3.25	78	127	226	2.8	1.7	1.0
7	216	8	27.0	3.25	78	127	226	2.8	1.7	1.0
8	168	8	21.0	3.25	78	127	226	2.2	1.3	0.7

[1] *Calculated using $c' = 1.5$ kN/m^2, $\phi' = 20^o$ and $r_u = 0.1$*
[2] *Calculated using $c' = 14$ kN/m^2, $\phi' = 20^o$ and $r_u = 0.1$*
[3] *Calculated using Cu = 100 kN/m^2*

Table E3 Results of short-term pull-out tests on 2, 4 and 6 m long nails

Nail No	P_{mes} *(kN)*	*Length* L_e *(m)*	P_{mes}/L_e *(kN/m)*	*Mean cover depth (m)*	P_{calc}^{1} *(kN)*	P_{calc}^{2} *(kN)*	P_{calc}^{3} *(kN)*	P_{mes}/P_{calc}^{1}	P_{mes}/P_{calc}^{2}	P_{mes}/P_{calc}^{3}
2.1	88	2	44.0	1.85	13	25	56	7.6	3.7	44.1
2.2	88	2	44.0	1.85	13	25	56	7.6	3.7	44.1
2.3	117	2	58.5	1.85	13	25	56	10.1	4.9	58.8
4.1	147	4	36.8	2.7	36	60	113	4.5	2.6	36.8
4.2	156	4	39.0	2.7	36	60	113	4.8	2.7	39.2
4.3	142	4	35.5	2.7	36	60	113	4.3	2.5	35.6
6.1	176	6	29.3	3.4	67	104	169	2.9	1.8	29.4
6.2	206	6	34.3	3.4	67	104	169	3.4	2.1	34.3
6.3	196	6	32.7	3.4	67	104	169	3.2	2.0	32.7

[1] Calculated using $c' = 1.5$ kN/m², $\phi' = 20^o$ and $r_u = 0.1$
[2] Calculated using $c' = 14$ kN/m², $\phi' = 20^o$ and $r_u = 0.1$
[3] Calculated using Cu = 100 kN/m²
[4] The designation 2.1 means a 2 m long nail, test number 1

The pull-out resistances were then calculated based on the approach commonly employed for calculating the available skin friction on piles. Following Tomlinson (1995) and Skempton (1959) for bored piles in London Clay an adhesion factor, $\alpha = 0.45$ was assumed. This gave a pull-out resistance, P_{calc}^{3}, of 226 kN which is closer to the mean P_{mes} of 186 kN and close to the estimated mean pull-out resistance of $P_{mes} = 220$ kN as stated above. This suggests that it might be possible to estimate the short-term pull-out resistance of the nails in a test on a particular site, but this would not necessarily aid the designer in predicting the long-term pull-out resistance of the nails.

E.5.2 Tests on 2m, 4m and 6m long nails

In all these tests, failure was generated at the grout to ground interface: the mean maximum pull-out resistance of the 2 m, 4 m and 6 m long nails were 98 kN, 148 kN and 193 kN respectively. The results and calculated resistances are presented in Table E3 and a summary given in Table E4. These pull-out results correspond to bond strengths of 48 kN/m, 37 kN/m and 32 kN/m. (The estimated value of 220 kN for the 8 m nails gives a bond strength of 28 kN/m). This is somewhat surprising because:

i) the longer the nails the greater the mean overburden acting on the nail (due to both the rising slope and the nail declination)

ii soil strength would usually be expected to increase with depth.

Possible explanations for the decreasing bond strength with increasing nail length include.

i Short-term pull-out resistance may be related primarily to the undrained cohesion of the soil, and so increasing overburden stress would not have any substantial effect.

ii The reaction frame applies loads to the soil surface and thereby increases the stresses within the upper layers of soil. These stresses reduce with increasing depth of cover and thus shorter nails would be more affected by them.

iii The near-surface soil could be looser and more fissured than it is at depth and the grouting process could generate more interlock and pull-out resistance near the surface.

Table E4 Summary of short-term pull-out tests

Nail length (m)	P_{mes} *(kN)*	P_{calc}^{1} *(kN)*	P_{calc}^{2} *(kN)*	P_{calc}^{3} *(kN)*	P_{mes}/P_{calc}^{1}	P_{mes}/P_{calc}^{2}	P_{mes}/P_{calc}^{3}
8	220	78	127	226	2.8	1.7	1.0
6	193	61	98	169	3.2	2.0	1.1
4	148	33	58	113	4.5	2.6	1.3
2	98	12	24	56	8.1	4.1	1.8

[1] Calculated using $c' = 1.5$ kN/m², $\phi' = 20^o$ and $r_u = 0.1$
[2] Calculated using $c' = 14$ kN/m², $\phi' = 20^o$ and $r_u = 0.1$
[3] Calculated using Cu = 100 kN/m²

iv When a small load is applied to the head, the upper length provides the necessary pull-out resistance but, because of elastic extension, there will be no relative movement between the lower part of the nail and the surrounding soil and thus the lower part will not provide any pull-out capacity. With increasing load, and relative movement between the nail and soil, a point will be reached when the peak strength of the upper part will be fully mobilised. Following this the soil weakens towards a critical state (and, perhaps, *in extremis* a residual) strength and pull-out resistance will be generated along an increasing length of nail: in effect pull-out failure will proceed progressively along a nail. This may mean that peak soil strength is generated over only a short length of nail, perhaps only one or two metres. Thus at failure a shorter nail could be generating the peak strength over a higher proportion of its length.

v The effect of 'constrained dilation' reduces with increasing overburden pressure.

As shown in Table E3 as nail length increases the calculated pull-out resistances more closely match the measured values. Furthermore the predictions based on the approach used for bored piles are in much better agreement with measured values - although this calculation, and the short-term test itself, may not give a good indication of long-term performance.

E.5.3 Long-term pull-out tests

A further three long-term tests on 6 m nails were undertaken over a period of 20 months. The results of these tests are summarised in Table E5. Purpose-built hydraulic jacks were used with a mechanical locking device to prevent substantial loss of load through leakage or temperature effects in the hydraulic system. In addition, a series of stiff springs were fitted between the jack and the nail head to minimise the reduction in load caused by nail pull-out. The jacking and movement measuring equipment was protected from the weather and vandals by a steel cabinet. The rate of loading for the long-term pull-out tests was approximately 10 kN per month over a 20 month period. On applying each increment of load the nail head would be displaced: during the following month the load would reduce, typically by 10 or 20 kN and the displacement could increase (due to nail pull-out) or decrease (due to the reaction frame pushing into the ground).

The nails failed by rupture of the nail thread: the failure loads were 225, 230 and 186 kN. This gives a mean rupture value of 214 kN. The mean short-term pull-out resistance of the three 6 m nails tested was 193 kN. The higher pull-outs measured in the long-term tests is again surprising because it would usually be assumed that in the short-term high values of undrained cohesion, Cu, would apply. In a long-term (20 month) test it would usually be expected that any porewater suctions generated by the application of the test load would be dissipated.

The measured pull-out resistance, P_{mes} of these nails is about two or three times the calculated pull-out values. For $c' = 1.5$ kN/m^2, $\phi' = 20^o$ and $r_u = 0.1$ the mean ratio of P_{mes}/P_{calc} is 3.5 and for $c' = 14$ kN/m^2, $\phi' = 20^o$ and $r_u = 0.1$ the mean ratio is about 2.2. The mean ratio of the measured and predicted loads based on the approach used for bored piles is 1.3.

Table E5 Results of long-term pull-out tests

Nail No	*P_{mes} (kN)*	*Length L_e (m)*	*P_{mes}/L_e (kN/m)*	*Mean cover depth (m)*	*P_{calc}^1 (kN)*	*P_{calc}^2 (kN)*	*P_{calc}^3 (kN)*	*P_{mes}/P_{calc}^1*	*P_{mes}/P_{calc}^2*	*P_{mes}/P_{calc}^3*
6.4	225	6	33.5	3.4	61	98	169	3.7	2.3	1.3
6.5	230	6	37.5	3.4	61	98	169	3.8	2.3	1.4
6.6	186	6	31.0	3.4	61	98	169	3.0	1.9	1.1
Mean	214	6	35.7	3.4	61	98	169	3.5	1.9	1.3

[1] *Calculated using $c' = 1.5$ kN/m^2, $\phi' = 20^o$ and $r_u = 0.1$*
[2] *Calculated using $c' = 14$ kN/m^2, f¢ = 20^o and $r_u = 0.1$*
[3] *Calculated using Cu = 100 kN/m^2*

Appendix F: Scheme 6 – Motorway widening

F.1 General requirements

The slope of the existing cuttings on this scheme was between 20^o and 25^o and a steepened face at 68^o with a typical vertical height of 2 m was required to accommodate carriageway widening over a distance of some 1.5 km

The soil nails used for the contract were Ischebeck Titan 30/16 self-boring nails which consist of a 30 mm hollow steel bar (minimum yield stress 470 N/mm^2) with a drill bit welded to the tip. The design comprised two rows of nails at 1.5 m horizontal spacing. Two drill hole diameters were used, 75 mm for the top nails and 42 mm for the lower. While the nail is being installed, grout is pumped through the hollow steel bar. A galvanised steel plate was attached to the end of the nail to help support the face. Typical plate sizes of 400 mm x 400 mm for the top nails and 250 mm x 250 mm for the bottom nail were used. A facing cage, 300 mm thick, covers the excavated slope. The facing was constructed of galvanised steel grids and meshes: a geomesh was provided on the inside face to retain the soil and allow the application of seeding or planting.

F.2 Design parameters

The cuttings comprise firm to stiff slightly sandy clay with some gravel. The following effective strength values were selected for design; c'_{peak} = 2 kN/m^2, ϕ'_{peak} = 33^o. The value of c'_{peak} appears reasonable, but the value of ϕ'_{peak} appears high given that the measured values ranged between 22^o and 33.5^o, with a mean of 26.25^o. Some pockets/horizons of sand were found within the Glacial Till and these were assigned effective strength values of; c'_{peak} = 0 kN/m^2, ϕ'_{peak} = 35^o. During the installation of the nails along the northbound carriageway, a 1 m to 1.5 m thick band of sand was encountered. This, in combination with the presence of land/counterfort drains at 20 m intervals, supported the view that a high water table could exist at this horizon after periods of heavy rainfall. Notwithstanding this, the porewater pressure parameter r_u was taken to be zero; this would appear to be rather optimistic for the long-term condition of some parts of the slope.

There appears to have been no allowance for surcharge loading on the slope, such as maintenance plant.

F.3 Design methodology

The design of the soil nails is given in the approval documents dated 1995 by the designer to the client. The designer applied the method given in HA 68 and used the program ReActiv to calculate the length and distribution of the nails. This resulted in the design comprising two rows of nails at 1.5 m horizontal centres and the top nails being 6 m long and the lower nails 2 m long. Details of the layout are shown in Figure F1.

TRL understand that the client had instructed the designer to assume the existing slope (and thus the shallow, upper slope after widening) to be stable. Using the soil parameters taken by the designer and applying the partial factors of safety on soil strength recommended in HA 68 the upper slope is considered unstable.

F.4 Design check

The Engineer for the whole project ensured that the design work was undertaken in accordance with all appropriate procedures set down by HA 68. After development of the design by the soil nail designer, the design parameters were agreed with the Engineer and submitted to the client

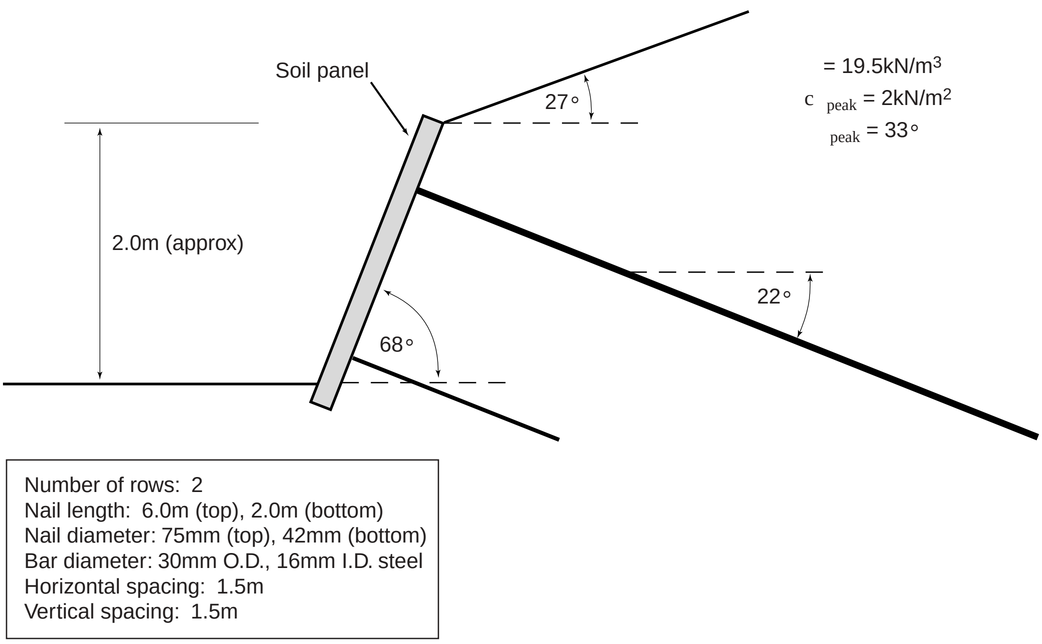


Figure F1 Scheme details

for comment. General comments from the client were incorporated into the final design including a soil nail test programme to validate the design assumptions with regard to design pull-out capacity and factor of safety.

F.5 Pull-out tests

Trial nails have been undertaken both by the contractor as part of the works and TRL. The tests and findings are described below.

F.5.1 Contractor's tests

Soil nail pull-out testing was undertaken by two specialised testing houses for the soil nail designer based upon a method statement provided by the Engineer. Nails for testing were installed a minimum of seven days prior to testing. The grout strength of the tested nails was a minimum of 40 N/mm^2 after 7 days. Details of the test frame and loading regime are presented in a feedback report. In summary four types of test were undertaken as follows.

1. *Working load test - fast rate of loading*. Load of up to 150% of nominal axial working load applied at 2.5 kN per minute.
2. *Working load test - slow rate of loading*. As above but at a rate of 2.5 kN per 5 minutes.
3. *Tests to failure - fast rate of loading*. Load applied in increments of 2.5 kN per minute. Failure defined as no increase in load with continued extension.
4. *Tests to failure - very slow rate of loading (creep test)*. Load increments of 5 kN (6 m long nail) and 2.5 kN (2 m long nail). Nail movement monitored every 15 minutes until movement is less than 0.005 mm per minute for two consecutive intervals.

The testing regime is shown in Table F1. In total 23 tests were undertaken (13 working load tests and 10 failure load tests). The Engineer states that in Tests 1 to 5 the nails were erroneously loaded to 150% of the design pull-out load rather than 150% of the nominal load. This effectively resulted in test forces applied being 195% of the nominal working load leading to failure of Test Nail 2. However it is uncertain whether this is actually an error since the Engineer also refers to the working load test specification being 150% of the design load in the feedback report. Because of this, three additional soil nails were installed adjacent to Test Nail 2. The results show that under the specified test load of 25.5 kN (1.5 x working load) the nail performance was satisfactory resulting in a very small displacement of 0.36 mm.

Table F1 Testing regime

Carriageway	*Test type*	*Load type*	*Nail No*
Southbound	Working load	1	1 - 5
	Working load	2	6 - 7
	Failure load	3	8 - 9
	Failure load	4	10 - 11
Northbound	Working load	2	17 - 23
	Failure load	4	12 - 16

Failure load tests on Test Nails 8 and 9 were carried out using a fast rate of loading (test type 3) and as such were thought to represent undrained pull-out tests and were not used for validation of the design. In all cases the measured pull-out forces exceeded the calculated design pull-out resistances.

Table F2 Results of contractor's pull-out tests to failure

Nail No	*P_{mes} (kN)*	*Length L_e (m)*	*Hole diameter (mm)*	*P_{mes}/L_e (kN/m)*	*Mean cover depth (m)*
2	25.0	4.6	75	5.4	1.96
8	47.7	1.7	42	28.1	0.71
9	147.7	4.7	75	31.4	2.14
10	115.0	4.8	75	24.0	2.14
11	25.0	1.6	42	15.6	0.71
12	145.0	4.0	75	36.2	1.78
13	65.0	3.1	42	21.0	1.25
14	170.0	4.5	75	37.8	1.96
15	180.0	4.8	75	37.5	2.14
16	190.0	5.5	75	34.5	2.31

The pull-out resistance for the 42 mm diameter nails varied from 15.6 kN/m^2 to 28.1 kN/m^2 with a mean value of 25.1kN/m^2. For the 75 mm diameter nails the pull-out resistance varied from 5.4kN/m^2 to 37.8 kN/m^2 with a mean value of 29.51kN/m^2. The variable results are thought to be due to the rather mixed soil conditions, especially pockets of sand within the clay.

F.5.2 TRL pull-out tests

A further four pull-out tests (TRL.1 to TRL.4) were undertaken by TRL on 4 m long nails. The results from these tests are given in Table F3. The ground conditions in the area of the test nails appeared to vary from those of the main scheme and comprised very stiff clay to weak mudstone. The nails were inserted into a 75 mm diameter hole and grouted over a 3 m length from the base. The remaining 1 m length of nail near the surface was sleeved in a plastic tube that passed through a 150 mm diameter hole in a 500 mm by 500 mm steel facing plate. The plate was pinned and grouted at the surface and acted as a reaction frame to the applied test load. This arrangement is thought to have surcharged the immediate area of the nail and this, combined with the local high stiffness soil, resulted in particularly high measured pull-out forces, P_{mes}.

Table F3 TRL pull-out tests

Nail No	*P_{mes} (kN)*	*Length L_e (m)*	*P_{mes}/L_e (kN/m)*	*Mean cover depth (m)*	*$P_{calc}{}^1$ (kN)*	*$P_{calc}{}^2$ (kN)*	*$P_{mes}/P_{calc}{}^1$*	*$P_{mes}/P_{calc}{}^2$*
TRL.1	273	3	91	1.78	15.3	111	17.9	2.5
TRL.2	181	3	60	1.78	15.3	111	11.9	1.6
TRL.3	272	3	91	1.78	15.3	111	17.8	2.4
TRL.4	176	3	59	1.78	15.3	111	11.5	1.6

[1] *Calculated using $c' = 5$ kN/m^2, $\phi' = 35^o$ and $r_u = 0$*
[2] *Calculated using Cu = 350 kN/m^2*

Test nails TRL.1 and TRL.3 were short-term tests completed in one working day. Long-term incremental load tests were performed on nails TRL.2 and TRL.4 over a period of 6 and 8 months respectively. These latter tests were undertaken to try to evaluate the effects of drainage and creep on pull-out strength. Test nail TRL.2 failed after 6 months at approximately 170 kN during incremental loading. The coupling (between the grouted and free length) appeared to have ruptured. In the case of nail TRL.4 failure occurred at the grout to ground interface.

The pull-out resistance P_{calc}^{1} can be calculated using the method in HA 68 and based on the total grouted nail length, L_e = 3m not the effective length of nail behind the potential failure plane. A calculated pull-out resistance of 15.3 kN is estimated based on peak drained strength parameters for the mudstone of $c' = 5$ kN/m^2 and $\phi' = 35^o$ and a pore pressure coefficient, $r_u = 0$.

For comparison the pull-out resistance has also been calculated using undrained parameters following the practice for the design of skin friction on piles, see for example Tomlinson (1995), Skempton (1959). Taking a mean Cu = 350 kN/m^2 for the mudstone and assuming an adhesion, α of 0.45 the undrained pull-out resistance, P_{calc}^{2} of a 3 m trial nail was calculated to be 111 kN.

The maximum measured pull-out forces, P_{mes}, are presented in Table F3. It can be seen that the ratio, P_{mes}/P_{calc}^{1}, for the 3 m nails is approximately 18 for the short-term tests and about 12 for the long-term tests. This suggests some reduction in pull-out resistance under long-term loading, either due to creep or softening of the ground. However given the premature coupling failure of nail TRL.2, the uncertain effects of surcharge loading of the nail from the test load reaction facing plate, and the fact that the test nails were in mudstone rather than the sandy clay (with assumed design properties of $c' = 5$ kN/m^2 and $\phi' = 35^o$), the true pull-out resistance of these nails is uncertain.

Appendix G: Scheme 7 – Motorway widening

G.1 General requirements

Due to constraints on land take, two cuttings on this scheme have been widened by forming a 60^o slope and employing soil nails to maintain its stability. The steepened slopes are typically 0.5 m to 1.5 m in height and have been reinforced with up to two rows of nails each at a horizontal spacing of 1.8 m (Site 1) or 1.5 m (Site 2) and 1 m vertical spacing. The nails are fixed to a steel cage. This cage is filled with topsoil and then vegetated with an appropriate grass mixture. The client for the works appointed a Design, Build, Finance and Operate (DBFO) contractor, who appointed a design consultant. The soil nail design and installation was undertaken by a specialised sub-contractor. The client also appointed a checking engineer for the scheme.

Trial pull-out tests have been undertaken by a testing house for the soil nail sub-contractor. Five tests were undertaken at Site 1 and four at Site 2. In addition, TRL installed six trial nails at the eastern end of the soil nailing works at Site 2.

G.2 Design soil parameters

Detailed information on ground conditions is given in the ground investigation report prepared by the checking engineer. The ground conditions are as follows.

Site 1

In general the sequence comprises glacial deposits overlying London Clay which in turn overlies Reading Beds. The glacial deposits generally comprise non-cohesive soils. The level of the groundwater table is not expected to be above the level of the carriageway. However, elevated porewater pressures are likely within the uppermost 2.5 m of any exposed section of London Clay.

Site 2

In this cutting, the ground comprises glacial deposits overlying Reading Beds which in turn overlie the Upper Chalk. On the basis of the investigation, groundwater was expected to be encountered in the Reading Beds and so a water table about 1.0 m below the base of the glacial deposits or 1.5 m above carriageway level was adopted for design. The soil properties used for design are summarised in Table G1.

G.3 Design methodology

The design of the soil nails was carried out using the Talren computer program. No surcharge loading appears to have been included in the design. Nail lengths and spacings were calculated such that an overall design factor of safety against instability of 1.3 was attained. Determination of the mobilised forces was undertaken with the assumption of axial tension only: i.e. any contribution from shear or bending of the nail was neglected.

The soil/nail adhesion was calculated from the equation:

$$q_s = 0.8\, K_l \tan\phi'_{peak}\, \Delta\sigma'_v$$

where $K_l = ½(1 + K_o)$

$\Delta\sigma'_v$ = vertical effective overburden pressure acting over the effective length, L_e, of the nail.

A partial factor of 1.3 was applied to q_s. The results of pull-out tests on trial nails were used to confirm the design.

The design required two rows of nails, installed at an angle of 20^o to the horizontal. The nails were spaced at 1.8 m horizontally at Site 1 and 1.5 m horizontally at Site 2. At both locations the nails were installed at a vertical spacing of about 1 m. The nail lengths are shown in Table G2 and appear to be longer than one might expect from intuition or engineering judgement. The nail diameter was assumed to be 132 mm (actual diameter of working nails 140 mm). Details of the layout are shown in Figure G1. Specific comments on the design are:

1 The design soil parameters are not factored in accordance with HA 68.

2 There is no indication over what length of nail the adhesion applies.

3 A partial factor of safety of 1.3 is applied to the 'adhesion value' (q_s): no other partial factors are applied to loads or materials, except a partial material factor of 1.15 on the tensile strength of the nail.

4 No surcharge loading has been allowed on the slope, such as for maintenance plant etc.

Table G1 Soil design values

Location	*Material type*		*Bulk density* γ_{bulk} *(kN/m³)*	*Cohesion* c'_{peak} *(kN/m²)*	*Effective angle of friction* ϕ'_{peak} *(degrees)*	*Coefficient of earth pressure at rest* K_o
Site 1	Glacial deposits		19	0	35	-
	London Clay	(0 - 2m)	20	1	20	-
		(>2m)	20	1.5	20	1.5
Site 2	Glacial deposits		19.6	0	37	-
	Reading Beds (clay)	(0 - 2m)	20.7	1	21	1.5
		(>2m)	20.7	5	21	1.5

Table G2 Design nail lengths

Location	*Top nail (m)*	*Bottom nail (m)*
Site 1	9 - 12	6
Site 2	12 - 13	8

G.4 Pull-out tests

G.4.1 Contractor's trial nails

Nine trial test nails were installed by the sub-contractor and tested by a testing house. The pull-out tests comprised three load cycles: unloading was through a minimum of three decrements to the fully unloaded state. If pull-out had not been reached by the third cycle, loading was continued in increments of 10% of the elastic limit of the nail up to failure. This meant that most of the tests did not measure the ultimate grout to ground pull-out resistance. The results of the tests are summarised in Table G3. At Site 1 the measured pull-out forces varied from 13 kN/m to 23.1 kN/m with a mean of 17.5 kN/m. At Site 2 the measured pull-out forces varied from 15.4 kN/m to 21.4 kN/m with a mean of 16.9 kN/m.

Also given in Table G3 are the calculated pull-out values, P_{calc}^{1}, based on the design effective strength parameters, $c' = 5$ kN/m^2, $\phi' = 21°$ and $r_u = 0$. In all cases the measured pull-out forces exceed the calculated pull-out resistances with respect to the grouted effective length of the nail, L_e, although for nails 3 and 5 (Site 1) the measured pull-out is approximately equal to P_{calc}^{1}. The maximum pull-out forces are, however, uncertain because

Table G3 Results of contractor's pull-out tests

Nail No	P_{mes} *(kN)*	*Length* L_e *(m)*	P_{mes}/L_e *(kN/m)*	P_{calc}^{1} *(kN)*	P_{calc}^{2} *(kN)*	P_{mes}/P_{calc}^{1}	P_{mes}/P_{calc}^{2}
Site 1							
1	130	6.0	21.7	43.6	197.9	3.0	0.6
2	115	5.0	23.1	31.6	164.9	3.6	0.7
3	156	12.0	13.0	152.8	395.8	1.0	0.4
4	155	9.0	17.3	89.9	296.9	1.7	0.5
5	151	12.0	12.6	152.8	395.8	1.0	0.4
Site 2							
1	200	13.0	15.4	177.4	428.8	1.13	0.5
2	201	13.0	15.5	177.4	428.8	1.14	0.5
3	149	7.0	21.4	57.1	230.9	2.62	0.6
4	200	13.0	15.4	177.4	428.8	1.13	0.5

1 Calculated using $c' = 5$ kN/m^2, $\phi' = 21°$ and $r_u = 0$
2 Calculated using Cu = 150 kN/m^2

the tests were terminated at a specified loads or deflections rather than at pull-out failure of the nail. For this reason P_{mes} is an under estimate of the pull-out capacity of the nail. It can be seen from Table G3 that the ratio, P_{mes}/P_{calc}^{1}, for the nails ranges from 1.0 to 3.0. Apart from most tests stopping before a true failure occurred, the difference between the measured and P_{calc}^{1} values is partly due to the fact that the design values, i.e. $c' = 5$ kN/m^2, $\phi' = 21°$ and $r_u = 0$, do not represent the conditions within the clay at the time of the test.

The pull-out resistance of the nail has also been calculated based on the approach normally employed for

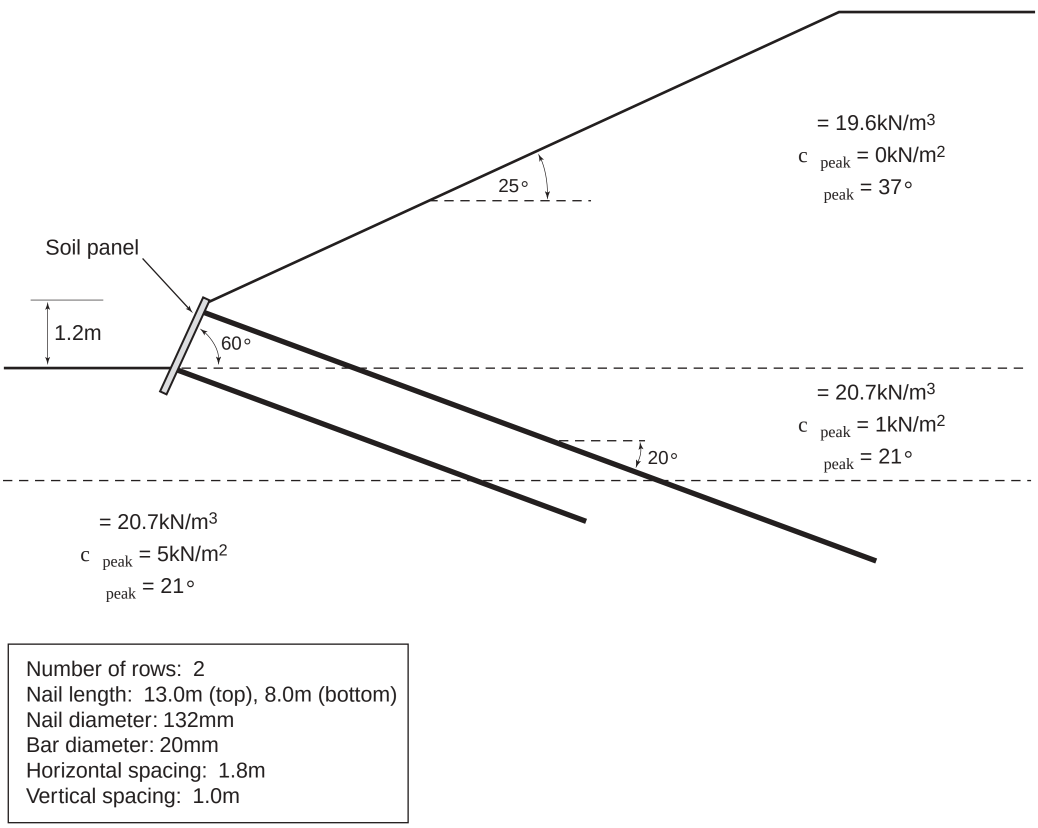


Figure G1 Scheme details for Site 2

calculating the available skin friction on piles by assuming an undrained strength of the Reading Beds clay of Cu = 150 kN/m^2. From Tomlinson (1995) and Skempton (1959) for bored piles in London Clay an adhesion factor, α = 0.45 was assumed. This gave a ratio $P_{mes}/P_{calc}{}^2$, of between 0.4 and 0.7 which indicates a closer agreement to the measured pull-out resistance.

G.4.2 TRL test nails

Six trial nails were installed by TRL just beyond the eastern end of the soil nail works at Site 2. To obtain maximum value from the tests it was important to obtain a failure at the grout to ground bond. Concern was expressed by the contractor about the possibility of a test nail rupturing and the head of the nail flying into the live carriageway. Thus a very conservative approach was adopted using a maximum grouted nail length of 3 m and using 28 mm diameter Macalloy 500 bar for the nails. It was agreed that the tests would be terminated well before the minimum quoted yield load of 308 kN.

Six nails were installed in 140 mm nominal diameter holes, i.e. the same size as used for the permanent works nails. Two had a nominal grouted length of 1 m, two of 2 m and two of 3 m. A borehole packer and bleed tube system was employed to keep the top 1 m length ungrouted but this was not a complete success since the grout flowed around the packer into the upper part of the borehole. Most of the excess grout was bailed out down to the packer but this problem resulted in a horizontal surface to the grout rather than the neat finish which the packer was intended to achieve. Several measurements were taken around each nail from the face of the cut to the grout surface to determine the mean grouted nail length.

Loads were applied to the nails using a hollow hydraulic cylinder seated on a reaction frame. The timber supports to the frame were located about 1 m away from the nail head. This, combined with the 1 m ungrouted length of nail should have minimised any additional head loads acting on the nail from the test itself. The pull-out force was calculated from the measured hydraulic pressure; a vibrating wire load cell was also fitted to act as a check. A dial gauge was mounted on a bar which was itself supported some distance from both the nail and the support timbers of the reaction frame to try to ensure a stable datum. An arrangement was employed by which a 'true' reading of movement was obtained even if the nail moved sideways under load. Loads were applied in increments of 100, 200 and 250 psi for the 1 m, 2 m and 3 m nails respectively: these corresponded to increments of 3.2, 6.4 and 8.0 kN. Increments of load were applied at two minute intervals.

A summary of the results of the test is given in Table G4. The measured grouted lengths of the test nails are included to permit the calculation of pull-out resistance per metre of nail. For example for nail 1.2, a maximum pull-out resistance, P_{mes} of 45 kN was obtained at 3.3 mm measured movement. A 'residual' pull-out resistance, P_{res}, of about 43 kN was observed as the nail was pulled about 25 mm from the ground. Some of the other test nails were pulled further from the ground and at 100 mm to 150 mm of movement the pull-out resistance dropped to about 90% of the peak. The pull-out forces per unit length of nail were fairly consistent and ranged from 30kN/m to 38 kN/m with a mean of 35.5 kN/m.

Table G4 Results of TRL pull-out tests

*Nail No**	*P_{mes} (kN)*	*Deflection at P_{mes} (mm)*	*Length L_e (m)*	*P_{mes}/L_e (kN/m)*	*$P_{calc}{}^1$ (kN)*	*$P_{calc}{}^2$ (kN)*	*$P_{mes}/P_{calc}{}^1$*	*$P_{mes}/P_{calc}{}^2$*
1.1	59	2.3	1.55	38	11.4	46.0	5.2	1.3
2.1	80	6.2	2.3	35	15.1	68.3	5.3	1.2
3.1	121	18.1	3.4	36	19.6	100.9	6.2	1.2
3.2	128	21.6	3.4	38	19.6	100.9	6.5	1.3
2.2	70	6.4	2.3	30	15.1	68.3	4.6	1.0
1.2	45	3.3	1.25	36	9.2	37.1	4.9	1.2

** 2.1 means a nominal 2 m grouted length, Test No 1*

1 Calculated using c′ = 5 kN/m^2, ϕ' = 21^o and r_u = 0

2 Calculated using Cu = 150 kN/m^2

Also presented in Table G4 are the calculated pull-out values, $P_{calc}{}^1$, based on the design effective strength parameters, c′ = 5 kN/m^2, ϕ' = 21^o and r_u = 0. Again in all cases, the measured pull-out forces exceed the calculated pull-out resistances with respect to the grouted length of the nail. The ratio, $P_{mes}/P_{calc}{}^1$, for the nails ranged from 4.6 to 6.5. The difference between the measured and $P_{calc}{}^1$ values is thought to be largely due to the fact that the design values adopted, i.e. c′ = 5 kN/m^2 , ϕ' = 21^o and r_u = 0, do not represent the conditions within the clay at the time of the test.

The pull-out resistance of the nails has also been calculated based on the approach normally employed for calculating the available skin friction on piles assuming an undrained strength of the Reading Beds clay of Cu = 150 kN/m^2. From Tomlinson (1995) and Skempton (1959), for bored piles in London Clay an adhesion factor, α = 0.45 was assumed. This gave a ratio $P_{mes}/P_{calc}{}^2$, of between 1.0 and 1.3. As observed on other sites this would imply that it might be possible to estimate the short-term pull-out resistance of a test nail using the undrained strength of the soil.

Appendix H: Scheme 8 – Steepened slope for new slip road

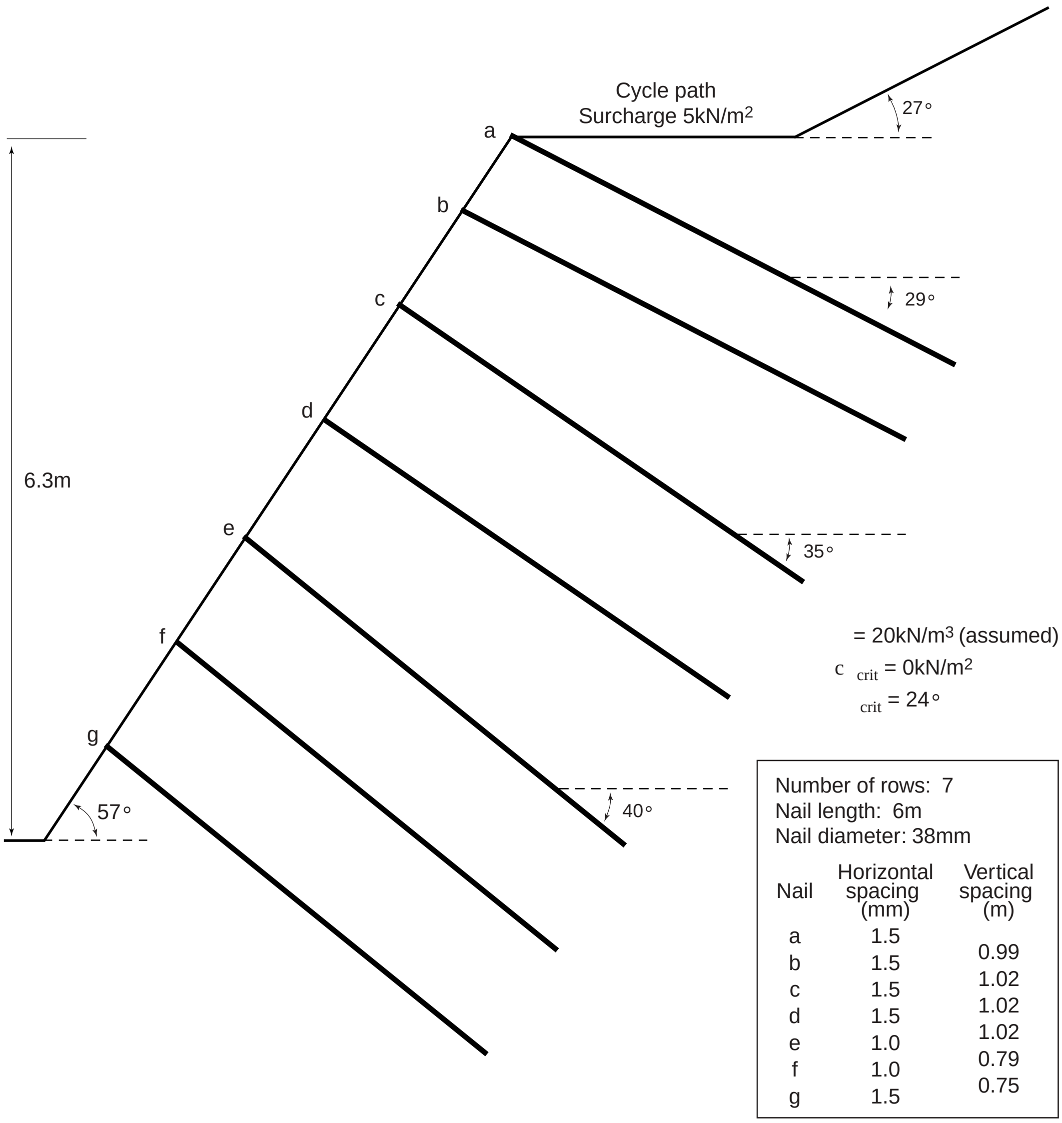
H.1 General requirements

A section of highway was upgraded as a Design, Build, Finance and Operate Contract. As part of the works a grade-separated interchange was constructed. The slip road leading onto the south-bound carriageway required the construction of a cycleway at a higher level parallel with the slip road. To create the area required for a cycleway, a steepened slope was built above the slip road. The slope varied from 22° to 57° and ranged in height from 5.6 m to 6.3 m. The slope was strengthened using 6 m long 38 mm diameter steel nails inserted directly into the soil. The nail layout at the 6.3m section of slope is shown in Figure H1.

Percussively installed nails were used in the slope with the design and installation carried out by a specialised sub-contractor for the DBFO organisation. Two designers oversaw and checked the design, which was also checked externally with assistance from TRL.

H.2 Design parameters

Details of the properties of the soil are given in the design report. The soil was Oxford Clay with a typical plasticity index of 35%. The document recommended that c'_{des} should be taken as 0 kN/m² and $\phi'_{des} = \phi'_{cv} = \phi'_{crit} = 24^o$ (based on the correlation between PI and ϕ'_{crit} given in BS 8002:1994). While the design submission mentions a porewater parameter value of $r_u = 0.1$, the sub-contractor's documents made available to TRL did not contain a design analysis which took account of any positive porewater pressures. The pull-out resistance of the soil nails was not determined using an effective stress calculation to HA 68, but the design pull-out resistance was based on the results of pull-out tests.



Nail	Horizontal spacing (mm)	Vertical spacing (m)
a	1.5	0.99
b	1.5	1.02
c	1.5	1.02
d	1.5	1.02
e	1.0	0.79
f	1.0	0.75
g	1.5	

Figure H1 Scheme details

H.3 Design methodology

The sub-contractor's design was based primarily on the Talren computer program which uses a circular slip analysis after Bishop. The use of this program tends to result in nails being installed over a range of rather steeper angles compared with the shallower, constant angle more usually seen (see Figure H1). The design nail layout was based on fifteen pull-out tests. These gave a mean peak pull-out resistance of 127 kN/m^2 (of nail surface area) and a mean 'residual' pull-out resistance of 81 kN/m^2. One standard deviation of the measured 'residual' pull-out was subtracted from the mean to derive a pull-out value of 65 kN/m^2. A partial factor of 1.5 was then applied to this value to give a P_{des} of 43 kN/m^2 or 4.6 kN per metre length of nail (assuming a post-corrosion diameter of 34 mm).

Using the effective stress approach given in HA 68, TRL calculated the pull-out resistance for the top and bottom nails where the height of the slope was greatest (6.3 m) of 1.9 kN/m and 12.2 kN/m respectively (assuming zero porewater pressure).

The design pull-out resistance derived from the tests is within the range calculated using an effective stress approach. However, there were a number of minor problems related to deriving the design pull-out resistances from the tests. The location of the pull-out tests and the local ground conditions were not clearly defined, and so it was not possible to confirm that the test area was representative of the site as a whole. For some tests (possibly only those during construction) the sub-contractor found it difficult to apply a 'perfectly' axial load during nail testing and it is thought that the test results were slightly higher than if a truly axial load had been applied. In the sub-contractor's design, there are no calculations regarding reduced slope stability or reduced pull-out resistance due to positive porewater pressures which could be generated during the 60 year design life of the slope.

H.4 Design check

The design was checked by two members of the DBFO consortium. They also assumed a circular slip but did not use the Talren program. Also, the check employed peak soil strengths $\phi'_{peak} = 25^o$ and $c'_{peak} = 5$ kN/m^2 factored down in accordance with BS 8006:1995. The circular slip was checked for stability using an r_u value of 0.1. The nail pull-out resistance was determined using the same method as employed by the sub-contractor and no comparison was made with the HA 68 effective stress method.

The external checker requested additional pull-out tests to be carried out during construction to increase confidence in the design values obtained through the initial tests together with the installation of piezometers to monitor groundwater levels.

H.5 Pull-out tests

Table H1 gives the data obtained from the preliminary sub-contractor tests. Table H2 gives the data obtained from the tests carried out as the slope was constructed. As shown, the mean residual pull-out force of 104 kN/m^2 is some 25% greater than that obtained in the preliminary tests. It is thought that the higher values were due to the fact that the nails were installed after bulk excavation and so the soil around the nails was stronger than for the earlier tests where nails were installed through a weathered surface.

Table H1 Preliminary tests used to assess the design pull-out resistance

Age at test (days)	*Peak unit skin friction (kN/m^2)*	*Residual unit skin friction (kN/m^2)*
7	113.3	97.9
7	108.3	100.9
7	149.0	105.9
7	107.9	56.6
7	134.4	69.8
7	148.2	91.0
7	120.1	69.8
7	150.1	99.4
7	137.4	78.7
7	148.5	105.5
7	105.1	77.0
7	121.4	77.8
7	124.1	49.9
7	92.2	70.5
7	148.2	76.7

Table H2 Later tests used to confirm the design pull-out resistance

Age at test (days)	*Peak unit skin friction (kN/m^2)*	*Residual unit skin friction (kN/m^2)*
15	161.7	112.1
8	184.2	129.5
7	146.0	97.3
8	114.5	80.8
7	134.1	98.8
7	157.7	121.9
7	114.7	78.9
3	186.8	129.3
3	76.6	48.8
6	153.2	111.4
9	191.4	126.4
29	137.8	89.9
6	168.5	132.6
34	136.8	96.7
10	153.4	113.4
14	131.6	106.7
14	119.6	89.9
15	118.5	94.8

Table H3 gives the results of pull-out tests undertaken by TRL on additional nails installed by the sub-contractor. The mean load per square metre of surface area of the nails was 85 kN, close to the values obtained by the sub-contractor in the preliminary tests. However, this good agreement may be fortuitous as the nominal 2 m long nails had a pull-out resistance of 60 kN/m^2, the 4 m nails of 93 kN/m^2 and the 6m nails of 111 kN/m^2: these values correspond to 7.3, 11 and 13 kN per metre length respectively. The contractor's peak pull-out results on nominal 6m nails were 81 kN/m^2 in the preliminary tests and 104 kN/m^2 in the later tests compared with the TRL

Table H3 Short term pull-out tests by TRL on percussive nails

Test number	*Length (nominal/ actual) (m)*	*Failure load (kN)*	*Load per metre length (actual) (kN)*	*Load per square metre of surface area of nail (kN/m^2)*
2.1	2/1.8	13	7	62
2.2	2/1.8	16	9	73
2.3	2/1.8	10	6	46
4.1	4/3.9	51	13	110
4.2	4/3.9	44	11	95
4.3	4/3.9	36	9	76
6.1	5.5/5.4	70	13	111
6.2	5.5/5.4	70	13	111

peak pull-out of 111kN/m^2 for 6 m nails

A second specialised sub-contractor installed six drilled and grouted nails adjacent to the percussively installed nails. These consisted of a spirally ribbed hollow steel bar of 30 mm external diameter grouted into a 125 mm augered borehole. Because of site constraints, the drilled and grouted nails had significantly less overburden than the percussive nails described above. The results of five short term tests undertaken on the drilled and grouted nails are given in Table H4. The mean load per square metre of surface area at 36 kN is significantly lower than the 85 kN derived for the percussive nails. This may be partly because of the lower overburden on the drilled and grouted nails. A more important factor may be that the percussively installed nails displace the soil and generate an enhanced normal force in the soil which increases the frictional pull-out. Also, grouting will tend to generate a better bond in granular soils where borehole roughness and grout ingress into the soil will aid pull-out resistance. In clays, grouting is likely to provide relatively low pull-out resistance. Unlike the percussive nails, the different lengths of the grouted nails all generated similar pull-out resistances per unit area. The nominal 2 m nails had a pull-out resistance of 31 kN/m^2, the 4 m nails 34 kN/m^2 and the 6 m nails 31 kN/m^2. These values correspond to 12, 13 and 12 kN per metre length respectively.

Table H4 Results of short term pull-out tests on drilled and grouted nails

Test number	*Length (nominal/ actual) (m)*	*Failure load (kN)*	*Load per metre length (actual) (kN)*	*Load per square metre of surface area of nail (kN/m^2)*
2.1	2/1.9	23	12	31
2.2	2/1.9	23	12	31
4.1	4/3.9	51	13	34
4.2	4/3.9	51	13	34
6.1	5.5/5.4	66	12	31

A long term pull-out test was also undertaken by TRL on a 6 m long percussive nail and a 6 m drilled and grouted nail. These tests were similar to those carried out at Scheme 5 to evaluate the effects of drainage and creep on pull-out strengths (see Appendix E). Similar equipment to that used at Scheme 5 was employed and load increments were applied to the nails at monthly intervals until the nails failed after eight months for the drilled and grouted nail and ten months for the percussive nail. The results are shown in Table H5.

The percussive nail failed at a peak load of 60.5 kN (which is equivalent to 93.8 kN/m^2 or 11.2 kN/m): this compares with a peak load of 70 kN (111 kN/m^2 or 13 kN/m) derived from the quick tests. The drilled and grouted nail failed at a peak load of 59 kN (25.9 kN/m^2 or 10.2 kN/m): this compares with a peak load of 66 kN (31 kN/m^2 or 12 kN/m) derived from the quick tests.

Table H5 Results of long term pull-out tests

Test number	*Length (nominal/ actual) (m)*	*Failure load (kN)*	*Load per metre length (actual) (kN)*	*Load per square metre of surface area of nail (kN/m^2)*
Percussive	5.5/5.4	60.5	11.2	93.8
Drilled and grouted	5.5/5.4	59	10.2	25.9

Appendix I: Summary of other schemes

Scheme 9 – Culvert (temporary works)

Slope

This job involved the temporary support of a vertical face 8.5 m in height. The soil was described as black boulder clay with a c' of 10 kN/m^2 and a ϕ' of 30º to 35º. The design was carried out using the Snail and Goldnail computer packages to provide a factor of safety of 1.3.

Nail details

Four hundred and sixty drilled and grouted nails were used. They were made from 25 mm or 28 mm grade 500 threaded steel bar and were 6 m to 8 m in length. The nails were grouted into 114 mm boreholes inclined at 15º to the horizontal. The horizontal spacing was 1.25 m to 2 m and the vertical spacing 1.5 m. 400 mm x 400 mm facing plates were used over metallic reinforced geogrid 'MacMat R'.

Notes

Pull-out tests were carried out on six sacrificial nails. These were tested to 80 per cent of the yield strength of the steel bar without pull-out failure occurring.

Scheme 10 – Bridge abutment (temporary works)

Slope

A 70º slope 6.2 m high required stabilising as temporary works. The soil consisted of 1 to 3 m loose silty sand and soft clayey sand overlying firm sandy clay and silty fine sands. The water table was at 4 m to 5 m depth and well points were installed. A maximum pore pressure of 20 kN/m^2 was allowed for in design. The design was carried out using the Goldnail package and the effective stress method in HA 68. A factor of safety of 1.3 was required for the final condition, and one of 1.2 during construction.

Nail details

Self drilling Ischebeck Titan nails 30 mm OD and 16 mm ID were employed in a 76 mm borehole. A total of 376, six metre long nails was installed. The nails were inclined at 20º at a 1 m horizontal and vertical spacing. The facing was 100 mm sprayed concrete reinforced with a single A142 mesh fabric and the facing plates were 200 mm x 200 mm x 12 mm mild steel.

Notes

Pull-out tests were carried out. Two 5 m long sacrificial nails were subjected to 3 cycles of loading to 3 times working load or pull-out failure. Tests indicated a capacity of 3 to 5 times the design load.

Scheme 11 – Culvert (temporary works)

Slope

This job required the temporary support of an 80º slope, which was up to 5.3 m in height. The soil consisted of firm silty clays and silty sands with a ϕ' of 24° and a c' of 5 kN/m^2. The design approach used parts of Goldnail, FHWA (1996), HA 68 and conventional pile design. The design assumed a 20 kN/m^2 surcharge at the top of the slope and required a factor of safety of 1.25 for the 2 year design life.

Nail details

Five hundred drilled and grouted nails of either 8m or 11 m length were used. They were constructed of 25 mm diameter Gewi bar and installed in either 114 mm or 168 mm boreholes. The nails were installed at an angle of 12.5° and at 1.1 m horizontal spacing and 1.25 m vertical spacing. The facing consisted of 150 mm thick sprayed concrete reinforced with a single A252 mesh and 225 mm x 225 mm x15 mm steel plates.

Scheme 12 – Cliff stabilisation

Slope

A 6 m to 12 m cliff face needed stabilising. The 55° slope was made of silty sandy clay with angular rock clasts overlying igneous rock. The soil properties were a c' of 10 kN/m^2 and a ϕ' of 25°. The Talren design package was used to provide a factor of safety of 1.5

Nail details

The nails employed on this scheme were 25 mm Dywidag bars with double corrosion protection grouted into 110 mm holes. The nails were 6 m long and installed at an angle of 15° at a 2 m horizontal and a 1.5 m vertical spacing. The facing employed Lotrak 16/15 and Tensar mat 200 held down with 500 mm x 500 mm x 20 mm or 300 mm x 300 mm x 20 mm galvanised steel plates. Metal items were further protected with zinc based epoxy paint or bitumastic paint.

Notes

Tests were carried out with pull-out resistances of 483, 484 and 273 kN/m^2 of nail surface area recorded. Note some portion of the test nails were in rock as well as soil. Further information is available in Ground Engineering, December 1999, p12.

Scheme 13 – embankment strengthening

Slope

An 11.5 m high embankment with a slope angle of 62° required strengthening. The soil consisted of compacted fill overlying boulder clay and the soil properties were taken as c' of 0 to 3 kN/m^2 and ϕ' of 21° to 25°. The Snail design package was used to provide a factor of safety of 1.4.

Nail details

One hundred and fourteen drilled and grouted nails were used. They consisted of 32 mm MAC 500 bar with 80 mm diameter corrugated pvc-u corrosion protection grouted into 120 mm pre-bored holes. The nails were 12.25 m in length and were installed at an angle of 30° at a horizontal spacing of 1.86 m and a vertical spacing of 4.5 m to 6.5 m. An 800 mm x 800 mm x 12 mm galvanised steel plate was used.

Notes

Further information is available in Martin J (1997). The design and installation of soil nail slope stabilisation schemes using 'Snail'. Ground Improvement Geosystems. Thomas Telford, London.

Scheme 14 – Underbridge (temporary works)

Slope

A 75° temporary slope 10.5 m high was required in a poorly compacted embankment with minimal site investigation data available. The design was carried out using the Snail package to give a factor of safety of 1.5 for the three month design life of the structure. The dynamic loading effects from passing trains was thought to be an important factor in the performance of the nailed structure.

Nail details

One hundred and forty seven drilled and grouted nails were used. They were made from 25 mm Gewi reinforcing bar grouted into 100 mm diameter boreholes. The nails were 8 m in length and were installed at a 20° angle on a 1 m by 1 m triangular grid. The facing consisted of a fine plastic matting covered by a stronger geogrid held in place by 450 mm x 450 mm steel plates.

Notes

Pull-out tests were carried out on two nails. The grout to ground bond strength was recorded at 48 and 58 kN/m^2. Further information is available in Martin J (1997). The design and installation of soil nail slope stabilisation schemes using 'Snail'. Ground Improvement Geosystems. Thomas Telford, London.

Scheme 15 – Slope strengthening

Slope

Parts of an 85° weathered mudstone cut face needed stabilising. A nailed solution was developed using the Talren design package

Nail details

One hundred and eighty ballistic nails were used to stabilise two sections of slope. The 38 mm diameter mild steel nails were installed to a depth of 4.5 m. They were angled at 15° to the horizontal and placed on a 1 m by 1 m regular grid. The facing comprised a 200 mm layer of single-sized crushed rock held in place with two galvanised steel grids. The steel grids were sandwiched between a steel plate welded to the nail and an aluminium plate locked above.

Notes

Further details are given in Hall G J (1995). The use of ballistic soil nailing and reinforced soil in Huddersfield. The Practice of Soil Reinforcing in Europe. Thomas Telford, London.

Scheme 16 – Railway embankment strengthening

Slope

A clay fill embankment up to 8 m high and with side slopes of 20° was showing signs of distress. A solution was developed using drilled and grouted nails.

Nail details

The nails were made of galvanised Gewi-steel from Dywidag and were 6 m to 11 m long. The facing consisted of a plastic-coated, galvanised steel mesh held down with face plates. The surface was top-soiled and seeded on completion.

Notes

Further information is given in Ground Engineering, June 2000, p13.

Scheme 17 – Steepened slope for new slip road

Slope

An existing cutting slope required steepening to a maximum angle of 50° and a maximum height of 6 m. The slope consisted of Kimmeridge clay with a c' of zero, a ϕ' of 25° and a Cu of 100kN/m^2. The design used a combination of BS 8006, ReActiv and Talren (only tensile forces were considered).

Nail details

The drilled and grouted nails were installed over a 300 m length of cutting. They were made from 32 mm steel bar and were 6 m, 8 m or 10 m in length. The nails were installed in 100 mm diameter boreholes drilled at 20° to the horizontal. The facing consisted of a plastic mesh containing a seeded coir fibre held down by a green-painted, galvanised steel plate 25 mm x 25 mm x 10 mm.

Notes

Two pull-out tests were carried out by TRL, one on a slightly bent nail. The failure loads were 106 kN and 116 kN(bent). The residual loads were 88 kN for both nails.

Scheme 18 – Cliff stabilisation

Slope

A vegetated cliff face up to 12m high and with a slope angle of 45° to 60° was showing signs of distress. The New Red Sandstone was weathered to a depth of several metres and some 140 m of the cliff needed stabilising.

Nail details

Three hundred and thirty three drilled and grouted nails were employed on the scheme. They ranged in length from

4.5 m to 9.5 m and were installed at an angle of 15°. The facing consisted of geogrid and stiff mortar blocks.

Notes

Further information is available in Ground Engineering, September 1996.

Scheme 19 – Slope stabilisation

Slope

A major slope stabilising project was required as part of a road widening scheme. The 60° slope was up to 14 m high and was made of cemented wind blown sands.

Nail details

One thousand two hundred simultaneous drilled and grouted nails were installed using the Dywidag MA1 hollow bar system. The nails were typically 6 m to 7 m long and the facing consisted of Tensar SS40 over a geotextile underblanket.

Notes

Further information is given in Ground Engineering, February 2000, p27.

Scheme 20 – Railway embankment strengthening

Slope

Weak embankments on a line had a history of slip failures and deterioration. The soil type was consolidated ballast and ash overlying medium firm clay. Drilled and grouted nails were used to stabilise the slopes.

Nail details

More than 2,500 Ischebeck self drilling nails were used. The nails were 8 m to 12 m long and were installed in 75 mm holes at angles up to 60°.

Notes

Nails were test loaded to 170 kN without failure (double the original design requirement). Further information is given in Ground Engineering, February 1999, p21.

Scheme 21 – Railway embankment stabilisation

Slope

A 50 m length of embankment needed repairs. The embankment, which was made of fill and ballast overlying clay and Sherwood Sandstone, was 10.5 m high with 38° sideslopes. The facing consisted of mesh and face plates.

Nail details

One hundred and forty eight drilled and grouted nails were used. They were made from 16 mm to 36 mm galvanised Gewi bar from Dywidag. Nail lengths were 9 m to 15.5 m and they were installed at a horizontal and vertical spacing of 1.75 m.

Notes

Further information is given in Ground Engineering, February 1999, p15.

Scheme 22 – Cliff stabilisation

Slope

Cliff erosion had led to a number of landslides. The slope angle was up to 60° and the soil consisted of head deposits over Green Ammonite beds and calcareous Belemite beds. Laboratory ring shear values varied between 19° and 26°.

Nail details

The nails employed McCalls galvanised, threaded bar and lengths from 7.5 m to 16.7 m were used at different locations. They were grouted into 100 mm boreholes at various spacings. The facing consisted of a geotextile and Armater (honeycomb cells) followed by topsoil and hydroseeding.

Notes

Further information is given in Ground Engineering, February 1999, p18.

Scheme 23 – Cutting slope

Slope

This job required the construction of a cutting 250 m long and up to 10 m in height. The nails were installed mainly into glacial sands and gravels and glacio-lacustrine silt and clay. The design was carried out using BS8006 and the Talren computer package.

Nail details

Some 1100 percussively installed nails were used on this scheme. The nails were 5 m lengths of 38 mm diameter mild steel bar. They were installed at 45° on a staggered grid with 1.5 m horizontal spacing and 0.5 m vertical spacing.

Scheme 24 – Steepened cutting

Slope

A cutting in mixed glacial deposits was required for a road improvement. The cutting slope was about 60 m long, up to 16 m high with a face angle of 45°. The glacial till comprised sand, gravel, cobbles and occasional boulders in a silt and clay matrix overlying sandstone. Part of the slope contained ‘running’ silty sand deposits between 2 m and 6 m in thickness. The design method was based on HA 68.

Nail details

Three hundred and forty drilled and grouted nails, generally 8 m to 14 m in length were employed. They were made from 25 mm galvanised Dywidag threadbar grouted within a pvc sheath grouted into a 140 mm borehole. They were installed on a 1.5 m staggered grid and inclined at 10° to the horizontal. The facing consisted of a plastic-coated galvanised steel mesh over a coir ‘soil blanket’, which was subsequently hydro-seeded.

Notes

Tests were carried out on nails fully in the glacial till and also on nails part way in the underlying sandstone. All the nails comfortably exceeded the design requirements. Further information is available in Barley A D *et al.* (1997). The use of soil nails for the stabilisation of a new cutting for the realignment of the A4059 at Letty Turner Bends. Ground Improvement Geosystems. Thomas Telford, London.

Scheme 25 – Strengthening to minimise differental movement of a road pavement in a landslip area

A road pavement required frequent repair and maintenance as it was built on a slope subject to occasional landslip movements. The road was built on a thick layer of industrial waste including mining spoil, foundry waste and old tiles overlying earlier landslip material. The design philosophy was to try to tie the road foundation into a single coherent mass to minimise differential movement of the road structure. It was accepted that it would not be possible to prevent the slope as a whole from moving. A trial using 123 ballistic nails was carried out after which a 500 m length of the highway was stabilised.

Nail details

The nails were used in conjunction with geotextiles to help hold the road foundation and road pavement together. Eight hundred and thirty five nails were fired into the road foundation. They were 38 mm diameter mild steel bar up to 6 m in length. Immediately above them a reinforced soil foundation was built up followed by the road structure. This consisted of a high strength geotextile at the bottom of the sub-base, a geogrid within the sub-base and a glass grid within the asphalt.

Notes

Further information is given in Ground Engineering, March 1996, Reinforced Soil Supplement, page xviii.

Scheme 26 – Steepened cutting slope

Slope

A new cutting slope with a face angle of 40° and up to 20 m in height was required for a new highway scheme. The soil consisted of variable glacial tills and shattered bedrock.

Nail details

Drilled and grouted nails up to 12 m in length and inclined at 10° were installed on a 3 m by 3 m grid. A geogrid was fixed across the slope to ensure surface stability, followed by topsoil and a retention mat and seeded to provide a green finish.

Notes

Further details are given in Ground Engineering, February 1996, p22.

Scheme 27 – Steepened slopes for a new development

Slope

A slope some 14 m in height required strengthening. For aesthetic and technical reasons a tiered structure was employed with steep upper and lower slopes connected by a shallower, 34° intermediate slope. The slopes were cut into an old slag heap composed of granular slag, clinker, sand, general construction material fill and debris.

Nail details

A total of 1,100 drilled and grouted nails was installed in the three slopes. The upper slope was faced with the contractor’s proprietary ‘Soil Panel’ and the shallow intermediate slope with the company’s ‘Recultex’ system and ‘Greenfix’ matting. The lower slope used a reinforced concrete face subsequently ‘clad’ with the contractor’s Permacrib wall system.

Notes

Further information is given in Ground Engineering, February 1999, p17.

Scheme 28 – Strengthened embankment slope

Slope

A railway embankment constructed of silty clay had failed and a rapid repair and strengthening solution was needed. Nails were installed into weak mudstone beneath the embankment and terminated at the face with gabions held back with bearing plates.

Nail details

Dywidag hollow nails were installed using a simultaneous drill and grout method. Grout was injected along the entire length of the borehole as the nail was drilled in. Two rows of gabions were used to support the slope, each supported by the nails.

Notes

Further information is given in Ground Engineering, May 2001, p11.

Scheme 29 – Strengthened embankment slope

Slope

An embankment approximately 9 m high with a slope angle in the order of 30º needed stabilising. The soil properties on site were a combination of ballast overlying ash and clay. The Slope W design package was used to provide a factor of safety of 1.3.

Nail details

Approximately 1,990 Ischebeck Titan 30/16 injection soil nails were employed, with lengths ranging from 7 m to 11 m. The angle of installation was 25º and the nails were installed at 1.5 m centres both vertically and horizontally. The facing employed on the surface of the slope was a

‘MacMat R’ reinforced geogrid, held in place by a 200 mm x 200 mm x 8 mm galvanised washer plate, wedge discs and a spherical collar nut. At the toe of the slope, nails were installed through gabion baskets, again employing a 300 mm x 300 mm x 8 mm washer plate, wedge discs and a spherical collar nut.

Notes

Pull-out test were carried out on a total of 22 test nails which were installed at randomly selected locations throughout the site.

Scheme 30 – New cutting slope

Slope

A new cutting was required about 6 m deep and approximately 6 km long for the extension of a rail line. The slope angle was 45°.

Nail details

Some 7,200 carbon fibre nails, 16 mm in diameter and up to 12 m long were grouted into place. Seven rows of nails were installed as the cutting was excavated in two 3 m benches. Carbon fibre nails were used, as conventional steel nails were deemed unacceptable. Steel nails could act as an earth for currents induced in the rail tracks, leading to severe corrosion of the nails. Another advantage was the light weight of carbon fibre especially when installing the top row of nails in the 3 m bench. It was estimated that the carbon fibre nails were installed at about twice the rate that would have been achieved for steel nails.

Notes

Further information is available in Ground Engineering, November 2000, p10.

Scheme 31 – New railway cutting

Slope

A cutting was required through ground consisting of superficial deposits over chalk containing clay infilled solution features. The solution was a 45°, 8 m high lower slope with a 22° slope above.

Nail details

Soil nails were used to ensure the stability of the clay materials. The face was completed using Armater (honeycomb cells) filled with topsoil and subsequently hydroseeded.

Notes

Further information is given in Ground Engineering, October 2000, p13.

Appendix J: Acknowledgements and participating organisations

J.1 Acknowledgements

The authors are particularly grateful to the staff associated with the schemes described in Appendices A to H for their help. The authors would especially like to acknowledge the assistance and support received from Mr Alex Kidd of the Highways Agency and Mr Bernard Myles of BMGeo.

J.2 Participating organisations

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Amec Capital Projects
Amec Piling
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British Reinforced Concrete Engineering Co
Can Geotechnical
Channel Tunnel Rail Link
Dywidag International Systems
East Devon District Council
E C Civil Engineering
Ernest Farley & Sons (Mowlem)
Groupe TAI
Highways Agency
Howard Humphreys
Huddersfield Town F C
Hyder Consulting
Ischebeck Titan
Jersey Public Services Department
Keller Colcrete
Kvaerner Cementation
L G Mouchel and Partners
London Underground Limited
Melbourne Contest Weeks
Miller Civil Engineering
Norwest Holst Construction
P Trant
Parkman
Peter Brett Associates
Peter Fraenkel and Partners
Phi Group
Railtrack
Richies
Road Management Group
Rust Consulting
Scott Wilson Kirkpatrick
Sharrock Shand
Sir Alexander Gibb and Partners
Sir William Halcrow and Partners
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W S Atkins
W T Specialist Contractors

Abstract

Soil nailing is a useful, economic technique for the construction of new steep cuts or the strengthening of existing slopes. While the technique has much potential it has been adopted more slowly in the UK than in other countries. It can be difficult to achieve the optimum balance between economy and safety and, despite the availability of an Advice Note and a British Standard, design solutions have varied widely. A number of soil nailing schemes were examined (eight of them in some detail) and used to produce a view of current experience and, where possible, current best practice was identified. The eight case histories are included in the appendices. This report should be of value in providing guidance to clients and designers involved with soil nailing works.

Related publications

TRL466 *A review of the use of spaced piles to stabilise embankment and cutting slopes* by D R Carder and J Temporal. 2000 (price £25, code E)

TRL373 *The use of soil nails for the construction and repair of retaining walls* by P E Johnson and G B Card. 1998 (price £35, code H)

RR380 *The development of specifications for soil nailing* by R T Murray. 1992 (price £50, code P)

CR54 *Soil corrosivity assessment* by D Eyre and D A Lewis.1987 (price £20, code C)

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