TRANSPORT RESEARCH LABORATORY

The use of soil nails for the construction and repair of retaining walls

Prepared for Quality Services (Civil Engineering), Highways Agency

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Executive Summary

The aim of this report is to encourage the use of soil nailing in the construction of new retaining walls and the strengthening of existing ones, where technical or economic benefits would result. Soil nailing is a relatively new technique and some guidance is available in BS8006:1995 *Code of practice for strengthened/reinforced soils and other fills* and the Department's Implementation Standard BD70 (DMRB 2.1) *Use of BS8006:1995.* However, most of this advice relates to reinforced earth (using imported fill) rather than to soil nailing (into natural ground). This report discusses the design of essentially vertical walls (battered back to a maximum of, say, 5^{o}). The analysis of inclined walls is more complex and only limited comment is provided. At the present state of experience it is recommended that nails are not used for bank seats, bridge abutments or in situations where very high or cyclic external loadings are likely. The report attempts to draw together the relevant parts from the documents above and also useful parts from other standards and publications. Where published guidance is not available some discussion and advice is provided which should be of value to prospective designers and clients.

When producing a soil nail design engineers have generally adapted related guidance documents and standards on soil reinforcement and ground anchorages. These documents are not always completely appropriate to the design of soil nailing solutions, particularly for highway schemes where compliance with the Department's existing Standards and Advice Notes is required. The publication in 1994 of HA68 (DMRB 4.1) *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques* provided a design method for nailed slopes. There has been no comparable design method published for soil nailed structures and this lack of definitive guidance may have discouraged their use. The recently published BS8006:1995 *Code of practice for strengthened/reinforced soils and other fills* gives limited guidance on soil nails used to support structures.

A review of two soil nailed retaining walls, built on the trunk road network, has been completed with particular emphasis on the method of design and the selection of design parameters. One scheme involved the construction of a new wall while the second required an existing wall to be strengthened. The structures have been inspected and discussions held with the designer, engineer and contractor to identify and understand the design philosophy, obtain feedback on problems encountered and solutions developed. The report also provides worked example design checks of these two schemes.

While both designs were based largely on a draft version of BS8006 (dated July 1991) there were some significant differences of approach. One design took a simple Coulomb wedge as the critical failure plane, whilst the other employed a slightly more complicated formula for a single wedge which took into account the nail orientation. A major difference between the designs was the way in which pull-out resistance of the nails was handled. In one design the pull-out resistance was assumed to be independent of cover depth and the design value was based on site testing and previous experience. In the second design, pull-out resistance was calculated taking cover depth into account, with the resistance being confirmed by site testing.

There are uncertainties regarding the choice and value of the partial factors to use within a design, particularly those concerned with loadings, and the appropriate choice of soil parameters to represent long-term conditions. For this reason trial pull-out tests are normally carried out on site to help to confirm the design calculations. The results of these tests carried out by the contractor, and others by TRL, on the two schemes are included in the report. Generally, the measured pull-out values significantly exceeded those calculated or assumed in design, but interpretation of the tests requires careful consideration.

At the present state of knowledge it is not possible to provide a definitive guide to the use of soil nails, but the use of existing documents allied to sound geotechnical engineering judgement should ensure that reasonably safe and economic designs are produced. It is hoped that the design examples and discussions in this report will be of value to prospective designers and clients.

1 Introduction

1.1 Background

Soil nailing can be a useful, economic technique for the construction of new retaining walls or the strengthening of existing ones. While the basic concept of reinforcing soil with tensile elements is reasonably straightforward, the exact mechanism by which nails support a wall cannot be modelled easily. Numerous assumptions and simplifications must be made to produce a quantitative design in which the nail properties and spacings are defined. The details of the method of installing the nails can have a significant effect on their performance and a detailed design requires a good deal of information and experienced engineering judgement.

At the present time there is little guidance available to assist in evaluating the potential for using soil nails or for selecting the appropriate method of analysis, for either new structures or for the strengthening of existing ones. The recently published BS8006:1995 *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 structures. This document is implemented for DOT schemes by BD70 (DMRB 2.1) *Use of BS 8006:1995.*

Where soil nailing has been employed, design engineers have adapted related guidance documents and standards on soil reinforcement and ground anchorages. These documents are not always completely appropriate to the design of soil nails for retaining structures, particularly for highway schemes where compliance with the Department's existing 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 and different assumptions are made by various 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 less safe or less economic than the optimum. Because of the uncertainties associated with the installation of reinforcement in natural ground, designs have tended towards the safe rather than the most economical solution. 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 where technical or economic benefits would result. The report should be of value to design engineers and clients who wish to consider the use of soil nails. Given the present state of knowledge it is recommended that nails should be considered for retaining walls including wing walls but not for bridge abutments and bank seats. Soil nails can also provide the best technical and economic solution for the repair or strengthening of existing retaining structures.

This report is written primarily around the design and construction of two vertical nailed walls on the trunk road network - one concerning the construction of a new wall and the other the strengthening of an existing wall. Because there was no commonly accepted design method available, the designers of both schemes adapted sections from a number of documents and applied soil mechanics principles to develop their designs. However, because the long-term behaviour of soil can be difficult to predict accurately and the behaviour of inclusions in the ground is not fully understood, there will always be some uncertainty in predicting the performance of soil nails.

Section 2 of this report provides some general discussion of soil reinforcing techniques. Sections 3.2 and 3.3 provide a summary of the two schemes examined. Section 4 includes a discussion of a number of items which a designer might wish to consider before or during the design of a nailed wall while Section 5 summarises the important points for design. Appendices A and B contain a check of the two designs carried out, as far as possible, in compliance with BS8006:1995, other relevant British Standards and DOT documents. Since there is no widely accepted comprehensive design methodology available, these checks should not be considered 'definitive'. Other design methods and design assumptions may also be valid.

Both the new construction and the repair works involved vertically faced walls. According to DOT requirements, as given in HA43 (DMRB 4.1), walls with faces at an angle of 70° or more to the horizontal are always considered as structures and slopes between 45° and 70° may be considered as structures depending on the consequences of failure. The design of non-vertical walls is more complicated and limited discussion on their design is given in Section 4.5.2.

1.3 Methodology

The report is based on a review of existing schemes where soil nailed retaining structures have been designed and constructed. 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 their design, construction and durability. In addition a number of British Standards and other documents were examined to identify and assemble those parts most useful to a designer.

1.4 Main published sources of information

There is no single document which provides a full design methodology for soil nails in the construction or repair of walls but there are a number which provide some guidance or advice. These include:

BS8006:1995 *Code of practice for strengthened/ reinforced soils and other fills*. This gives guidelines and recommendations for the application of reinforcement techniques to soils, as fill or *in situ*. Most of the document relates to reinforced earth techniques in which horizontal tensile elements are incorporated into a structure built from the base up using fill material of a specified quality. Little guidance is given for nailed structures where the reinforcement (generally inclined to the horizontal) is installed into natural ground whose strength, porewater pressure and corrosion potential are not well defined.

BD70 (DMRB 2.1) *Strengthened/reinforced soils and other fills for retaining walls and bridge abutments. Use of BS8006:1995*. This Departmental Standard implements the above British Standard for structures on DOT schemes.

BE3 (DMRB 2.1) *Reinforced and anchored earth retaining walls and bridge abutments for embankments* (revised 1987). This Departmental Technical Memorandum has been superseded by BS8006:1995 as implemented by BD70. One of the design methods described in BS8006:1995 for horizontal reinforcement is based on the tie-back wedge method of analysis which was given in BE3.

HA68 (DMRB 4.1) *Design method for the reinforcement of highway slopes by reinforced soil and soil nailing techniques* (1994). This Departmental Advice Note gives guidance on the strengthening of highway earthworks using either horizontal or inclined reinforcement.

BS8081:1989 *Code of practice for ground anchorages*. Ground anchorages differ from soil nails in that they are active, pre-tensioned reinforcements, but some methods of analysis using wedges are common to both techniques.

BS8002:1994 *Code of practice for earth retaining structures.* This document is relevant to the new construction or repair of a wide range of retaining walls.

Table 1 provides a summary of the published methods and a comparison of the key factors which influence design. BS8081:1989 uses a global approach to the factor of safety while the other three methods use partial factors. As different approaches and assumptions are made in the documents care should be taken when comparing the values of the factors.

2 Soil strengthening techniques

2.1 General

There are a number of techniques available to increase the stability of new or existing soil structures by the inclusion of reinforcement of a greater strength than that of the soil or the fill. This greater strength could be in tension, shear or a combination of the two. The reinforcements may be built into the structure as it is constructed from the bottom up using a specified fill material. Alternately the reinforcements may be installed into existing native soil, and in the case of the construction of a new nailed wall the building sequence will be from the top downwards. Reinforcements built up in fill will normally be horizontal 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 4.7.2. 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 develop in the nails and resist further movement.

Alternatively the reinforcement may be shorter and thicker and installed more steeply so that it crosses the potential failure plane approximately at right angles, Figure 1b. In this case movement of the soil wedge would tend to induce bending and shear in the reinforcement which would act as dowels. There is no generally accepted method of calculating the restoring force which could be developed in this situation.

For a soil nail to develop the same restoring force in bending and shear as it would in axial tension, a greater soil displacement will generally be required (Jewell and Pedley, 1990) particularly with soft soils which would tend to flow around the reinforcement. Thus where reinforcement is intended to work in axial tension it should be installed at an angle whereby a small movement of the soil will quickly generate tension in the nail. In a vertical wall nails will typically be installed at a downward inclination of 10^o to 20^o.

Whichever reinforcing technique is chosen, it is important to consider the porewater pressures which could develop during the service life of the structure. A reinforced earth wall constructed from the bottom up, using a specified, generally free-draining fill should enable the designer to control the build-up of porewater pressures. Careful detailing of drainage measures within and around the structure should prevent excessive porewater pressures being generated during the service life of the structure.

Porewater pressures may be much more problematic with *in situ* nailed structures. The designer will need to consider whether the present or future porewater pressures generated in the soil is such that soil nailing is simply inappropriate. Drainage measures are generally included in nailed structures because of the importance of preventing the build-up of destabilising positive porewater pressures.

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. Some of the discussion and disagreement in the technical press regarding reinforcing systems may have been due to a lack of clarity over the appropriate behaviour of the systems.

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.

Table 1 Comparison of design methods and their partial factors

	BS8006:1995	*HA68*	*BS8081:1989*	*BS8002:1994*
Design approach	Limiting equilibrium	Limiting equilibrium	Limiting equilibrium	Limiting equilibrium
Shape of failure surface	Single wedge (for near vertical)	Double wedge (applies <70° slopes)	Single wedge (granular) circular (cohesive)	Log spiral
Representative soil parameters	Cautious estimate/ worst credible	Minimum conceivable	No specific guidance	Cautious lower limit
Water regime	No specific guidance	Conservative values	No specific guidance	Most onerous reasonably possible
Material factor				
tanϕ'	1.0	Varies: 1.0 - 1.5 on peak, critical state or residual	N/A	Lessor of 1.2 on peak or 1.0 on critical state
c'	1.6	1.0 - 1.5 on peak strength ($5kN/m^2$ max.)	N/A	1.2
c_u	Not used	Not used	Adhesion factor = 0.3 to 0.35 on c_u	1.5
Height, embedment and unplanned excavation	H =vert dist from toe to intersect of arc tan 0.3 and upper ground line plus minimum embedment of 0.45m	H = lower slope height		H = retained height plus the greater of: a) 0.5m b) 10% retained height for unplanned excavation
Minimum surcharge	Not given	Not given	Not given	$10kN/m^2$
Load factors (ULS)	Comb A Comb B			Soil Structure
Vertical soil loads	1.5 1.0	1.0	Overall FS = 1.5	1.0 1.0
Vertical surcharge loads	1.5 1.5	1.0	Overall FS = 1.5	1.2 (BD 37/88 1.2 (BD 37/88)
Non-vertical soil loads		As vertical		1.0 1.0
Non-vertical surcharge loads		As vertical		1.5 (BD 37/88) 1.5 (BD 37/88)
Pull-out capacity	1.3	Interface sliding factor based on tests or residual strength	FS = 3 on ultimate load to give design load	N/A
Foundation bearing capacity	1.35	N/A	N/A	No specific guidance
Base sliding	1.2	Depends on interface sliding factor	N/A	No specific guidance

2.2 Soil nails

Soil nails involve the insertion, either by boring or driving, of tensile elements into otherwise undisturbed soil or fill and must cross the potential slip planes along which failure is most likely. When inserted into bored holes they need to be grouted to ensure 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 part of the soil begins to move relative to the rest, although some pre-load may be generated during the construction process if small soil movements occur. The stabilising force is taken to be generated by friction (plus possibly a small contribution from adhesion) along the nail in the stable part of the soil (generally termed the resistant zone) and this force is transmitted to the potential failing block (the active zone) by a combination of friction along the nail in the block and bearing pressure on the wall or facing.

In the UK it has generally been taken that soil nails increase the stability of a wall through axial tension within the nail although there could be some small contribution from shear forces in some circumstances. This shear contribution is normally ignored in design (HA68, DMRB 4.1; Davies and Jewell, 1992).

Table 2 lists published soil nailing applications giving details of the range of ground conditions and geometry.

There are few published UK case studies covering the repair of retaining structures using soil nails. Long et al (1984) describe the procedure for repairing a conventional reinforced soil wall that had suffered localised damage as a result of the freezing of saturated backfill directly behind the

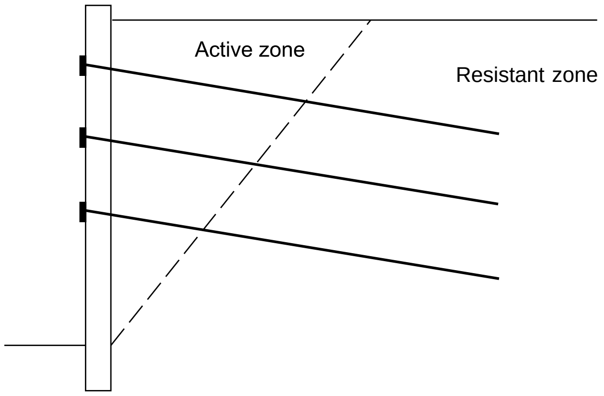

Fig 1a Soil nails

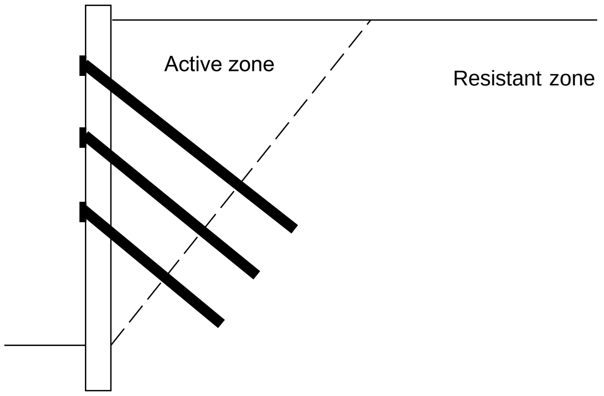

Fig 1b Soil dowels

Figure 1 Soil nails and soil dowels

facing. The method of repair involved reconstructing the damaged face with cast *in situ* panels that were connected to existing undamaged reinforcement. Drilled and grouted nails 4.2m in length and 28mm diameter were then installed to overlap the existing damaged reinforcement.

Bruce and Jewell (1987) describe the repair of a 2m to 3m high, 125m long drystone masonry retaining wall, at Denholme Clough, near Bradford using bored and grouted 16mm diameter steel nails installed into a 115mm diameter hole, with one nail for 2.5m^2 of facing. The repair of these walls without demolition was found to be technically and commercially attractive. Construction details could be varied, for example the old facing could be retained by setting the nails heads into the drystone wall itself.

Other case studies of the use of soil nails to repair existing walls in the UK and Europe are given by Whyley (1996), Thompson (1994), Barley (1993) and Schwing and Gudehus (1988).

2.3 Ground anchorages

Ground anchorages provide a stabilising force from a grouted length of tendon behind the potential failure plane which is transferred along a debonded length of shaft to a surface bearing plate (BS8081:1989). 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 the surface bearing plate. Thus stabilising forces are generated without the need for any soil movement within the structure. Ground anchorages are often installed at approximately right angles to the worst potential failure plane and thus their effect is mainly one of increasing the frictional resistance along the plane by an increased normal force across that plane.

Littlejohn (1990) has reviewed the design and construction of ground anchorages, and Turner et al (1993) has described the design and use of ground anchorages to support a large precast anchored retaining wall to the A55 North Wales Coast Road.

2.4 Soil dowels

Soil dowels are relatively large diameter piles inserted into the ground across a potential failure plane. They are often installed approximately at right angles to the failure plane and provide enhanced shearing resistance mainly by their large diameter to length ratio and high bending stiffness. There is no standard design method for calculating the stabilising force which they can generate.

2.5 Reinforced and anchored earth

These techniques differ from the three above in that the tensile elements are incorporated into the structure as it is constructed using layers of selected fill. Thus much better control of the fill properties and drainage conditions are possible. With reinforced earth (with strips attached to the rear of the facing) no rigorous 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

Table 2 Applications of soil nailing (after Murray, 1993)

Application	*Soil types*	*Nail type*	*Max. Height (m)*	*L/H[1] range*	*Slope range (deg.)*
Retaining wall	Sands and Gravels Overconsolidated clays Glacial Till Silty clay Mudstone	Drill/grouted (DG) and driven (D)	14 (DG) 12 (D)	0.5-1.2 (DG) 0.5-1 (D)	80-90
Cut & cover tunnel	Sand	Driven	11.5	1	90

[1] *H = height of structure and L = length of nail.*

anchored earth (typically with the threaded end of the anchor passing through the facing) some pre-load is induced during construction and there is also an opportunity to tighten the facing nut to a specified value, 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 would tend to increase the tensions in the reinforcing strips or anchors.

Although this report deals with soil nails it is useful to summarise the design approach for reinforced earth (built from the bottom up using fill) in BS8006:1995 to help identify the similarities and differences between reinforced earth and soil nailing. The main features of a reinforced or anchored earth wall are as follows:

1. The fill material will have to meet specifications for its mechanical properties (primarily relating to its coefficient of internal friction), and grading.
2. The fill material must be reasonably consistent to meet the specification throughout construction.
3. The fill is placed and compacted during construction and so its density and strength can be controlled and maintained.
4. The fill material must be free-draining.
5. The structure will usually be built up above the existing ground level and this will assist drainage of the fill.
6. There is the opportunity to include drainage measures into the structure during construction.
7. The reinforcements are always placed horizontally (which simplifies design and construction).
8. The vertical spacing of the reinforcement is usually based on proprietary facing units and lifts of fill material suitable for compaction.
9. There is a large body of experience on the behaviour of reinforced earth structures.

In many respects soil nailing is similar to conventional reinforced soil but there are differences that can greatly influence the requirements for design and construction. Important among these are the following:

1. The properties of the natural soil, with regard to strength and corrosion potential may be much more variable and greatly inferior to those existing in a conventional reinforced soil structure where selected granular fill is used. For the same reason the prediction and control of porewater pressures is rather more difficult in natural cohesive soils than in selected granular fill.
2. Most commonly, soil nails are installed at an inclination to the horizontal in contrast to conventional reinforced soil where the elements are installed horizontally. The inclination affects their interaction with the soil.
3. The reinforcements are installed by drilling and grouting or by driving rather than by placement and compaction within fill. Soil nailing installation methods may generate different pull-out resistance (generally higher but possibly lower) than those generated by placement and compaction of fill.
4. The construction process will often involve starting at the top and working downwards.
5. The facing to structures is usually formed on site from sprayed concrete rather than by using precast concrete or other prefabricated units.
6. Drainage requirements must be introduced as a separate part of the installation process rather than by forming part of the construction.

3 Design examples

3.1 General

The two designs examined in this report were both based largely on BS8006:1995 (in practice they were based on a late draft version since the final version was not published until late 1995). This British Standard is implemented for DOT schemes through BD70 (DMRB 2.1). The design checks on the as built schemes, given in Appendices A and B, are also based largely on BS8006:1995 with additions where the British Standard does not fully cover certain aspects.

3.2 Design of a new retaining structure structure (Scheme A)

3.2.1 Design requirements

Scheme A comprises a new grade separated junction at Ashley Heath in Dorset where the A31 meets the A338 (both being dual carriageways). It involved relocating the A31 in cutting beneath the roundabout which is formed by two new bridges. The cutting necessitated a considerable amount of ground retention and soil nailing was chosen to form a reinforced soil retaining structure. The maximum height of retained cut is 6m formed through medium to coarse gravel underlain by weakly cemented fine to medium sand. A typical cross section of the wall is shown in Figure 2. The bridge abutments were supported by conventional bored piles.

3.2.1.1 Soil parameters

The design team had previous experience of the local ground conditions and a comprehensive site investigation had also been carried out. Additional investigations were carried out during the design to check such matters as the maximum depth of self supporting excavation. The soil parameters used in the design are set out in Table 3.

For design, a partial factor f_{ms} = 1.45 was applied to tan ϕ'_{peak} to obtain tan ϕ'_{des} to take account of variability in the soil and other uncertainties during the service life of the wall. Zero cohesion was assumed for all soils.

The groundwater table was at depth beneath the structure and drainage measures were incorporated within the structure; for these reasons it was assumed that positive porewater pressures would not be generated during the service life of the structure.

3.2.1.2 Soil nail design

The original design was based on the draft BS8006:1991 but incorporated advice given in Technical Memorandum

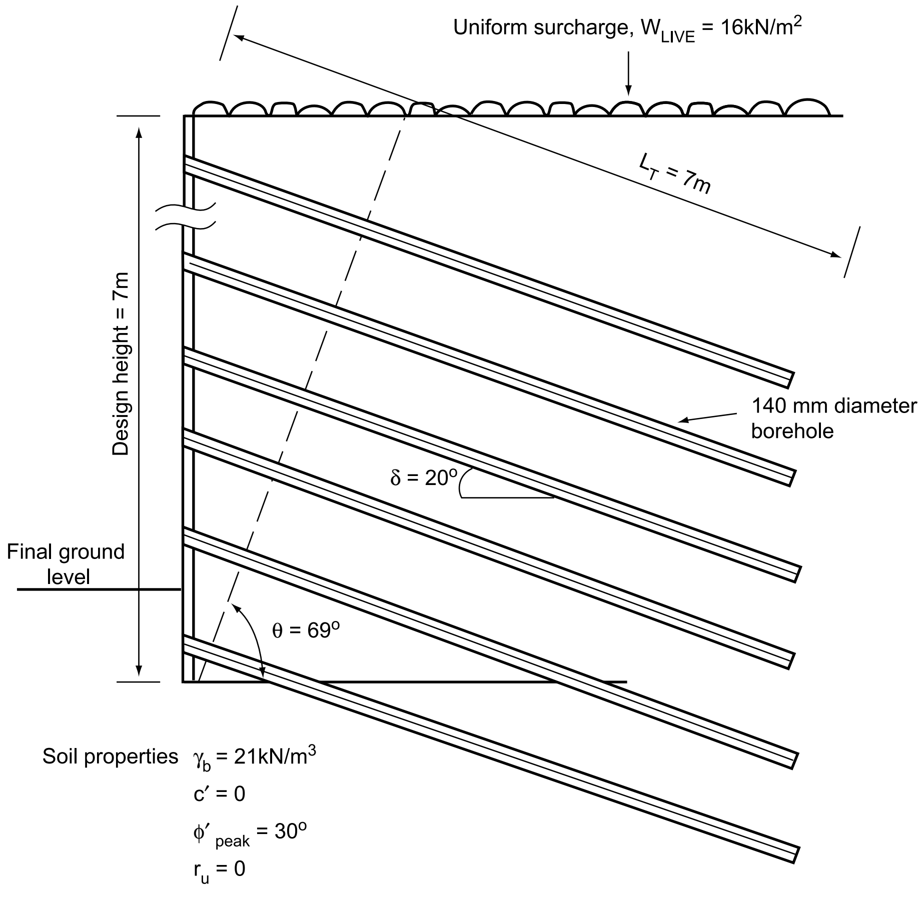

Figure 2 Typical cross section for the new retaining structure - Scheme A

Table 3 Soil design parameters

Parameter	*Design value*
Soil density, kN/m^3	21
Effective peak angle of friction, ϕ'_{peak}, in degrees (all soils)	40
Effective design angle of friction, ϕ'_{des}, in degrees (all soils), f_{ms} = 1.45	30
Effective cohesion, c'_{des}, kN/m^2	0
Pore water coefficient, r_u	0
Bond coefficient, f_b (from BS8081:1989)	0.8
Characteristic strength of nail, N/mm^2	460
Partial material factor on nail strength, γ_{ms}	1.15

BE3. A uniform surcharge of 16kN/m^2, equivalent to 37.5 units of HB loading, was adopted. Partial load factors were employed in accordance with BD37 (DMRB 1.1) and not Table 6.4 of the draft BS8006:1991 (Table 17 in BS8006:1995). In this case a value of γ_{fL} = 1.2 was applied to the vertical loads and soil weight instead of the f_{fs} = 1.5. This lower value decreases both the disturbing force and available pull-out resistance of the nails. Partial material factors were applied to the soil strength, tan ϕ'_{peak}, the yield stress of the steel nail, σ_y, and the bond or skin friction between the soil/grout interface, f_b, to obtain the design values as summarised in Table 3.

Equilibrium of the reinforced structure was checked for potential failure mechanisms on several two-part wedges using the design soil and allowable nail resistance. A limiting equilibrium calculation was then applied to determine the soil nail reinforcement required. Overall equilibrium of the structure was also checked for failure mechanisms passing through the nailed zone and emerging at the toe of the excavation. For a vertical wall the two-part wedge analysis reduces to an equivalent single wedge equation as described in Section 7.5.5.3 of BS8006:1995. This equation was used for analysis in conjunction with the software program NAIL-Solver (Jewell, 1990). This program is based on limiting equilibrium of a two-part wedge mechanism but was originally devised for slopes not structures. The method of analysis is similar to that described in HA68 (DMRB 4.1) for slopes using a two-part wedge.

The reinforced soil walls have been designed for a height of 7m, comprising the maximum retained height of 6m with a 1m allowance for embedment and unplanned future excavations. This is not strictly in accordance with Section 6.4 of BS 8006:1995 which recommends a *mechanical height* equivalent to the design height for the particular structure geometry. However, the allowance of 1m appears reasonable and is more consistent with the advice given in BS8002:1994 which suggests a minimum additional allowance of 0.5m or 10% of the retained height.

For a nail installation angle of 20° below the horizontal the original analysis, using a single wedge mechanism, indicated a required total reinforcement force of T_{max} = 321kN per linear metre of wall. To provide this reinforcement force, needed seven rows of 140mm diameter bored and grouted nails with a nail length of 7m, based on a vertical and horizontal spacing of 1m and 1.5m respectively. The number of nails and the spacing were chosen to satisfy both design and construction considerations as described in Section 3.2.2.

3.2.1.3 Summary of final design

The final design can be summarised as follows:

● Length of nails	7m
● Orientation	20° below the horizontal
● Vertical spacing	1m
● Horizontal spacing	1.5m on a regular grid
● Diameter of steel bar	25mm high yield steel
● Borehole diameter	140mm
● Durability requirements	Galvanizing followed by factory grouting into an impermeable sleeve

3.2.2 Construction

Excavation and installation of the soil nails progressed in 1m benches using a top-down construction method. The exposed vertical cut was limited to a height of 1m between soil nails to ensure stability of the excavation. After cutting each bench the vertical face was protected with a 100mm thick sprayed concrete layer with a central steel reinforcing mesh through which the nails were installed. On completion of the wall a concrete face was cast against the sprayed concrete. This was designed to give the appearance of a masonry wall but the combined concrete wall was considered as facing not structural support.

A filter drain was provided at the back of the face of the wall to deal with run-off and vertical drains were installed at 3m centres to remove any perched water accumulating behind the face of the wall. In view of this the pore water pressure coefficient, r_u, was set to zero in the design.

3.2.3 Pull-out testing

Pull-out tests were carried out at the start of construction and the results of these tests were higher than those required for design. Proof tests were also carried out during the works; as the soils are non-cohesive sands and gravels these pull-out forces should be available in the long-term.

3.2.3.1 Initial testing

Before any permanent works nails were installed the designer required a test panel of 12 nails to be installed with the nails subject to pull-out tests generally in accordance with the procedure given in Murray (1993). A trial area was excavated and the nails installed in 140mm boreholes inclined downwards at an angle of 20^o. Four nails had an effective cover depth of about 2m (TP1/1-TP1/4), four had 3m (TP2/1-TP2/4) and four had 4m (TP3/1-TP3/4). Three nails in each row were formed of 25mm diameter high yield steel bars and being 7m long were identical with the nails to be used in the permanent works. To try to generate failure at the grout to ground interface one nail in each row was only 3m long and was made from 40mm diameter high yield steel bar.

The 25mm diameter steel bar thread had a 24mm thread at one end. From previous experience it was known that the 24mm thread would fail at a load of about 240kN. A limit of 220-230kN was applied in the tests to ensure that a sudden failure would not occur which could be of danger to the operatives and damage the vernier measuring equipment. For each nail the theoretical pull-out resistance was calculated by the method given in Section 2.27 and Appendix D of HA68 (DMRB 4.1). The reaction frame was placed over the nail. This consisted of two steel I-beams about 2m in length with support stools welded to each end. The design was such that as the nail was pulled the reaction force loaded the ground at least three-quarters of a metre away from the nail to minimise any effect this might have on the pull-out result. A hollow jack was placed over the end of the nail followed by spacers, washers and nuts which were tightened to remove any slack from the system. A vernier measuring device was mounted on a surveyor’s tripod and adjusted against the tip of the nail to read zero. A load of 10% of the calculated ultimate load was applied and the axial movement at the tip of the nail measured after 0, 1, 2, 5, 10 and 15 minutes. An additional 15% of the ultimate load was applied and the movement measured at the same time intervals. This procedure was repeated with 15% load increments and six displacement readings until either the nail failed or the 220/230kN limit was reached. Where possible at the end of the test the nails were unloaded and the vernier read to give an indication of the permanent deflection of the nail. A summary of the results is given in Table 4.

As can be seen in Table 4 all the test nails exceeded the maximum calculated pull-out values. Eight out of the twelve tests had to be stopped when the load reached the safe limit for rupture of the threaded part of the nail. For the 7m long nails the measured grout to ground bond exceeded the calculated value by at least 50% for the 4m deep nails, 100% for the 3m deep nails and 300% for the 2m deep nails.

The tests on TP1/2 and TP2/3 were stopped prematurely because of problems with the equipment and the nail thread but both nails had comfortably exceeded their calculated pull-out resistance at this point.

Two of the three short nails were pulled to failure of the grout to ground bond. Test nail TP1/3 with about 2m of cover pulled from the ground under a 180kN load, nearly eight times the calculated pull-out achievable. The test on TP2/2 (3m of cover) was stopped at 230kN, nearly six times the calculated load. Test nail TP3/3 (4m of soil cover) pulled out at 350kN which was over six times the calculated value. Based on these results the Engineer and Client were content for the main works to proceed.

3.2.3.2 Proof testing

During the works some 30 nails were tested to ensure continuing compliance with the design. This testing acted as a check on the consistency of the ground conditions as well as the quality of drilling and grouting by confirming that the good results obtained from the trial panel were

Table 4 Results of initial pull-out tests

Test nail	*Nail length (m)*	*Calculated maximum load (kN)*	*Measured maximum load (kN)*	*Measured max/ calculated max (%)*	*Maximum deflection (mm)*	*Permanent deflection (mm)*
TP 1/1	7	68.5	222.6	330	7.34	2.27
TP 1/2	7	68.8	192.8	280		[1]
TP 1/3	3	22.9	180	785		[2]
TP 1/4	7	72.2	223.9	310	14.73	6.56
TP 2/1	7	102.5	225.5	220	5.21	1.95
TP 2/2	3	39.9	230	575	4.15	2.42
TP 2/3	7	103.5	165.6	160		[3]
TP 2/4	7	106.2	217.8	205	10.39	-
TP 3/1	7	136.5	218	160	5.15	0.14
TP 3/2	7	136.8	219	160	3.97	-
TP 3/3	3	57.3	350	610		[4]
TP 3/4	7	140.2	224	160	6.83	-

[1] *Test stopped at this point because of failure of the electronic vernier.*
[2] *3m nail failed at the grout to ground bond.*
[3] *Test stopped when thread on nail started to deform.*
[4] *3m nail failed at the grout to ground bond.*

being achieved over the whole site. The proof test load could have been based either on the calculated pull-out required in the design at a particular part of the works or the calculated pull-out available at a particular point. The latter was generally the more onerous case (higher pull-out) and the testing was based on this. For each test nail the pull-out resistance was calculated in the same way as for the original twelve test nails. These nails were loaded incrementally to 70% of the calculated value subject to an overriding maximum of 180kN (approximately 70% of the rupture strength of the nail thread). All the nails tested carried the proof loads satisfactorily.

3.2.3.3 Additional strain gauged nails

Two additional test nails were installed at the site. They were similar to the permanent works nails but had six sets of strain gauges located at one metre intervals down the nail. The nails were installed at a horizontal spacing of 1.1 metres and with similar cover depths. Each nail was loaded to failure while reading the strain gauges on both nails. Both nails failed by rupture of the nail thread at a load of approximately 220kN which confirmed the Engineer’s assessment of ultimate tensile strength of the nail thread. In one test the load was applied over the course of about six hours while the second was loaded incrementally over about 12 months. Little useful information could be obtained by comparing the quick and slow test results as the failure mode was nail rupture. Had it been possible to fail the nails at the grout to ground bond it was expected that there would have been no significant difference between the two tests in the granular material on site.

However, the tests did provide information on the distribution of stress along the nail during the test and also on the strain induced in one nail when load was applied to the other. The long-term nature of the tests meant the equipment had to be fairly compact to fit within a two metre wide cabinet (required to minimise the risk of vandalism to the equipment). This limitation, together with the fairly large loads anticipated, resulted in a concrete reaction pad approximately 2m wide by 1m high with two 150mm diameter holes for the nails. This arrangement could produce complications through the application of additional confining stresses close to the nail (see Section 4.11). Also if the grout column around the nail formed a continuous body with the concrete pad then the nail test might simply become a test of the compressive strength of the concrete and grout. To minimise these effects the grouting process was stopped when the level reached to within about half a metre of the rear face of the pad. In addition the gravel was ‘undercut’ by about 100mm around the nail access holes to move the point of load application a little away from the nail.

The distribution of strain along a nail at various applied loads is shown in Figure 3. As one might expect the strain drops progressively from the front of the nail where the load is applied to the far end of the nail where the tension must fall to zero. The rate of strain reduction along the nail is not constant and will depend on many of factors including the consistency of the soil and the grout. The test stopped when the nail thread ruptured at a load of some 220kN. Discussion on the progressive development of pull-out resistance is given in Section 4.11.

In addition, deformation of the nailed structure and the strain within permanent, working, instrumented nails were monitored post construction by the Engineer as part of the contract.

3.3 Strengthening of an existing wall (Scheme B)

3.3.1 Design requirements

Scheme B comprised the strengthening of an existing natural stone retaining wall supporting the A5 trunk road at Nant Ffrancon in Gwynedd. This location is within the Snowdonia National Park and an important design constraint was to maintain the visual appearance of the wall after strengthening. The wall was thought to be over 100 years old and had received various maintenance treatments over the years. The face showed signs of bulging but it was not possible to determine if this was the result of current or earlier distress. The face had been

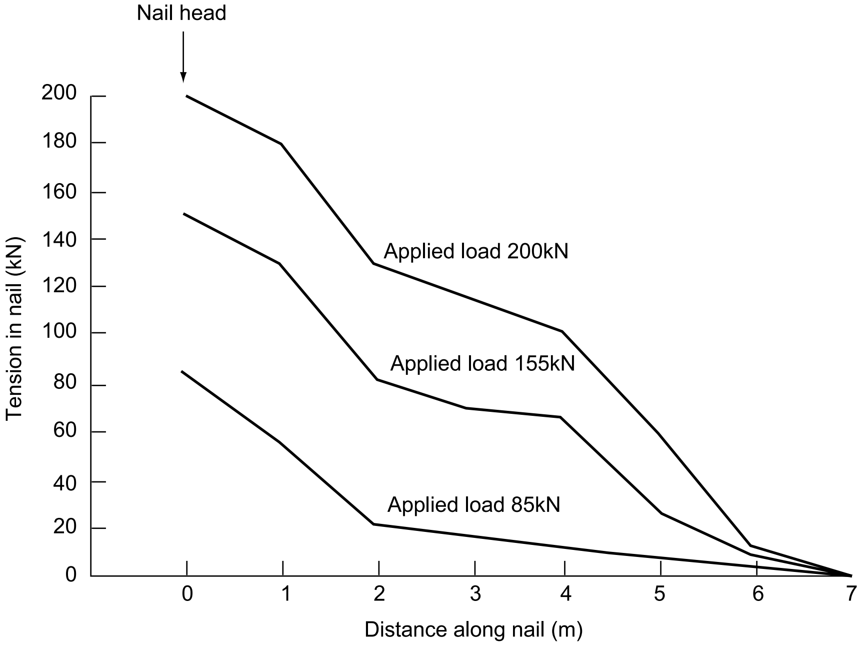

Figure 3 Distribution of tension along nail at a number of applied loads

pointed and some grouting had taken place in the past. The maintaining authority stated that for these earlier repairs care had been taken to use a thick grout in limited quantities to ensure that drainage paths remained to minimise the build-up of water pressure behind the wall. The wall is some 340m in length and varies in height between 1.5m and 4.5m. The thickness of the wall was measured during trial nail installation to be about 0.5m. The overall hill slope was assumed to be 30°. The geometry of the wall employed in the design calculations is shown in Figure 4.

3.3.1.1 Soil parameters

The ground conditions assumed in design were based on the results of a ground investigation in January 1990 and observations made during drilling for five trial nails in November 1993. The wall was identified as retaining Glacial Till which was mostly granular in nature with a subordinate clay fraction. The design assumed that the material behind the wall and wall foundation material was predominantly granular scree material, with a peak effective angle of friction (ϕ'_{peak}) of 35°, an effective cohesion(c') of zero and unit weight of 18kN/m^3.

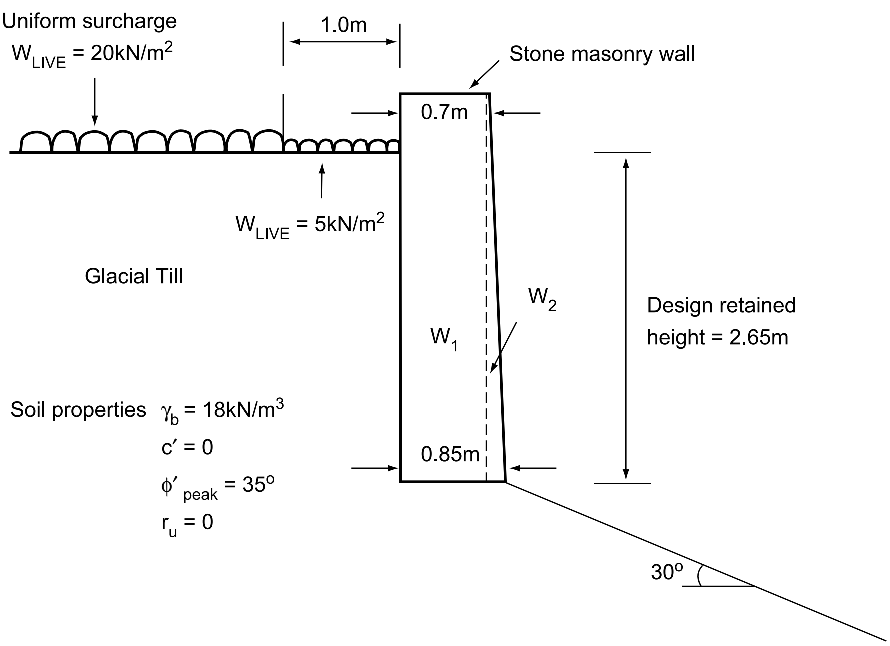

Figure 4 Typical cross section for strengthening of existing wall - Scheme B

The wall was assumed not to be influenced by groundwater. For this reason the pore water pressure coefficient, r_u, was set to zero in the design.

3.3.1.2 Soil nail design

The design of the strengthening scheme using soil nails was based on the draft BS8006:1991 (current at the time of the works). Partial load factors were applied to the disturbing earth pressure, $f_f = 1.5$ and surcharge load, $f_q = 1.5$ in accordance with Table 6.4 of draft BS8006:1991 (Table 17 of BS8006:1995). However, the soil strength was also factored with a mobilisation factor of 1.5 being applied to tan ϕ'_{peak} to give a ϕ'_{des} value of 25°.

A uniform highway surcharge of 20kN/m^2 was assumed beyond a one metre strip adjacent to the wall (representing a 1m footway). This loading is equivalent to 45 units of HB loading in accordance with the requirements of BD37 (DMRB 1.3).

A single wedge analysis was used to determine the critical wedge and to calculate the required restoring force (T_{max}) as given by Eq 8.12 in draft BS8006:1991, (BS8006:1995 Section 7.5.5.3) which is simplified as:

$$T_{max} = W \tan(\phi'_{des} - \theta) / [\cos\delta + \sin\delta.\tan(\phi'_{des} - \theta)]$$

where:

W = weight of wedge and the portion of surcharge acting on the wedge

θ = angle of slip plane, taken to be 45° + (ϕ'_{peak})/2 = 62.5° (to horizontal)

δ = soil nail inclination (downwards) = 12°

ϕ'_{peak} = peak angle of shearing resistance = 35°

ϕ'_{des} = design angle of shearing resistance = 25°

The analysis assumed the back of the wall face was vertical. The back of the wall face may have been slightly inclined but this would have had only a small influence on the calculated value of T_{max}. Ignoring the strength of the existing wall, the required restoring force was 65kN per metre run of wall.

The ultimate pull-out resistance of the nails was assumed to be 10kN per metre length of nail; the designer took the view that the pull-out resistance was not greatly influenced by the depth of overburden. Five preliminary pull-out tests were carried out followed by a further nine at the start of the works to confirm that a value of 10kN/m was achievable. Further information on the tests is given in Section 3.3.3.

The anchorage length of all the nails was assumed to be the length of nail beyond the slip plane at mid-height of the wall, giving an effective length of 5.6m for a nail of 7m total length. This overestimates the anchorage length of nails installed in the upper part of the wall, but underestimates it in the lower part.

The working resistance of the nail was taken to be the ultimate resistance divided by 2.5. This factor of 2.5 was produced by considering the factor of safety on the grout/ ground interface (f_m) in Table 2 of BS8081:1989 and a factor relating to the ramifications of failure (f_n) in Table 6.2 of draft BS8006:1991 (Table 3 of BS8006:1995). For permanent works BS8081 suggests f_m is in the range of 3 to 4 while BS8006 gives f_n as 1.1. On first inspection this would appear to give a combined factor of 3.3 to 4.4. However, the BS8081 approach is one of a global factor of safety so it is inappropriate to multiply it by a partial factor from a different standard. Because a mobilisation factor of 1.5 had already been employed to obtain ϕ'_{des} and a load factor of 1.5 applied to the soil unit mass and external loading the designer chose a lower combined factor value of 2.5 to calculate the working resistance of the nails. This produced a working pull-out resistance of 22.4kN for a 5.6m effective length of nail.

The following approaches were considered when determining the required number of nails and spacing.

- Three nails per metre length of wall would be required to provide the entire restoring force (65/22.4), assuming the wall provides no resistance.
- One nail per metre would be required to increase the factor of safety of the wall from 1.0 to 1.5, assuming the wall was currently in equilibrium (factor of safety = 1.0).
- Two nails per metre would be required if the shear resistance of the wall base was subtracted from the required resistance. The base resistance was calculated on the basis of an angle of shearing resistance of 35° and applying a mobilisation factor, f_m of 1.5. Wall thickness was taken to be 0.6m.

In the event it was decided to employ 1.5 nails per metre run of wall. To restrain local bulging, the nails were spaced at 2m centres horizontally and 0.75m vertically in three rows, on a diamond pattern as shown in Figure 5.

3.3.1.3 Summary of final design

The design details can be summarised as follows:

- Length of nails: 7m
- Orientation: 12° below the horizontal
- Vertical spacing: 0.75m
- Horizontal spacing: 2m on a diamond grid
- Diameter of GRP nail: 22mm OD, 12mm ID
- Borehole diameter: 68mm
- Durability requirements: Glass reinforced plastic (GRP) nail employed (option of 25mm stainless steel also permitted)

The top one metre of each soil nail was left ungrouted to permit some pre-load to be applied to the nail during construction.

3.3.2 Construction

The wall was located in a National Park and the strengthening works had to maintain the character and appearance of the original wall. This was a contributory reason for employing soil nails. The wall supported the road and access scaffolding was installed on the ground in front of the wall.

Nails were installed approximately on the 0.75m (vertical) and 2m (horizontal) diamond grid pattern as shown in Figure 4. Since a notional ‘invisible’ strengthening was required the construction process was as

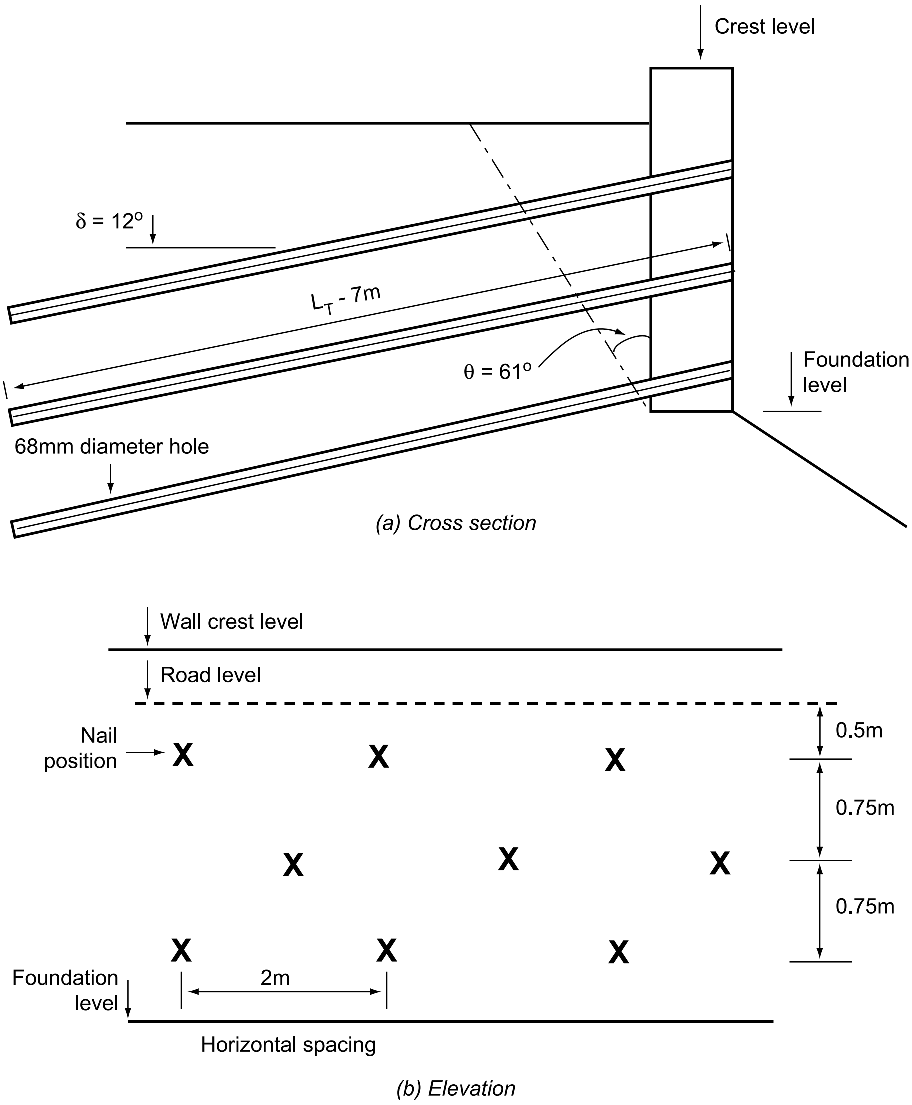

Figure 5 Soil nail layout for strengthening of existing wall - Scheme B

follows. The contractor chose a large stone block in the wall in approximately the required position of a nail. The drilling equipment was mounted on the scaffolding and aligned at the correct location and orientation. A 150mm diameter by 150mm deep hole was cored in the block and the core retained. A 68mm diameter borehole was then drilled through the remainder of the block and into the ground behind the wall to a depth of 7m. A 7m long glass reinforced plastic (GRP) nail was fitted with centralisers and inserted into the hole. Grout was then tremied through the 12mm bore of the nail until it flowed up the annulus around the nail to within about 0.5m of the wall.

After the grout had set, a 130mm diameter spreader plate was placed over the nail and a nut tightened against the plate. The grout usually settled slightly in the hole after placement to leave a 0.5m to 1m unbonded length of nail immediately behind the wall. Thus some pre-stress could be applied to the nail by tightening the nut. The nails were not tightened using a torque wrench but the contractor and Engineer estimated the tightening effort which should be applied to develop a pre-load of about 20kN in the nails. This method of installation should reduce the potential in-service movement of the wall.

The cores were then trimmed to length and replaced in the blocks using mortar of the same colour to produce an essentially invisible repair.

3.3.3 Pull-out testing

3.3.3.1 Preliminary testing

At an early stage, prior to design, the Client employed a specialised soil nailing contractor to install and test five nails at the proposed site to give an indication of the pull-out forces which could be generated. From the earlier site investigation and from the trial nail boreholes it was found that the material was a glacial till of a mainly granular nature with some clay, granite and slate infill behind the wall. Five trial nails were installed, two of steel and three of glass reinforced plastic (GRP). Three slightly different sizes of hole were drilled largely as a check on the suitability of different drilling techniques. The nails were grouted along their full length and the grout permitted to set before the tests were carried out. A summary of the results of these preliminary pull-out tests is given in Table 5.

Table 5 Preliminary pull-out tests

Test nail No	*Nail type*	*Nail length (m)*	*Hole dia. (mm)*	*Max pull-out force (kN)*	*Max grout/ ground bond (kN/m)*	*Max grout/ ground shear stress (kN/m^2)*
1	Steel	5	43	170	34	252
2	GRP	5.5	46	10	1.8	13
3	Steel	5	46	20	4.0	28
4	GRP	5	50	50	10.0	64
5	GRP	3.4	50	70	20.6	131

As can be seen a very wide range of pull-out forces was obtained. From his experience on similar sites, the soil nailing contractor took the view that the cover depth over the nail was not a major factor in generating pull-out resistance and no attempt was made to relate measured pull-out capacities to overburden depth. Nails 1 to 4 had a mean cover depth of about 2.5m while nail 5 had about 1.5m of cover. The nailing contractor explained the two very low results (tests 2 and 3) as being due to poor grouting resulting from trying different mixes and different grouting pressures to optimise the process. The contractor was confident that good, consistent pull-outs would be obtained for the permanent nails. From the remaining three results and pull-out tests carried out on other similar sites the contractor recommended that an ultimate pull-out resistance of 10kN per m length should be used in design. This value was taken in the design calculations with a partial factor of 2.5 to provide a working pull-out resistance of 4kN/m length for the nails. The designer required a number of pull-out tests to be carried out at the start of the main works to confirm that the assumed design ultimate pull-out values were being achieved or exceeded.

3.3.3.2 Testing at the start of the works

The testing done at the start of the works employed the same GRP nails as would be used in the main works. They were 7m in length and had an OD of 22mm and an ID of 12 mm. The outside diameter was reduced slightly at the end and a plastic thread moulded onto the nail to accept the locking-off nuts. The manufacturer’s literature gave the ultimate tensile strength of the body of the nail as 310kN while the threaded portion of the nail had an ultimate tensile strength of 160kN and a working load capacity of 100kN. Unlike the working nails these trial nails were debonded over the top 3m by means of a plastic tube around the nail thus leaving only the bottom 4m length of nail fully bonded to the ground. (In practice the grout around the tube covering the top 3m of nail would provide some additional pull-out resistance but this was assumed to be small and was ignored) (Table 6).

The permanent works nails and these nine test nails were installed in 68mm bored holes rather than the 43mm to 50mm holes in the earlier trials. This greater diameter would have helped generate larger pull-out forces than were obtained in the preliminary trial. The first two test nails failed at the grout to ground bond at rather lower values than were anticipated. This was probably because the grout was too thin and tended to run into the voids in the wall. A thicker grout was employed in the later tests and these provided higher pull-out resistances. The first test nails failed suddenly and the gauges used for monitoring movement were damaged. It was decided that once the test loads had reached 80kN the test would be terminated because this load represented a pull-out capacity of 20kN per metre of nail, twice the ultimate value assumed in design. On the basis of these results the Engineer and the Client were content to proceed with the works as designed.

3.3.3.3 Proof testing of working nails

As the works progressed regular ‘proof testing’ was carried out on about 10% of the permanent works nails. This amounted to 51 tests. The normal working nails were grouted to within about 0.5m to 1.0m of the wall face. The thick grout employed settled slightly in the hole giving a grouted nail length of about 6m. In the proof tests the nails were subject to a load of 40kN applied in 10kN increments. The grouted length of nail was about 6m rather than the 5.6m effective length assumed in design. This equates to a required working pull-out resistance of 24kN instead of the

Table 6 Pull-out tests at start of works

Test nail No	*Bonded nail length (m)*	*Hole dia. (mm)*	*Max pull-out force (kN)*	*Did nail fail?*	*Max grout/ ground bond (kN/m run)*	*Max grout/ ground shear stress (kN/m^2)*
1	4	68	40	yes	10	47
2	4	68	30	yes	7.5	35
3	4	68	80	no	20	93
4	4	68	90	no	22.5	105
5	4	68	80	no	20	93
6	4	68	70	yes	17.5	82
7	4	68	80	no	20	93
8	4	68	80	no	20	93
9	4	68	80	no	20	93

22.4kN required in design for a 5.6m length. The 40kN proof load was thus 1.66 times the design load for a 6m nail. The results of all of these tests showed that the nail to grout bond and grout to ground bond were satisfactory.

3.3.3.4 Additional strain gauged nails

Six of the GRP nails were strain gauged before installation on site. Apart from the strain gauges they were identical to the production nails used on site. Five sets of resistance strain gauges were fitted to the nails at distances of 2m, 3m, 4m, 5m, and 6m from the threaded end of the nail.

Two nails were installed as part of the permanent works and the strains monitored for a year; as with the other permanent works nails they were grouted to within about 1m of the wall face. The strain gauges were read two days after grouting and these readings were taken as the zero strain values. Immediately following this, the nail reaction plate and nut were installed and the nut tightened. No specific torque value was specified but it was estimated that this operation generated a tensile load of about 20kN in the unbonded length of nail. The strain gauges were read then and subsequently at about monthly intervals. The strain gauges installed 2m from the top of the nail showed a tensile strain of about 300microstrain (µstrain) due to the tightening of the nut. This corresponds to a tensile load of about 4kN in the nail and suggests that the load was quickly shed into the ground. The gauges 3m from the top showed a tensile strain of about 110µstrain (1.5kN). The lower gauges showed strains of the order of 20µstrain (0.25kN). Over the subsequent 12 months the strain in the top gauges increased slightly to about 400µstrain (5.3kN) while at the remaining locations reduced slightly. Because of the inherent inaccuracies in reading resistance strain gauges in damp conditions and because of possible temperature effects and grout shrinkage effects it is not possible to infer very much from these fairly small changes in strain. The tightening of the nut had a measurable effect on the tension in the nail but it seems unlikely that any significant strains had been induced in the nail by movement of the soil or wall.

The other four strain gauged nails were installed in a similar manner to the earlier test nails with about a three metre sleeve over the top section of nail which gave a 4.1m grouted length. Two nails were tested to failure in a standard ‘quick test’ which took about four hours to perform. The other two were subject to a ‘slow test’ in which increasing loads were applied over about a six month period until the nails failed. The original intention of these tests was to show whether or not the grout to ground bond was lower under the slow application of load than with the normal quick test. It was possible that in the partly cohesive soil, negative porewater pressures might be generated by the movement of the nail during testing. In a quick test these would increase the nail’s pull-out capacity by increasing the effective stress acting on the grout/ground boundary. In a slow test, taking several months, any negative pressures would have time to dissipate and would therefore give a more accurate measure of the long-term pull-out resistance of the nails. However, in all four tests it was found that the nail failed by rupture of the GRP and not by pull-out from the ground. Thus no conclusions could be drawn regarding the grout to ground bond, except to confirm that it was significantly higher than the value assumed in design.

The stress distribution along the nails was generally similar to that found in the tests on Scheme A (Section 3.2.3.3), but the stresses reduced even more quickly in the tests on the GRP nails. In test ‘Quick 2’ a load of 124kN was maintained at the nail head but no strains were registered over the bottom 2m of the nail.

The nails ruptured at the point where the plastic thread was moulded onto the body of the GRP tube. In the two slow tests the GRP failed at much lower values than in the two quick tests, see Table 7. Also given in this table are the pull-out values estimated using the bond assumed in design (10kN/m length) and also those calculated in accordance with Section 2.27 and Appendix D of HA68 (DMRB 4.1). For these calculations the following were assumed:

- Effective nail length, L_e = 4.1m
- Borehole diameter, d_{hole} = 68mm
- Best estimate of the peak angle of friction, $\phi'_{peak} = 35^o$
- Cohesion, c'_{des} = 0kN/m^2
- Grout/ground interface sliding factor, α = 1.0
- Soil unit weight, γ = 21kN/m^3.

Table 7 Comparison of the results of quick and slow pull-out tests

Type of test	*Mean depth of cover over nail (m)*	*Estimated pull-out based on ultimate bond resistance assumed in design (kN)*	*Calculated pull-out resistance based on HA68 (kN)*	*Load at tensile failure of the nail (kN)*
Quick 1	2.24	41	20.2	131
Quick 2	2.79	41	25.2	130
Slow 1	2.29	41	20.7	92
Slow 2	2.92	41	26.4	84

Because the measuring system employed in the tests did not provide instantaneous load values the failure loads reported in Table 7 (131kN, 130kN, 92kN and 84kN) were calculated as the mid-value between the highest load carried satisfactorily and the next load being attempted (see Table 8). It is of interest to note that the simple estimation of pull-out resistance for these nails based on 10kN per metre length of nail gave a value of 41kN while the HA68 methodology but using unfactored ‘best estimate’ parameters listed above produced a mean value of some 23kN (for a mean cover depth of 2.6m). The actual failure values obtained were significantly higher than either of these with a mean of 131kN for the quick tests and 88kN for the slow tests. As all four nails failed by rupture of the nail rather than pull-out from the ground one can only say that the grout to ground bond must have been greater than these failure values. The measured loads in the quick pull-out tests were over three times those estimated in design and in the slow tests were over twice the estimated values in the design. The measured loads in the

Table 8 Loads applied during slow tests

Time since installation (days)	Test Applied load (kN)	Slow 1 Applied load as a percent -age of short-term ultimate load	Test Applied load (kN)	Slow 2 Applied load as a percent -age of short-term ultimate load
0	30	25	25	21
42	42	35	40	33
73	53	44	51	43
110	71	59	72	60
160	83	70	77	64
194	100 attempted	83 attempted	90 attempted	75 attempted

quick pull-out tests were nearly six times those calculated using the HA68 calculation (and the unfactored 'best estimate' of parameters) while the slow test loads were nearly four times greater than the HA68 calculated values.

It is of interest to compare the pull-out resistances calculated according to HA68 (Table 7) with those determined in the design check (Appendix B). In Table B3 of Appendix B, a nail with an effective length of 5.77m and a mean cover depth of 2.67m produces a design pull-out (P_{des}) of 31.63kN. Factoring this to find the design pull-out of a 4.1m long nail at this depth gives a P_{des} of 22.5kN. The mean pull-out resistance for the four 4.1m test nails in Table 7 at a mean cover depth of 2.56m (calculated using HA68) is 23kN. While on first inspection this appears to provide excellent agreement between the pull-outs calculated by the two methods, this result is rather surprising since the mean HA68 value of 23kN in Table 7 uses unfactored 'best estimates' of the various values while the Appendix B calculation includes a number of partial factors which one might expect to produce a lower pull-out. A possible reason for this anomaly is that the Appendix B calculation interprets the intention of BS 8006:1995 to be that the load factor f_{fs} of 1.5 should be applied to the soil mass when calculating the pull-out resistance (as well as for calculating the disturbing force).

An important feature of these tests is that the moulded thread or the GRP nail ruptured at a mean load of 88kN in the slow tests compared with 131kN for the quick tests (Table 7). This shows that some mechanism of strength reduction with time was operating. Further details of the loading regime are given in Table 8. As stated above, the failure loads were taken to be the mid-point between the maximum load carried and the final load attempted.

While GRP is not subject to electro-chemical corrosion its strength can reduce with time due to a phenomenon known as stress corrosion. A sample of new GRP will exhibit a higher short-term ultimate tensile strength than a similar, but aged sample. More importantly, a sample of GRP subjected to a constant tensile load for some time will have a much lower strength. This would appear to be borne out by the tests reported in Table 7.

Previous investigations of the long-term behaviour of GRP recommended that the maximum design working load for GRP straps should be taken as about 10% of the short-term ultimate strength (Mallinder, 1979; Greene and Brady, 1994). One GRP product was assessed and granted a British Board of Agrement Roads and Bridges Certificate (No 83/25, since lapsed) for use as soil reinforcement. In this certificate the maximum recommended strengths of the straps for a design life of 120 years was specified as 9% of the short-term tensile strength.

Using the 10% value for the available long-term strength, the GRP nails would have a maximum working load of 31kN based on the manufacturer's claimed short-term ultimate tensile strength, but only 16kN based on the manufacturer's claimed ultimate strength of the thread. Applying a 90% reduction to the results of the two quick test given in Table 7 would lead to a maximum design load of 13kN. The mean failure load of 131kN from the quick tests (all failures occurring at the nail/thread interface) appears reasonably consistent with the manufacturer's claimed ultimate strength of 160kN. One would expect a higher strength to be obtained from carefully controlled laboratory conditions compared with site testing where loads may not have been applied truly axially. A reduction of 90% in the thread strength could be overly conservative since the thread is primarily a plastics material and may not be subject to the same strength reduction mechanisms as the nail body. The manufacturer claimed an ultimate breaking load of 160kN for the thread arrangement and a working load (assumed long-term) of 100kN.

4 Design considerations

This section considers the elements of the design of a soil nail wall. It is based primarily on BS8006:1995 and on the experience of the two schemes described in Section 3 as well as other case studies examined during the project.

4.1 Professional roles

There is currently no widely accepted detailed method for designing soil nailing for retaining walls. Thus a designer will have to apply geotechnical expertise and engineering judgement at various stages throughout the design process.

For highway schemes undertaken for the Department of Transport and equivalent Overseeing Departments the technical approval procedures given in Departmental Standard BD2 (DMRB 1.1) must be employed. Under this procedure, the level of geotechnical input should ensure a satisfactory design.

However, 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 variables in the design may be dependent on the method of construction. In such situations it is imperative that the specialised sub-contractor is aware of the Departmental requirements for soil nailed structures 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 to be the case where relatively inexperienced design agents and soil nailing contractors are carrying out design on highway schemes. 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 of the nailing works.

General advice for the construction of new soil nailed structures is provided in BS8006:1995 and BD70 (DMRB 2.1). These give guidance on soil strengths, loads and partial material and load factors. The British Standard also provides some information on design principles although this is given more in the context of reinforced earth than soil nailing. Parts of HA68 (DMRB 4.1) may also be helpful to the designer particularly those sections relating to the calculation of the pull-out capacity of nails.

For the repair and strengthening of existing structures, BS8002:1994 would appear to be the most appropriate standard for use in conjunction with BD21 (DMRB 3.4.3) and BA16 (DMRB 3.4.4). The methodology for the assessment of substructures, which is also relevant to reinforced soil walls and abutments, is set out in BA55 (DMRB 3.4.8). Parts of BS8006:1995, BD70 (DMRB 2.1) and HA68 (DMRB 4.1) have information relevant to the design of nailing employed to strengthen the wall.

4.2 Environmental constraints

At an early stage the designer will consider the suitability of various options such as a conventional reinforced concrete wall, embedded wall, reinforced earth or a nailed structure. These options will be considered in the framework of existing Departmental Standards for the design of highway structures. With soil nails there will be little or no control over the nature and consistency of the soil into which the nails are inserted. For any structure, but particularly for a nailed structure the soil strength properties and porewater pressure regime are of prime importance. In addition for a nailed structure the corrosion potential of the soil is important. The nature and consistency of the soil and the inclusion of cobbles and boulders or buried obstructions such as foundations or sub-structures might preclude the use of soil nails as a technical or economic option.

For new works the soil must be sufficiently self supporting to permit the construction of benches (typically 1m deep) while the facing and nail are constructed. In addition a soil nailed wall will be constructed from the top downwards and adequate room is required for mechanical plant to form benches, to remove excavated material and to install the nails. In certain situations the installation of soil nails may be prevented by wayleaves imposed on tracts of land to protect underground services and pipelines or sub-structures and foundations.

The method of repairing an existing retaining structure will depend on several factors such as the type of structure, extent and form of damage or deterioration, ease of access, type of backfill and proximity of structures and buried services. In circumstances where the existing structure is in need of repair because of the onset of instability, consideration will need to be given to methods that do not further reduce stability. It is possible that some driving techniques could cause disturbance to walls which are already in an unstable state and thus drilling and grouting may be the preferred method. Care must be taken with both the drilling and the grouting processes. If casing is required for the hole this will increase the difficulty and cost of drilling. Generally grouting pressures are kept as low as reasonably possible commensurate with using fairly thick grout to minimise excessive grout loss and the use of fairly narrow bore tremie tubes (typically 10mm to 15mm internal diameter). It is important that the designer makes a clear assessment of likely porewater pressures and the drainage techniques necessary to keep them at an acceptable level. Further advice on drainage is given by Murray (1993).

Repair using soil nailing will be feasible if the structure is generally intact. Soil nailing may be the only reasonable solution if the structure has moved, short of complete reconstruction. Ideally, to avoid loss of strength with displacement, soil nailing repairs would be best carried out when some early indication of movement becomes apparent. On the wall strengthening scheme described in Section 3.3 the upper metre length of nail was left ungrouted so that some pre-load could be applied to the wall. These nails could be considered a hybrid nail/ anchorage and have the advantage of applying some restoring force to the wall without the need for any movement of the wall or soil.

4.3 Preliminary sizing and layouts

Table 19 and Section 6.4.1 of BS8006:1995 give advice on the minimum reinforcement length for the design of various structures based on the concept of *mechanical height*. These dimensions are relevant to reinforced earth structures and are not directly related to soil nails; in particular the guidance does not allow for inclined nails. Furthermore, the soil properties and the water regime for the *in situ* soil or fill to be nailed could be considerably more demanding than those within the imported material of a reinforced earth structure. The lengths of nails can thus be much greater than the equivalent elements in reinforced earth structures.

Table 9 provides some information on sizing and reinforcement layout based on a review by Bruce and Jewell (1987) of successful soil nailing works for structures having face angles of 80 degrees or more; more recent data from soil nailing schemes investigated by TRL have also been included in Table 9.

As can be seen from Table 9 the data collected from TRL studies suggest that designs are considerably more conservative, in terms of nail length and effective bond area, than the earlier studies reported by Bruce and Jewell (1987). One reason for this may be that the later schemes relate to structures on the trunk road network and as such are subject to a more demanding specification and loading regime.

Table 9 Preliminary indication of sizing and nail layout

Parameter[1]	*Drilled and grouted nails*[2]	*Drilled and grouted nails*[3]	*Driven nails*[2]
L/H	0.5 - 0.8	1.1 - 1.5	0.5 - 0.6
CL/S_vS_h	0.9 - 1.8	0.6 - 3.5	1.8 - 3.3
A/S_vS_h	0.3×10^{-3} - 0.6×10^{-3}	0.2×10^{-3} - 3×10^{-3}	1.0×10^{-3} - 1.4×10^{-3}

[1] *L is the nail length*
H is the effective retained height allowing for any over-excavation
C is the characteristic circumference of the hole in which the nail and grout (if any) is placed
A is the characteristic cross-sectional area of a nail
S_v S_h are the vertical and horizontal spacings of the nail
[2] *From Bruce and Jewell (1987).*
[3] *Data from recent TRL studies.*

In the Clouterre (1991) report nails are divided into two categories:

- closely spaced nails usually driven into the soil and
- more widely spaced nails generally installed by drilling and grouting.

In the former, where vertical and horizontal spacings are up to one metre, nail lengths are about 0.5H to 0.7H (where H is the retained height of the wall). For the more widely spaced grouted nails, the lengths are typically 0.8H to 1.2H. Juran et al (1990) also provides a method for preliminary sizing and spacing based on tests to failure of model structures.

Excavation in front of a nailed wall for services, etc. will increase the effective height of the wall and some additional allowance should be made for this. There is no guidance provided on this aspect in extant Departmental Standards. Information is available, however, in BS8002:1994 which is applicable to retaining structures and should therefore be appropriate to a soil nailed retaining structure. The standard states that a minimum depth of unplanned excavation in front of a wall should be:

- not less than 0.5m and
- not less than 10% of the total height retained height for cantilever walls, or of the height retained below the lowest support level for propped or anchored walls.

Table 20 of BS 8006:1995 provides advice on the depth of embedment to avoid local failure and piping; the required depth will vary according to the ground slope and bearing pressure but the minimum should be 0.45m. Clouterre (1991) recommends that for long-term structures the embedment should be *'never less than 0.20m for rocky soil and 0.40m or H/20, whichever is the higher value, in a soil'*.

A preliminary design may be developed based on earlier soil nailing projects in similar ground conditions or advice from Engineers or contractors experienced in the technique. This design must be checked (and refined) to ensure its adequacy for the specific project requirements.

4.4 Design parameters

4.4.1 Design soil parameters

The selection of the values of the soil variables for design of permanent structures requires careful judgement by a geotechnical engineer experienced in the interpretation of site investigation data and the selection of design values. The soil 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.

BS8006:1995 recommends the use of design strengths based on peak strength parameters (c'_{peak} and ϕ'_{peak}) for walls. Section 2.5 of the Standard 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 16 of BS8006:1995) the design soil strength is not reduced below the worst credible value. The use of peak values appears to be sensible for a retaining structure where soil displacements are controlled to satisfy serviceability limit state criteria and the depth of weathering and softening of the soil should be limited by the structure itself. Farrar and Murray (1993) suggested that the mobilized 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.

For both Schemes A and B, a design cohesion, $c'_{peak} = 0$ was adopted. This seems a reasonable approach given the generally granular nature of the soils. An assumption of zero cohesion in the long-term seems appropriate for most soils unless there is strong evidence to the contrary.

The adoption of undrained soil parameters together with a lumped factor of safety (the approach adopted in BS8081:1989) is a simple design methodology but it is not recommended for soil nailed structures, because of the difficulty in evaluating their undrained strength, particularly locally to the ground/grout interface. The approach given in BS8081:1989 is based on many years experience with ground anchorages but there is much less experience of nailed structures in the UK.

4.4.2 Loading

Section 2.4 of BS8006:1995 advises that dead and live loads should be calculated as 'raw loads' and then the appropriate partial factor, as given in Tables 16 and 17, applied to give the design load. Table 17, however, indicates that the factor f_{fs} should be applied to the mass of the reinforced soil body. In the original design for Scheme A a load factor (γ_{fL} from BD37, DMRB 1.3) was applied throughout the calculations. Similarly in the design checks a load factor (f_{fs} from BS8006:1995) has been applied to the soil mass for all the calculations. Where f_{fs} is greater than unity this 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 BS8006:1995 are meant to be used as a package and are intended to provide reasonable designs.

Highway live loading for DOT structures is defined in BD37 (DMRB 1.3). The partial factors given in BS8006:1995 are generally consistent with this Departmental Standard.

4.4.3 Ground water

Porewater pressures can substantially affect the stability of a nailed retaining wall. Higher porewater pressures require a higher restoring force to maintain the stability of a potential failure wedge. Also positive porewater pressures reduce the effective stress acting on the nail/ground interface along which pull-out resistance is generated. Both effects increase the nail length required to maintain stability over those required for an identical structure with lower porewater pressures.

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 on this subject is obtained during the site investigation for the works. As a starting point it should be assumed that porewater pressures and the phreatic surface remain unchanged immediately following the construction of a nailed wall, consistent with the advice in Padfield and Mair (1984) for embedded retaining walls in stiff clays.

Methods for including porewater pressure effects in an analysis are rather imprecise. An appropriate value for the porewater pressure parameter (r_u) may be estimated and included in an analysis. Positive 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. The design check of the new wall given in Appendix A includes a comparison of the restoring force required (T_{max}) for the cases of $r_u = 0$ and $r_u = 0.1$ to provide an indication of the sensitivity of the analysis to porewater pressure.

Alternatively the designer may assume a groundwater profile or flow net. 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 by estimating the porewater pressure, and hence the effective stress, at various locations. The results obtained through such an approach could again be regarded as only an approximation to what will pertain in service.

Because reinforced earth is built from the bottom up it is possible to incorporate drainage layers within the structure with little difficulty. Furthermore, the frictional fill will have a permeability higher than that of many natural soils. Thus a reinforced earth structure is unlikely to exhibit high positive porewater pressures and it can be assumed that this was taken into account in drafting Table 19 of BS 8006:1995 for initial sizing of reinforcement in a wall. The presence of negative or low positive porewater pressures is much more difficult to ensure in a nailed structure in natural ground.

For both Schemes A and B, the designers took the view that positive porewater pressures would not occur during the service lives of the structures, and both designs incorporated measures to ensure this would be the case. At Scheme A, the site investigation indicated that the level of the water table in the gravelly material was several metres below the toe of the wall. During construction, pipes were placed against the excavated face before shotcreting took place; these were subsequently joined into a carrier pipe and a back-up system of low level weep holes through the final facing was also included.

At Scheme B, it was known that the generally granular material behind the existing wall was reasonably free draining. While much of the masonry face had been pointed at some earlier time, weep holes drained water through the wall at regular intervals. A thick, viscous grout for the nails was specified and controlled partly to ensure that excessive volumes were not ‘lost’ through the voids in the structure and partly to ensure that existing drainage paths through the wall were not blocked.

Generally drainage should be incorporated in any soil nailed wall. As a minimum this should comprise provision for drainage through the face; this could involve filter and drainage layers, porous pipes and/or weep holes. Additional drainage measures penetrating into the existing ground may be considered necessary, for example inclined drilled drains (Bruce and Jewell, 1987). Cut-off drains in the retained soil above the wall may also be of value and care should be taken in the detailing of any surface water drainage system to minimise the likelihood of surface water entering the soil. 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 (1993).

4.5 Method of analysis

For new construction works, the basic design philosophy is to identify, for the completed structure, the failure plane which generates the largest out-of-balance force (T_{max}). A trial nail array is then assumed and is checked to ensure it can develop sufficient restoring force to maintain stability. Both Schemes A and B followed this approach and draft BS8006;1991 and HA68 (DMRB 4.1) were the main documents employed.

An alternative (and possibly more rigorous) approach for strengthening existing walls is given in the design check in Appendix B. In this the existing wall is assessed for stability in accordance with BA55 (DMRB 3.4.8) and associated documents. The additional restoring force to maintain stability is then calculated in accordance with BS8002:1994. Finally a trial nail array is checked to ensure it can generate sufficient force to maintain stability (using BS8006:1995). The main difficulty with this approach is that the various British Standards, DOT Standards and Advice Notes do not provide consistent advice on loads, soil properties and partial factors for design. The designer is required to make a number of judgements on the choice of these parameters where no simple, single interpretation is available.

4.5.1 Internal stability

Both design examples examined used a limit equilibrium approach to calculate the total nail tensile force required to maintain stability of a single wedge. Essentially this method of analysis requires the identification of the failure surface which provides the largest out-of-balance force, T_{max}, at the ultimate limit state. An array of nails is then developed with a length in the resistant zone to provide sufficient restoring force to stabilise the wall. The failure surface is assumed to be in the form of a simple wedge passing through the toe, consistent with the two-part wedge analysis of Section 7.5.5.3 of BS8006:1995 simplified into a single wedge for vertical walls.

The designer needs to decide the installation angle (δ) of the nails. Generally an installation angle of 10^o to 20^o downwards will be suitable as this permits grout to be tremied around the nail (if necessary) but should also ensure that when the failure wedge starts to move the nail will quickly develop tension. In all calculations the T_{max} force is the restoring force necessary applied at the nail angle, δ.

For Scheme B, in the original analysis of the strengthening required, the critical failure surface was taken as a simple Coulomb wedge dependent only on the effective shear strength of the soil, ϕ'_{peak}. For Scheme A, the original analysis for the new wall employed the rather more complicated approach given in BS8006:1995 in which the nail angle to the horizontal is also taken into account ($\delta = 20^o$). This produced a critical failure surface at 69^o to the horizontal and an out of balance force (T_{max}) of 441kN/m run of wall (Section A.6 of Appendix A of this report). A calculation was also made assuming an r_u value of 0.1 which produced a critical wedge angle of 71^o and a T_{max} of 522kN/m run.

If the simple Coulomb wedge approach, as used in Scheme B, had been used on Scheme A it would have produced a critical wedge angle of 65^o and a T_{max} of 261kN/m run of wall for Scheme A using the unfactored value of $\phi'_{des} = \phi'_{peak} = 40^o$ in both calculations. Alternatively the factored value of $\phi'_{des} = 30^o$ produces a wedge angle of 60^o and a T_{max} of 422kN/m run of wall. Both of the above are for a nail angle $\delta = 20^o$ and a porewater pressure coefficient r_u of zero.

The next stage is to develop a nail layout to provide the necessary restoring force. Vertical spacing is generally determined by the stability of vertical benches for nail installation (often 1m). The critical height, h_c, of a vertical bench for stability 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

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

The necessary restoring force is usually calculated for a unit horizontal length of wall, and various horizontal spacing and nail lengths are tried until the layout provides sufficient additional restoring force to stabilise the wall. On both schemes examined the design called for a constant nail length to be used throughout the works. Where possible constant nail lengths have generally been used on most schemes to simplify installation operations. There is, however, no technical reason for this and the designer may specify more than one nail length if he wishes.

Typical horizontal and vertical spacings are of the order of 1m to 2m. HA68 (DMRB 4.1) recommends a maximum horizontal and vertical spacing of 2m for nailed slopes. For walls, the designer must judge the most suitable layout bearing in mind the stable height of construction benches, restoring force required per unit length of wall and the strength of the facing or wall to support point loads applied at the nail head. It will be possible to provide the same restoring force from a larger number of small nails (both length and diameter) or a smaller number of large nails.

For Scheme A, the new wall design, the nail pull-out resistance was calculated in accordance with Appendix D of HA68 (DMRB 4.1). For the wall strengthening scheme (Scheme B) pull-out resistances were calculated on a basis of 10kN per m length of nail as derived from the results of preliminary tests. Another approach to calculating pull-out resistance is given in Section 2.12 of BS8006:1995, although this is more applicable to flat horizontal strip reinforcement than nails. BS8006:1995 also provides the tie back wedge method for checking the local tension in individual reinforcements (Section 6.6.4.2.1): this method was also given in BE3 (DMRB 2.1) but it would appear to be more applicable to reinforced earth. For soil nails, a simple check could be made that the total pull-out which could be generated by the nails exceeds the out of balance force. For Scheme B, however, the masonry wall facing might be considered as comprising individual blocks attached to reinforcement elements. The calculation of the ultimate nail force required based on local stability of layers may therefore be valid and this methodology has been used in the design check given in Appendix B.

The Clouterre report (1991) states that, given the present state of knowledge, limit equilibrium methods using potential failure surfaces are the recommended approach for soil nail structures and that limit equilibrium methods only provide the total nail tension for an ultimate limit state. They cannot be used to derive estimates of movements, or how the forces are shared between individual rows of reinforcement for the serviceability limit state. Similarly, for the case of strengthening an existing structure, the methods do not provide an indication of how the forces are shared between the wall face and the reinforcements.

4.5.2 Shape of failure surface

The design approach for vertical walls suggested in this report, and the one followed in the two designs examined, is to assume that the failure surface approximates to a single plane passing through the toe of the wall. This is consistent with one of the methods provided in BS8006:1995 (Section 7.5.5.3). Alternative failure planes which have been proposed for reinforced structures include the two-part wedge, logarithmic spiral, circular and other forms of curvilinear surfaces. It should be appreciated that these are idealised surfaces and it is not possible to identify one as being the definitive 'correct' surface. In some cases it is possible that a broad failure zone rather than a thin failure surface would be developed.

One major advantage of assuming a single wedge failure plane passing through the toe is that of simplicity of analysis. For Scheme B, the wall strengthening, the critical failure mechanism was taken to be a simple Coulomb wedge whose position was dependent only on the unfactored ϕ'_{peak} of the soil (see Section 3.3.1.2). For Scheme A, the new construction, the critical failure plane was based on Section 7.5.5.3 of BS 8006:1995 - the two part wedge simplified to a single wedge (see Appendix A of this report). This latter analysis includes the inclination of the nail and thus the designer had to select what value to use (in Scheme A this was 20^o). Calculations are repeated for a range of wedge angles to the horizontal (θ) until the largest out-of-balance force (T_{max}) is found. Some comment on the different results from the two methods is given in Section 4.5.1 above.

A designer must decide whether to use the 'best estimate' of ϕ' or the design value ϕ'_{des} when calculating the position of the critical failure wedge. There are many aspects to be considered. For a simple Coulomb wedge analysis it could be argued that the best estimate would provide a better estimate of the likely failure plane (as long as $\phi'_{best\ estimate}$ was determined as a long-term value). Where the method given in BS8006:1995 Section 7.5.5.3 method is employed, it might be more appropriate to use the ϕ'_{des} value since here it is being used to calculate the inherent resistance to movement of the soil wedges.

There are innumerable possible failure planes, and the analysis identifies the one which requires the greatest restoring force (T_{max}) to maintain stability. It is then assumed that the resistant zone and failure wedge behave as rigid bodies. As mentioned above, movement may occur within a 'failure zone' rather than along a discreet failure plane . The rear edge of such a failure zone may be located further back than the 'design' failure plane. Thus the effective length of the nail (L_e) resisting pull-out may be rather less than that assumed in design, but there is probably enough conservatism in other aspects of a design to balance this. However, where calculations indicate that a designer's first estimated nail layout generates more restoring force than is required, it is suggested that first consideration is given to increasing the spacing or decreasing the diameter of the nails rather than decreasing their lengths.

For slopes, as opposed to vertical walls, both BS8006:1995 and HA68 recommend a two part wedge analysis. It is not possible to give a definitive face batter at which the simplicity of the single wedge is outweighed by the increased accuracy of the two part wedge analysis. For walls inclined at 85^o, and possibly 80^o, the single wedge is likely to be satisfactory. At 60^o or 70^o the bi-lineal failure plane is likely to be more appropriate. The single wedge approach generates a triangular failure block while the two wedge analysis produces one which tends towards a rectangle. The two part wedge analysis is more complicated because the designer has to search for the critical failure mechanism by trial and error. The relative applicability and economy of the two approaches is further complicated where the designer employs nails of a constant length rather than providing nails of various lengths based on L_e, the effective lengths in the resistant zone.

Computer programs to help identify different failure mechanisms are available, and are often helpful to the designer in allowing a variety of situations to be explored. A difficulty with these procedures, however, is that the analytical basis and associated assumptions are not necessarily entirely valid; moreover the limitations of the applicability of the computer program may not be clear. A further difficulty is that different interpretations may apply to the design variables and factors of safety used in different programs. While the results obtained from one of these programs, by a skilled user, may be entirely satisfactory, it is important that a designer develops an understanding of the problem and the influence of the individual factors on its solution.

Where soil nailing is to be used to strengthen an existing wall, the existence of cracks in the wall or in the surface of the retained material, may provide evidence of the location of the failure plane.

4.5.3 External and overall stability

For any type of earth retaining wall, the designer must check the external stability of the structure against sliding, overturning, foundation failure or deep seated failure. On occasions with nailed walls, some nails may need to be lengthened to ensure that one or more of these external failure modes does not occur.

BS8006:1995 indicates that the stability check should be made on the block of reinforced soil (as if it were a thick gravity wall). For new construction where the nails are installed on a fairly close spacing this way of viewing the structure seems reasonable. However, where an existing wall has been assessed and found to require only a few widely spaced nails (or possibly anchorages) then the concept of the whole soil block acting as a monolith is less applicable.

With regard to overall stability, conventional methods of slope stability analysis are considered appropriate. However differences of approach in different codes require judgement in selection of soil strength parameters and suitable factors of safety.

4.5.4 Soil/nail interaction

The ability of a nail to generate sufficient pull-out resistance is of fundamental importance to the performance of a nailed structure. For reinforced earth BS8006:1995 and the earlier BE3 (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 appears to be an effect sometimes termed 'constrained dilation' where the soil has to dilate 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, and other effects in the soil, measured pull-out values for strips are almost always greater than those calculated.

When nails are installed in natural ground there may be additional complicating factors. Nails are more likely to be used in clayey soils and thus porewater pressures are more likely to be a problem. Where nails are installed by a displacement technique, such as firing, they will tend to increase the normal stress in the soil surrounding the nail which may increase 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 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 in Section 2.23 and Appendix D of HA68 (DMRB 4.1). Normally, pull-out tests are carried out at the start of the works to confirm that measured pull-out values are higher than the expected values. As can be seen in Sections 3.2.3 and 3.3.3 of this report, both Schemes A and B produced pull-out test results significantly higher than those calculated. Because the exact behaviour of a nail is not fully understood, especially in the long-term, in neither scheme was the nail length or layout modified to take advantage of these higher pull-out resistances. At the present state of knowledge and experience it is recommended that this approach is maintained because of the difficulty of guaranteeing these higher pull-out resistances. However, where a large number of pull-out tests on a scheme give consistently higher values than those expected, an experienced designer may wish to re-examine the analysis and uprate the pull-out resistances of the nails. However, uprating should be done with caution: the long-term performance of the nails must be assessed. Further discussion on the interpretation of pull-out tests is given in Section 4.11.

4.6 Partial factors

In a limit state approach to design, partial factors should be related to the level of uncertainty associated with a variable or behaviour mechanism. 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 BS8006:1995 does not follow this philosophy. The value of the partial factors are based on a calibration exercise which was aimed to give similar layouts to those obtained using earlier design methods. In addition, since the new standard deals primarily with reinforced earth and as there were no accepted design methods for soil nails the calibration exercise was based on reinforced fills; it did not include *in situ* reinforcing techniques. Thus the partial factor values given in BS8006:1995 may not be the same as those arrived at by a designer through engineering judgement and experience.

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

- The partial load factors contained in Tables 17 and 18 of BS8006:1995 are in accordance with the partial factors given in Table 1 of BD37 (DMRB 1.3).
- Lower values of the partial factors for external loads are given in Table 26 of BS8006:1995 (relating to slopes) than in Tables 17 and 18 (relating to walls). This probably reflects the greater consequences of failure for a wall than a slope.
- There have been different interpretations as to the requirements of BS8006:1995 regarding the factor f_{fs} to be applied to the soil mass. One interpretation is that it should be applied to all calculations. The other interpretation is that it is unrealistic to apply it to the calculation of pull-out resistance (where it would provide a greater normal stress and pull-out resistance than could be justified theoretically).
- A lower partial factor (of unity) for soil mass is applied in HA68 (DMRB 4.1) than is given in Table 26 of BS8006:1995 while a higher factor is applied to the peak angle of shearing resistance ϕ'_{peak} (both relating to slopes).
- The partial factor value for pull-out of 1.3 given in BS8006:1995 is considerably lower than the factor of safety on pull-out of 3 defined in BS8081:1989. This is not surprising since the former is a partial factor while the latter is a global one.

Some of the partial factors for the two schemes examined were derived from the draft BS8006 (1991). However in Scheme A a value of 1.2 was used for the partial factor on soil mass rather than 1.5. Soil strength was factored by an f_{ms} of 1.45 on Scheme A, and by an f_{ms} of 1.5 in Scheme B, compared with an f_{ms} of 1 recommended in BS8006:1995.

For nailed walls, where no large movements are expected, it would appear reasonable to base the design soil strength on peak values ϕ'_{peak} and c'_{peak} (with c'_{peak} generally taken as zero). For consistency with BS8006:1995, the factor f_{ms} should be applied (generally unity to ϕ'_{peak} and 1.6 to c'_{peak}). Where the value of the factors differ from those provided in Tables 16 and 17 (for walls) it will be necessary to assess all the partial factors since, as mentioned above, the values given in BS8006:1995 are a 'package' which are meant to be used in combination.

As mentioned above, one particular inconsistency is that some designers read the intention of BS8006:1995 as applying the partial factor for soil self weight (f_{fs}) to just the disturbing effects of the soil while others applied it throughout the analysis (thereby providing an enhanced pull-out resistance).

The choice of partial factors becomes more complicated where an existing wall is to be strengthened. For the assessment of stability, BA55 (DMRB 3.8.4) provides values for certain factors and if strengthening is required BS8002:1994 recommends a different set for calculating the required stabilising force. If nails are to be used to provide stability then BS8006:1995 gives a third set of factors. Various assumption are used in the design checks in Appendices A and B: while these may be considered a satisfactory design checks, other approaches and assumptions may be equally valid.

4.7 Soil nail layout

4.7.1 Nail spacing and length

At Scheme A the vertical spacing was determined largely by the stability of the excavation between the layers of nails. At Scheme B the vertical spacing was determined by engineering judgement to control the likelihood of bulging of the existing wall. In both schemes the horizontal spacing was determined to provide sufficient restoring force per metre run of the wall. Both schemes employed a uniform length of nail to meet the design pull-out requirements and for convenience of construction. Scheme A installed nails on a regular grid while Scheme B employed a diamond pattern.

In theory, there are many possible combinations of nail spacing and length which satisfy the requirement for internal stability, namely the sum of nail pull-out in the resistant zone and the sum of nail strengths are each greater than the required restoring force of the critical failure surface. Soil reinforcing systems have scope for redistributing the load between elements and if the wall is relatively stiff, e.g. where an existing wall is being strengthened or if a stiff facing is provided, then this would act to redistribute loads. But to limit excessive movement and to prevent overstress of one layer of reinforcement which could lead to progressive failure the designer should consider the local balance between restoring and disturbing forces:

- BS8006:1995 states that the adherence capacity of each layer of reinforcement should be compared with the local force to be resisted. However, as discussed in Section 4.5.1, this appears to relate primarily to the use of horizontal reinforcement in fills attached to small facing units.
- HA68 (DMRB 4.1) provides rules for 'optimising' the vertical spacing of nails in slopes by varying the spacing of the nails throughout the slope.

HA68 (DMRB 4.1) recommends for nailed slopes a maximum vertical spacing of 2m, and that the horizontal spacing should not exceed the vertical spacing . It is suggested that this should also be the maximum spacing for nailed walls, and such spacings a wall or facing sufficiently strong to spread the loads between the nails should be provided. As spacings increase the nails becomes more like ground anchorages in that a relatively small number of anchors each provide a large resisting force.

All methods of designing retaining walls require checks on external stability (sliding, overturning and bearing capacity failure) and overall slope stability where applicable. Longer nails (at the top or bottom of the structure) may be required to satisfy external and overall stability than are required to satisfy internal stability. The T_{ob} mechanism given in HA68 (DMRB 4.1) is a useful means of checking the basal sliding of the reinforced block.

HA68 (DMRB 4.1) recommends the checking of potential mechanisms beyond the assumed 'critical' failure mechanism, 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 BS8006:1995 a designer might wish to do so, particularly if the original design minimised costs by reducing the nail lengths to a bare minimum.

4.7.2 Nail orientation

Reinforced earth structures are built with horizontal reinforcement but soil nails are usually installed at a downward angle to the horizontal; this is essential for grouted nails to ensure a good grout bond to the soil. The angle of inclination has a number of inter-related effects on the behaviour of the nailed structure and hence on its design.

Firstly, if the nail is taken to act solely or primarily as a tensile element then it should be installed at an angle such that the smallest incipient movement of the critical failure wedge would develop tension. One theoretical approach to this would be to install the nail parallel with the critical failure plane. Obviously, this orientation is not feasible because none of the nail would lie in the resistant zone of the soil and therefore would not provide any pull-out resistance. (An alternative way of considering the development of tension for approximately horizontal reinforcement is that when the active block starts to move a wedging action forces the nails into tension. However, should movements occur over a wide failure zone rather than a discreet failure plane, fairly large movements might be necessary to develop significant nail tension.) Should reinforcement be installed approximately at right angles to the critical failure plane they would tend to act as dowels, in bending, and a different design procedure would be more appropriate.

For an unreinforced (or horizontally reinforced) wall a unique horizontal out-of-balance force (T_{max}) can be calculated. For progressively steeper inclined reinforcements (nails) the angle of the critical failure wedge changes (generally producing a steeper wedge with a lower self weight). For each nail orientation, the critical failure plane can be identified as the plane having the largest out-of-balance force in the same orientation as the nails, $T_{max\delta}$, where δ is the nail declination to the horizontal. As shown in Appendix A following the selection of a value for δ (20^o in this case) the designer must search for the largest $T_{max\delta}$ and design his nail layout to balance this. Figure 6 gives an indication of the change in T_{max} for a range of nail inclinations for a typical wall. The curves show a clear trend of increasing T_{max} with increasing nail declination for each soil strength. However, the curves should not be extrapolated to significantly steeper nail angles since the mathematics become unstable when the nail tension is not applied in the approximately correct direction.

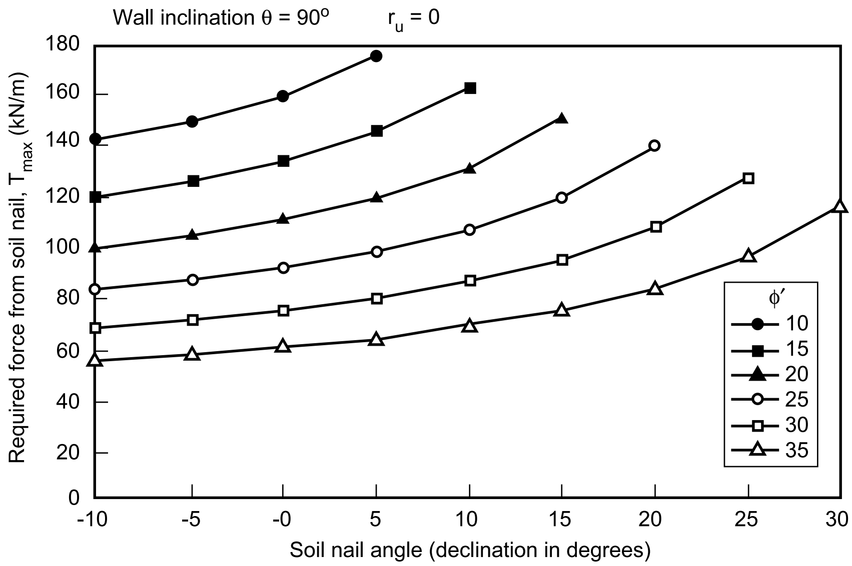

Figure 6 Variation of T_{max} with nail installation angle

The restoring force which nails can generate may affect the performance of the nailed slope by two different mechanisms (although this is not explicitly considered in soil nail design). The component of nail tension acting parallel to the failure plane directly opposes the tendency to slide down the plane while the component normal to the failure plane generates increased friction to resist movement by increasing the normal stress across the plane. For a specified failure plane and a fixed T value it can be shown that the maximum restoring effect (the sum of the direct and frictional resistance) occurs when the nail crosses the critical failure plane at an angle of φ' (the soil shearing resistance) to the normal to the failure plane. While this may be the theoretically optimum angle of installation it is normally not the one which has most influence on practical design.

Pre-loaded ground anchorages (BS8081:1989) are usually installed to cross potential failure surfaces at right angles and thereby aid stability by increasing the normal stress (and hence friction) across the failure plane. Nails are essentially passive reinforcements and cannot normally be pre-loaded significantly. Thus the approach followed in this report is that the nails should be installed at an angle at which they quickly generate tension (theoretically parallel to the failure plane, but in practice at a slight declination to the horizontal).

An additional complicating aspect is that after a required $T_{max\delta}$ has been calculated, the generation of pull-out resistance is dependent on the mean normal stress (thus cover depth) over the resistant part of the nail. Thus pull-out resistance can be increased by increasing the angle of installation of the nails.

Because of the complexity and interaction of the various factors it is recommended that designers should adopt an installation angle, δ, of between 10° and 20° unless they have strong reasons for a different approach.

4.7.3 Hole diameter

It would normally be assumed that greater surface area, and thus larger borehole, provides a greater pull-out resistance. At Scheme B the hole diameter used in the works was greater than that use in site trials, thus the works would have had a slightly higher ‘factor of safety’ than assumed in design but this may also have increased the costs of the scheme. Although this caused no difficulties in this instance, it illustrates the importance of specification and quality control procedures to ensure the hole diameter bored on site is the same as (or greater than) the diameter assumed in design. As boreholes increase in diameter (say, above 300mm), reinforcements are more able to provide a dowelling action and this aspect of their behaviour may need to be considered in design.

4.8 Movement

Compared to a conventional reinforced concrete earth retaining structure, a soil nailed wall or reinforced earth wall is much more flexible. Deformation (either during or after construction) is a requirement of a soil nailed wall in order to mobilise tension in the nails and reach a state of equilibrium. Soil nailed walls are not appropriate therefore in situations where large movement of the wall and/or retained soil cannot be tolerated during the service life of the structure. However, the use of pre-tensioned ground anchorages are likely to be a suitable technique for controlling movement. Some schemes have used a combination of nails and anchors (Clouterre 1991, Figures 31, 32 and 33) to try to obtain the benefits of both techniques. The hybrid nail/ground anchor approach used in Scheme B would tend to minimise wall movements.

A soil nailed wall should comply with the following requirements:

1 Sufficient flexibility to allow the structure to deform and mobilise tension in all nails but sufficient rigidity to permit load sharing in the nails. The normal method of

construction from the top down in benches, will tend to permit greater movement at the top of the wall. This, in turn, will tend to generate greater mobilised tensions in the upper nails compared with those lower in the structure. Thus, while calculations may indicate greater pull-out capacity lower in the wall (greater effective length and greater overburden) in practice larger tensions may be generated in the upper layers due to the larger movements.

2 Wall deformations should satisfy the serviceability limit state acceptance criteria. Deformation of the wall may lead to settlement of the ground behind the wall which may, particularly in urban areas, result in movement of adjoining structures and buildings. The Clouterre report (1991) indicates that lateral displacements at the top of a soil nailed wall can vary between 0.1% and 0.4% of the retained height.

3 For new structures, during successive excavations of the bench in front of the wall, the retained soil is subject to both lateral stress relief and settlement. As a result, at the end of construction, a slight tilting of the facing occurs where vertical and horizontal displacements are at a maximum at the crest of the wall. Allowance for this displacement is normally made by tilting the wall backwards by one or two degrees.

Typical movements of soil nailing structures are given in Table 10. Calculated displacements are included in the design check given in Appendix A. It is not normally practicable nor warranted to undertake a complete numerical analysis, for example using finite elements. Therefore it is suggested that the likely movements are evaluated on the basis of published data (such as Table 10), and compared with the tolerable movements. The relations given in Table 10 are based on the Clouterre report (1991) and are typical rather than definite because many factors can influence displacements on an individual site. For this reason, and given the present level of experience of soil nailing, it is recommended that nails are not used for bridge abutments or in situations where very large or dynamic loads apply.

The distance λ back from the facing at which deformations become negligible is given by :

$$\lambda = F H (1 - \tan\beta)$$

where:
H = perpendicular height of the structure
F = empirical constant given in Table 10
β = initial angle of inclination of the face relative to the vertical.

Woodward (1991) suggests that to minimise the deformation in excavations stabilised by soil nailing the cut depth at each stage of excavation should be kept small, say 1m to 1.5m, 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 soil nailed structures indicates that deformation often continues after the end of construction (Juran and Elias, 1991; Kakurai and Hori, 1991; Woodward, 1991) even in cohesionless soils. The deformation is particularly sensitive to climate and freezing effects (Juran and Elias, 1991) and such effects should be taken into account in design. The Engineer or Client may wish to carry out some post-construction monitoring of a soil nailed structure in certain circumstances.

Table 10 Typical movements in soil nailing structures (after Murray, 1993)

Vertical or horizontal deformation	*Coarse sand/ gravel*	*Sand*	*Clay*
$\delta_v = \delta_h$[1]	H/1000	2H/1000	4H/1000
δ_o		4H/1000 to 5H/10000	
F	0.8	1.25	1.5

[1] *See Figure A2 in Appendix A for definitions of symbols.*

4.9 Wall facing

This section does not cover the detailed design of the facing for a nailed wall but provides discussion on some of the relevant points. According to BD70 (DMRB 2.1) only hard facings are permitted for permanent structures. The required restoring force is generally calculated for a unit horizontal length of wall but the distribution of force on the back of the wall is not considered. If the nail array is able to provide sufficient total restoring resistance (T_{max}) to balance the driving forces and maintain stability, the design is taken to be satisfactory. This may be taken to imply that the facing should be capable of distributing the load among the nails.

The advice given in BS8006:1995 is that the facing should be designed to accommodate soil pressures corresponding to the reactions in the connections, which themselves are forces corresponding to between 75% and 100% of active pressure, depending on their elevation in the wall and the design method. The Clouterre report (1991) recommends that the facing is designed on the basis of a uniform pressure corresponding to the maximum tension which can be mobilised in the nail. This value is multiplied by a factor of between 0.5 and 1.0 depending on the spacing of nails.

When considering retaining walls, a designer will generally assume that the lateral earth pressure acting on the wall increases with depth. The designer may consider that the failure mechanism has some similarity to a circular slip. In both cases this would result in a greater restoring force being required nearer the bottom of the wall. For a constant nail length, the lower nails should be able to generate a higher pull-out resistance because of their greater length in the resistant zone and the higher normal effective stress due to overburden. However, in practice the resistance provided by a nail will be the sum of any pre-load induced during construction plus additional tension generated by incipient movement of the soil during service. Because of the uncertainty regarding the distribution of forces among the nails the simple approach of providing an overall balance between the restoring force and driving force seems reasonable at the present time.

For new construction, one approach is to cut the top bench and place reinforcing mesh about 50mm to 75mm away from the vertical face. Where nails are to be installed, the mesh is cut away and a short length of 150mm plastic pipe (blocked with rag) located through the mesh. The face is then shotcreted to a depth of about 100mm to 150mm and the nails subsequently installed through the holes provided by the plastic pipe.

The nails are normally grouted (or mortared) flush with the front of the shotcrete and allowed to harden before a retaining plate and nut are fitted and tightened. The next bench is excavated and shotcreted in the same way (with overlapping mesh to ensure continuity). Any movement of the top section of facing during the excavation of subsequent benches will generate a small but undefined load in the top row of nails. As the wall progresses some of its weight is supported by bending in the nails, some by friction between the soil and the wall and some by the soil below the bottom of the shotcrete. The designer might wish to overdig the bottom bench of a wall to provide a wider shotcrete 'foundation'. However, it will be difficult to quantify the proportion of facing weight carried by the nails, by friction and by the foundation. A final facing in masonry, cast concrete simulating stonework or other finish may be applied where the shotcrete finish is not acceptable. This second skin may be purely cosmetic or may be designed to add to the structural performance of the wall.

The use of shotcrete would appear to be the most practical approach to new construction. While it might be possible to install individual panels perhaps 1m square before or after nail installation it would be almost impossible to ensure they were in good contact with the soil face. When the second bench was excavated the top row would tend to move, probably inducing more bending in the nails than for a shotcrete face. There would be difficulty in aligning panels with those already installed and also with inserting dowels to connect adjacent panels. The use of a single, full height panel would again be theoretically possible through the construction of a diaphragm wall with subsequent excavation and nail installation: this would be a costly and difficult operation.

Where nails are used to strengthen an existing wall, the designer must make an assessment of the competence of the existing facing. Where it has little rigidity and additional stiffening is not possible then a larger number of low capacity nails will be appropriate. In Scheme B the existing wall was constructed of natural stone blocks of various sizes. The nail spacings were adjusted to ensure that they were installed through the larger blocks. These blocks were cored with 150mm diameter holes about 150mm deep before the 68mm boreholes for the nails were drilled. After installation and tightening of the nails the cores were trimmed and mortared back into the blocks to produce an 'invisible' repair.

4.10 Durability

In common with the requirements for other Department of Transport structures, the required service life of a new soil nailed retaining structure will normally be 120 years (BD70, DMRB 2.1). Where the following discussion relates to wall strengthening it is assumed that the service life for the nails and the strengthened structure are 120 years from the time of strengthening. However, this may not necessarily be the case and where a shorter service life is specified the advice should be modified accordingly. For both new and strengthening works some corrosivity assessment must be made of the soil and/or fill to determine the suitability or otherwise of the nails. Table 4 of BS8006:1995 gives limits on suitable fill to be used in reinforced earth construction. This table has been amended by BD70 (DMRB 2.1) to make it applicable to natural soil as well as to fill, and also to delete the option of using ungavanised steel for reinforcement. Where soil nailing is proposed for soils outside the corrosivity limits given in Table 4, BD70 (DMRB 2.1) calls for a separate evaluation of soil aggressivity using an approach such as that given in TRL's RR 380 *The development of specifications for soil nailing* (Murray, 1993).

The first difficulty which a designer may encounter is that unless soil nails (or possibly reinforced earth or corrugated steel buried structures) were considered at an early stage (perhaps at the desk study) the site investigation will probably not have included the tests required by Table 4 of BS8006:1995. Thus the designer will be unable to assess whether the natural soil falls within the corrosivity limits given in Table 4. 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 likely to be more difficult to provide a comprehensive assessment of material to be nailed than for excavated fills to a reinforced earth structure. The soil to be nailed could be under a considerable depth of cover at the site investigation stage and deep boreholes might be needed to obtain samples of the soil.

The aggressivity assessments given in Table 4 of BS8006:1995 and Tables 3 and 4 of RR 380 (Murray 1993) require similar sets of tests. Strictly, according to BD70 (DMRB 2.1) the test regime in BS8006:1995 should be followed. If the soil is less aggressive than the limits set in Table 4 (BS8006:1995), galvanized steel or stainless steel nails may be used and sacrificial thicknesses for a 120 year design life in these conditions are given in Table 7 of BS8006:1995. Examples of suitable materials are shown in Table 6 (BS8006:1995 as amended by BD70). BD70, Section 3.2.1 restricts the permissible steels to only those complying with the British Standards listed in Table 6 of BS8006:1995 or those holding a current BBA certificate. While certificates have been issued for strips and anchors for use in reinforced earth construction, at the time of writing, no such certificates have been issued for soil nails.

Steels other than those listed in BS8006:1995 Table 6 may be suitable for soil nails especially for the repair of retaining walls where a 120 year design life may not be required. Soil nails are not normally highly stressed and steel of a relatively low or medium tensile strength may often be used satisfactorily (eg Grade 250 or Grade 460 of BS4449:1988 *Carbon steel bars for the reinforcement of* *concrete*). However, designers or contractors may specify higher strength steels, in bar or tube form, such as used in rock bolting or pre-stressing applications. Very high tensile steels can suffer from problems of weakening from hydrogen embrittlement, sometimes associated with the acid pickling process prior to galvanising. Generally such problems relate only to extremely high tensile steels (ultimate tensile strength greater than, say, 100N/mm^2). The Contractor should provide evidence to satisfy the Designer and the Engineer that the steel to be installed is satisfactory for the application.

Steel for galvanizing should have a silicon content which readily permits a zinc coating weight of not less than 1000 g/m^2 (Section 3.2.2 of BD70). Alternatively, a high coating thickness may be achieved by grit blasting and pickling prior to galvanising; grit blasting may be sufficient on its own and this would reduce still further the possibility of problems of hydrogen embrittlement. Experience has shown that austenitic stainless steels (as given in Table 6 of BS8006) are suitable but ferritic stainless steel is unsuitable, because of its tendency to pit in the presence of chloride ions.

Should the soil be more aggressive than permitted by BS8006:1995 then further assessment such as that given in RR 380 (Murray 1993) may be necessary. This latter document divides soils into four categories; non-aggressive, mildly aggressive, aggressive and highly aggressive, using the data given in Eyre and Lewis (1987). It recommends that permanent nailed structures should not be constructed in highly aggressive soils. For the three remaining categories, Table 5 of RR 380 provides annual rates of galvanizing loss. For an initial coating weight of 1000g/m^2 (140 microns) these equate to galvanizing 'life' of 35 years in non-aggressive conditions, 18 years in mildly aggressive conditions and 10 years in aggressive conditions. Figure 2 of RR 380 provides a plot of the required sacrificial thickness of the underlying steel against service life for the three conditions. In practice the corrosion of both the galvanizing and the underlying steel is unlikely to be uniform but the guidance given in these figures is probably the best currently available. To comply with BD70 the substitution of the galvanizing by an additional sacrificial thickness of steel is not permitted (amendment to Table 7 of BS8006:1995 by BD70). The corrosion resistance of materials other than galvanized steel is not covered in RR 380.

An alternative approach permitted in Section 3.2.2.2 of BS8006:1995 is for nails to be protected in accordance with the recommendations for corrosion protection in BS8081:1989. For the two schemes examined, the designers' intention was to specify nails which were essentially non-corroding or at worst only slowly-corroding. Neither scheme included specific aggressivity assessments. For the new build scheme, the basic design of the nails includes some small corrosion allowance since the calculated rupture strength (eg Appendix A of this report) employs a partial factor f_m of 1.5 relating to material intrinsic properties, construction and environmental effects (BS8006:1995 Annex A Section A.2) so some (unspecified) sacrificial thickness was present. The nails were galvanized (although not to the enhanced coating thickness required by BD70 (DMRB 2.1). They were factory grouted into impermeable corrugated plastic sheaths which were grouted on site into a 140mm borehole. BS8081:1989 suggests that 'double protection' is required to reduce the possibility of corrosion to a negligible level. This can be achieved using an impermeable corrugated sheath filled with polyester resin or two concentric sheaths filled with grout (Figures 19 and 20 of BS8081:1989). While the single sheath employed on Scheme A would provide one protection layer for the nails, the sacrificial steel, galvanizing and two layers of cement grout would all improve the corrosion protection. Although no aggressivity assessment was made at the site, the sandy, gravelly conditions present would normally be associated with a non-aggressive or mildly aggressive material.

At Scheme B, the wall strengthening, no formal aggressivity assessment was made. However, one of the specified nail materials was stainless steel type 316S33 which listed in Table 6 of BS8006:1995. The calculated rupture strength of the nail at 152kN (Appendix B, Section B.4.3) is much higher than the 30kN maximum strength required (Appendix B, Table B2). This calculation included the material factor f_m of 1.5 which partly covers environmental effects. Using these figures the minimum nail diameter required is 11mm compared to the 25mm employed in practice. This could be considered as providing a 7mm sacrificial corrosion allowance on the radius of the nail. If the natural soil was non-aggressive and fell within the limits of Table 4 of BS8006:1995 then a sacrificial corrosion thickness of 0.1mm would be needed (Table 7 BS8006). Thus, although difficult to quantify, it could be argued that the actual 7mm sacrificial thickness could provide protection in more aggressive soils.

The alternative nail permitted in the strengthening works, and the one actually employed, was a glass reinforced plastic (GRP) tube having an outside diameter of 22mm and an inside diameter of 12mm. The specification called for a tensile strength of 310kN per nail. Corrosion protection was not covered in the original design document and it is assumed that the designer either considered GRP to be non-corroding or that the 310kN specified strength was greater than the calculated working load of 22.4kN by a sufficient margin such that the residual strength after any degradation would be satisfactory. As discussed in Section 3.3.3.4 of this report, while GRP does not corrode due to electrochemical effects it may be subject to a significant reduction in strength primarily through the mechanism of stress corrosion. Earlier reports (Mallinder, 1979; Greene and Brady, 1994) recommend that the long-term (120 year) working strength of a GRP reinforcement should be taken as 10% of its short-term ultimate tensile strength. For the nails employed on Scheme B this would give a working load in the anchor of 31kN and a working bolt head load of 16kN (based on the manufacturer's quoted breaking load of 160kN).

4.11 Interpretation of pull-out tests

Pull-out tests were conducted at both schemes. On Scheme A, twelve nails were tested at the start of the works and some 30 permanent nails were proof tested during the works. Where failure occurred at the grout to ground interface this was achieved at an applied load about seven times greater than that calculated (Section 3.2.3.1). On Scheme B, five nails were tested before the design was carried out with the design pull-out resistance being based largely on these test results. A further nine tests were done at the beginning of the works with some 50 proof tests on permanent works nails during the construction. Apart from some early tests where the quality of the grouting technique was suspect, high pull-out resistances were obtained. In quick tests 1 and 2 (Section 3.3.3.4) the grout to ground bond exceeded the calculated pull-out (to HA68) by a factor of five or six, when nail rupture occurred. Pull-out testing is generally carried out on most nailed structures and nailed earthworks schemes and some advice on pull-out testing has been provided by Murray (1993).

The relationship 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 exactly 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 slippage occurs along a defined failure plane rather than being spread over a wide failure zone (Section 4.5.2) although the latter may be more realistic in some cases.

Generally the designer will be interested primarily with the resistance to pull-out generated behind the potential failure surface. One approach to estimate this is to test a nail grouted along its full length and calculate 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 could 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 close to the face 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 generally important, but especially so with the testing 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 in length 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 designated active wedge enabling measurement to be made of the pull-out generated in the resistant zone. Possible disadvantages of this apparently more realistic method are that the soil stress regime is not as it would be on the point of wedge movement, 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 this should provide a more realistic test.

An effect which is sometimes observed in granular soils, especially with rough or ribbed reinforcement, is that of constrained dilatancy (Schlosser, 1979; BS8006:1995 Section 2.12). As a nail or other reinforcing strip starts to move, adjacent soil particles have to slide or roll over one another. They are prevented from moving readily by the constraint of 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 mechanisms which can produce higher pull-out test results compared with those calculated, and these may be considered either as further features of constrained dilatancy or as separate mechanisms. One concerns non-straight boreholes (either curved or dog-leg) and another concerns fissured or non-homogeneous ground, leading to some grout to soil mechanical interlock. All of these may contribute to higher pull-out resistance through a complex amalgam of mechanisms.

These effects may also be present, but perhaps to a lesser extent in cohesive soils. However, the short-term pull-out resistances of nails installed in clay may be higher than those attainable in the long-term since porewater pressures during construction (and testing) could be lower than those experienced during the service life of the structure. This approach would be consistent with the concept of using the undrained shear strength parameter (c_u) for short-term soil behaviour (say the excavation of benches) but the effective stress parameters, ϕ' and c', when considering 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 Schemes A and B gave higher values than those calculated using the design equations given in HA68 (DMRB 4.1). 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. For nailed earthworks measured pull-out resistances usually exceed the calculated values as well. On one slope stabilizing scheme low pull-out resistances were attributed to the tremie not reaching to the bottom of the borehole. 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 weak rock 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 an excessive water/cement ratio this could further wet and weaken the chalk. The grout might also weaken through segregation and shrinking away from the side of the borehole. In practice, the actual pull-out resistances should always exceed the design values.

The method given in Section 2.27 of HA68 (DMRB 4.1) for calculating pull-out resistance indicates that the design pull-out resistance is directly proportional to the effective length (L_e) of the nail. This implies that a uniform pull-out resistance is generated per unit length of nail. This is unlikely to be the case, but it may be approximately correct for a high strength nail at the point 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 (see Figure 3); the rate 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 greatest shear stress. With increasing applied 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'_{cv} and even to one related to $\phi'_{residual}$ (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'_{cv} and possibly to $\phi'_{residual}$. 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 stress threshold has been exceeded along the whole nail length and failure occurs.

This progressive development of pull-out resistance is also likely to occur at the ultimate failure condition of a nail in a structure except that the maximum load will be developed at the failure plane as the wedge moves. At the working condition it is unlikely that there will be a uniform stress condition along the nail and thus the design assumption of 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 readily be employed in the design process and thus the current approach in BS 8006:1995 and HA68 should continue to be used.

5 Conclusions

There is no single document which contains a detailed, comprehensive design method for the use of soil nails for retaining walls. However, the advice given in a number of documents, allied to good geotechnical input, can result in satisfactory designs. It should be accepted that as this is a relatively new and complex technique there are some uncertainties in design. The following items summarise the factors that must be taken into account to provide a successful design.

The design of a soil nailed structure requires a high level of geotechnical expertise. This input can be ensured for highway schemes undertaken in accordance with DETR technical approval procedures. If soil nailing is submitted as a contractor's alternative, design should be undertaken using parameters consistent with those used for the design of the main works and subject to the same rigorous checks (Section 4.1).

A soil nailed structure is feasible only in soils where a stable temporary vertical soil face can be excavated and reasonably high pull-out resistances can be achieved. Soil nailing is 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 (Section 4.2). 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 for retaining walls supporting bank seats, for bridge abutments or in situations where very large cyclic or dynamic loads might apply.

An initial layout of a soil nail array may be obtained from Table 9 (Section 4.3). Where the natural soils would prove suitable as fill to reinforced earth structures, Table 19 of BS8006:1995 may be helpful for initial sizing. Often the properties of natural soils and *in situ* porewater pressures will mean that nails will need to be longer than typical reinforced earth straps. Uniform spacing and nail length have normally been used, but may vary according to economy of construction and the extra cost of more complicated site installation practice. The design height will be rather more (perhaps 1m) than the retained height to allow for embedment (BS8006:1995) and unplanned excavation (BS8002:1994). The maximum height of self supporting benches may influence the spacings of the nails for new construction.

The dimensions and layout of the soil nails are sensitive to the strength of the soil. The determination of realistic long-term soil strength parameters c'_{peak} and ϕ'_{peak} is therefore critical if a safe and economic design is to be achieved (Section 4.4.1). A small cohesive strength is very beneficial to the stability of a structure but it is difficult to predict, with confidence, the long-term value for c'. Designers have tended to take c' as zero and it is recommended that this is taken as a starting point unless there is strong evidence to the contrary.

Porewater pressure is an important factor affecting the stability of a nailed retaining wall (Section 4.4.3). 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 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. Where high positive porewater pressures are anticipated then a technique other than soil nailing may be more appropriate. Generally drainage measures should be included in new wall construction or strengthening works incorporating nails.

The basic method of analysis is to identify the failure plane which generates the largest out-of-balance force (T_{max}) for the completed structure. For vertical walls a reasonable approximation is to assume the failure plane approximates to a single plane passing through the toe of the wall (Section 4.5.2). A preliminary nail array is then checked and modified as necessary to ensure sufficient restoring force is developed to satisfy stability at limit equilibrium (Section 4.5). The nail installation angle (δ) has a significant and complex effect on the design.

Changing the nail orientation affects:

- whether the reinforcement behaviour tends towards axially loaded nails or dowels working in bending
- the angle of the critical failure wedge (a relatively small effect)
- the value of the maximum force required for stability ($T_{max\delta}$) in the direction of the nail - see Figure 6
- the average overburden and hence the pull-out resistance

For ease of grouting and speedy generation of tension with soil movement it is suggested that designers take δ as 10^o to 20^o unless they have strong reasons to do otherwise.

For a vertical wall, the assumption that the failure plane approximates to a single wedge passing through the toe appears to be generally accepted, and the calculation of the critical failure wedge is fairly straightforward. For inclined walls the rather more complicated twin wedge analysis is more appropriate (Section 4.5.2).

While most discussion on soil nailing relates to internal stability, checks must also be made for sliding, overturning, foundation failure and deep seated failures. Analyses may be undertaken assuming the whole nailed block to be the wall or just the wall facing (or original wall in the case of strengthening). Where a high density of nails is used (close horizontal and vertical spacing) then the former appears more appropriate while if only a few widely spaced nails are used the latter seems better (Appendix B, Section B.6.1). On occasions, nails may need to be lengthened to ensure stability against external failure modes (Section 4.5.3).

The values of the partial factors have a significant effect on the nail layout. BS8006:1995 provides a ‘package’ of partial factors. Where different factors are employed it is necessary to check that all the partial factors are consistent with the design approach (Section 4.6).

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 wall or facing redistributes any locally high loads. The local stability calculation of Section 6.6.4.2.1 of BS8006:1995 indicates that higher pull-out resistances are required (and therefore provided) by the lower nails. Observed deflections (Clouterre, 1991; Murray, 1993) show that greater deflections (and possibly greater tensions) are developed in the upper nails. With this uncertainty regarding the distribution of forces and assuming some redistribution will be possible, the simple approach of ensuring that the total nail force exceeds the total out-of-balance force appears reasonable. Rather than failing along a discreet failure plane, soil movements may occur over a broad failure zone. 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 is increased (if appropriate) rather than the nail lengths reduced.

Some deformation of a soil nailed wall is required to mobilise tension in the nails (above any small tensions developed during construction) and to reach a state of equilibrium. Soil nailed walls are not appropriate, therefore, in situations where some movement of the wall and/or retained soil cannot be tolerated during the service life of the structure. Typical movements of a soil nailed structure are given in Table 10 (Section 4.8).

The required service life of a new soil nailed retaining structure for DETR is 120 years (BD70, DMRB 2.1). For repair or strengthening works to existing structures, the service life for the nails and the structure may or may not be 120 years from the time of strengthening. For both new and strengthening works, some corrosivity assessment must be made of the soil and/or fill to determine the suitability of the material used for the nails (Section 4.10). All the necessary information to make a definitive judgement may not be available during the design and assumptions and simplifications may have to be made to finalize the design. Unless the aggressivity of the ground can be confidently assessed as low, the designer may need to specify ’double protected’ nails (to BS8081:1989) or some other ‘non-corroding’ material.

Trial pull-out testing is commonly carried out before, at the beginning of and during the nailing works. The interpretation of these results is not straightforward (Section 4.11). 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 may consider increasing the design values.

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Volume 1: Section 1: Technical approval

BD2 (DMRB 1.1) *Part 1. Technical approval of highway structures on motorways and other trunk roads.*

Volume 1: Section 3: General design

BD37 (DMRB 1.3) *Loads for highway bridges. Use of BS 5400: Part 2.*

Volume 2: Section 1: Substructures

BE3 (DMRB 2.1) *Reinforced and anchored earth retaining walls and bridge abutments for embankments (Revised 1987). Now superseded by BS8006:1995 as implemented by BD70 (DMRB 2.1).*

BD70 (DMRB 2.1) *Strengthened/reinforced soils and other fills for retaining walls and bridge abutments: Use of BS8006:1995*.

Volume 3: Section 4: Assessment

BA16 (DMRB 3.4.4) *The assessment of highway bridges and structures.*

BD21 (DMRB 3.4.3) *The assessment of highway bridges and structures*

BA55 (DMRB 3.4.8) *The assessment of bridge structures and foundations, retaining walls and buried structures.*

Volume 4: Section 1: Earthworks

HA43 (DMRB 4.1) *Geotechnical considerations and techniques for widening highway earthworks.*

HA68 (DMRB 4.1) *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques.*

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Appendix A: Design check of a new retaining wall (Scheme A)

A.1 General design philosophy

The original design employed a draft version of BS8006 (dated 1991) *Draft code of practice for strengthened/ reinforced soils and other fills,* BD37 (DMRB 1.3) *Loads for highway bridges*, and BS8081:1989 *Code of practice for ground anchorages*. These documents provide only limited advice on the methods of analysis and design for soil nails, and a number of subjective decisions need to be made. These documents have also been used in this check (including the published version of BS8006:1995). In addition guidance has been taken from BD70 (DMRB2.1) *Use of BS 8006:1995*, TRL Research Report RR380 *The development of specifications for soil nailing* (Murray, 1993) and HA68 (DMRB 4.1) *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques*.

The philosophy adopted is one of limit state design using partial factors appropriate for ULS or SLS conditions. The limit state conditions to be considered are shown in Table A1 and the geometry of the wall is given in Figure 2 of the main report.

Table A1 Design limit states

Limit state	*Internal stability*	*External stability*
Ultimate (ULS)	yes	yes
Serviceability (SLS)	no	yes (wall movement only)

Generally, checking the ULS condition for internal and external stability should be sufficient for design. It may, however, be necessary to check the wall movement under the working loads to ensure they meet specification for vertical and horizontal wall deflection and/or ground movement within the retained soil. For both internal and external stability there are two cases to consider.

1 The completed structure when all nails are installed.

2 During construction when the installed nails are required to support the additional load of an unsupported face. The worst case occurs at the bottom of the wall where the total load of the retained soil is carried by the nails above the last row. For this condition it is appropriate to use a soil strength that reflects short-term conditions together with any temporary surcharge loads applied by the construction plant.

A.2 Design soil parametere

In accordance with BS8006:1995 a worst credible value of peak soil strength (c'_{peak} and ϕ'_{peak}) are used in conjunction with material factors of f_{ms} = 1.6 and 1 respectively for ULS and unity for SLS to derive design values. The peak soil strength parameters derived from the results of the ground investigation are shown in Table 3 of the main report. It appears that ϕ'_{peak} = 40° was the best estimate of the peak soil strength and this was reduced to ϕ'_{des} = 30° to give the worst credible value as required by

BS8006:1995 Section 5.3.4. In the medium to coarse gravel the assumption that $c'_{des} = 0$ is appropriate.

For construction, higher soil strengths could be used to reflect short-term conditions. In this example a material factor $f_{ms} = 1.2$ was adopted on $\tan\phi'_{peak}$ so that $\phi'_{des} = 35°$. In cohesive soils it may also be appropriate to incorporate some cohesion, c'.

A.3 Applied loads

In accordance with Section 5.8.2.1 of BD37 (DMRB 2.1) the nominal HB loading of $16kN/m^2$ (equivalent to 37.5 units of HB for principal roads) is applied as the live load surcharge on the wall giving $W_{LIVE} = 16kN/m^2$. For construction, a minimum uniform surcharge of $W_{LIVE} = 10kN/m^2$ is used, again in accordance with BD37 (DMRB 2.1). For the self weight of the soil it was assumed that the bulk density, $\gamma_{fill} = 21kN/m^3$.

The applied loads are factored according to Table 17 of BS8006:1995 which gives $f_{fs} = 1.5$. This partial load factor is rather more onerous than those in BD37 (DMRB 2.1) which give $\gamma_{fL} = 1.2$ for vertical loads. This is the main reason for the higher T_{max} value in this design check compared with the value calculated in the original design.

A.4 Preliminary design

Section 3.2.2.2 of BS8002:1994 allows for unplanned excavation in front of the wall and a figure of 1m is often used. Section 6.4.2 of BS8006:1995 requires some embedment depth to avoid punching failure and soil flow under the wall. Thus the design height, or *mechanical height*, used for calculation is 7m while the retained height of soil is 6m.

The initial choice of nail type, length, diameter, inclination, and corrosion resistance was based on published case histories of previous schemes, advice from contractors and engineers who had experience of soil nailing and the designer's own engineering judgement. These choices were confirmed or refined as the design progressed.

Drilled and grouted nails were chosen since they had to be installed into dense gravel. A constant nail length was chosen for the whole site to simplify installation and site control. The length of 7m is equivalent to the *mechanical height*. 25mm diameter high tensile steel reinforcing bar was selected as a readily available and suitable material.

A 140mm diameter borehole was selected on the basis of readily available drilling equipment. The vertical nail spacing of 1m was selected as a convenient height of excavated bench that was known to be stable when cut vertically. The horizontal spacing of 1.5m was related to the required restoring force per metre run of wall.

A.5 Method of analysis

The dimensions of the retaining wall are shown in Figure 2 of the main report. With a soil nailed structure it is not possible to check overall stability until internal stability has been satisfied and the length and spacing of the soil nails has been established. This is different from the methodology in BS8006:1995 which determines the preliminary size for the soil reinforced mass based on the retained height; this reinforced mass is then checked for external and internal stability.

A.6 Internal stability for ULS condition

A.6.1 Completed structure

Internal stability is determined using the two part wedge method of analysis as described in Section 7.4.4.2 of BS8006:1995 to determine the critical failure mechanism. However, for a vertical wall the critical failure mechanism corresponds to a single planar surface. For soils with $c'_{des} = 0$ the two -part wedge equation simplifies to:

$$T = [(W+Q)\tan(\theta - \phi'_{des}) + U(\sin\theta - \cos\theta.\tan(\theta - \phi'_{des}))]/[\cos\delta - \sin\theta.\tan(\theta - \phi'_{des})] \qquad (A1)$$

where:

$$W = 0.5H^2\gamma f_{fs}/\tan\theta \qquad (A2)$$

$$Q = f_q W_{LIVE} H/\tan\theta \qquad (A3)$$

$$U = r_u W/\cos\theta \text{ (with } r_u = 0 \text{ for this design)} \qquad (A4)$$

and:

T	=	required restoring force per unit length of wall
W	=	weight of the potential failure wedge per unit length of wall
Q	=	live load surcharge per unit length of wall
U	=	force due to porewater pressure acting on the base of the wedge
θ	=	angle of the potential failure wedge to the horizontal
δ	=	angle of declination of the soil nails
ϕ'_{des}	=	design effective angle of friction (= ϕ'_{peak})
H	=	design retained height = 7m
γ	=	bulk density of backfill = $21kN/m^3$
f_{fs}	=	partial factor on soil mass = 1.5
W_{LIVE}	=	highway surcharge load = $16kN/m^2$
f_q	=	partial factor on surcharge load = 1.5

In general, single wedge analysis for a vertical wall provides a reasonable approximation to the actual behaviour (Section 4.4.2 of the main report). Also, the single wedge analysis with the failure passing through the toe is a fairly simple calculation which can readily be undertaken by hand. When using the two-part wedge analysis it is necessary to search for the T_{max} mechanism and there is a risk that it may not be identified if the wrong assumptions are made for the geometry of the two failure surfaces.

Equation A1 is of a similar form to that given for ground anchorages in BS8081:1989 except that in this example the equation has been modified to allow for highway surcharge load, W_{LIVE} and pore water pressure through the introduction of r_u.

From Equation A1 the value of T (per horizontal metre run of wall) is calculated for the critical failure plane defined by the angle θ. In the as built design no allowance was made for a build up of pore pressure because of the provision of adequate drainage. Notwithstanding this, it is good practice to check the sensitivity of the design to the localised build up of porewater pressure during the design life of the structure. For comparison purposes values of T for $r_u = 0.1$ have also been computed. The results are shown in the Table A2.

Table A2 Determination of T for various values of θ

Angle of failure plane, θ, degrees	*T, kN/m run of wall* $r_u = 0$	$r_u = 0.1$
55	393.28	459.90
60	422.04	491.36
65	438.21	512.44
69	441.18	521.03
70	439.72	521.85
71	437.88	522.06
75	421.31	515.84

For $r_u = 0$, $T_{max} = 441$kN/m run of wall is required for a critical failure plane of $\theta = 69°$ whereas for $r_u = 0.1$, $T_{max} = 522$kN/m at a critical failure plane angle of 71°. In the original design a horizontal nail spacing (S_h) of 1.5m was chosen and thus the equivalent T_{max} that must be resisted by the nails is given by:

$S_h \times T_{max} = 662$kN per 1.5m run of wall (for $r_u = 0$)

The value of T_{max} must be provided by the pull-out resistance of the soil nails. To simplify the construction process by having a constant excavation depth and to provide an approximately uniform resisting load to the wall it is sensible to provide nails at a uniform vertical spacing. Given that the retained height of the wall is 7m, seven soil nails were located at 1m vertical spacing commencing at 0.5m below the crest of the wall.

For the case of $r_u = 0$ and $S_h = 1.5$m the required pull-out resistance of each nail (P_{res}) is given by:

$$P_{res} = T_{max}/7 = 662/7 = 95\text{kN} \qquad \text{(A5)}$$

This assumes that the disturbing force T_{max} is uniformly distributed over the height of the wall. For soil nailed walls comprising a hard facing (e.g. sprayed concrete face) this is a reasonable assumption since there is the possibility of load shedding of high local forces to adjacent nails. P_{res} is provided by the bond resistance for a single nail along the effective length (l_i) of nail behind the critical failure plane, as shown in Figure 2. The total available resisting force per unit length of wall (T_{res}) is the summation of the bond resistance of each nail as follows:

$$T_{res} = \Sigma P_{res} = \Sigma \pi d l_i (a\sigma_n \tan'\theta_{des}) / f_p \qquad \text{(A6)}$$

where: d = diameter of nail = 0.14m

l_i = effective length of the i_{th} nail behind failure plane

= $L_T - \{(H - D_i)\cos\theta/\sin(\delta + \theta)\}$

D_i = depth of the i_{th} nail from the crest

L_T = total length of the nail

f_p = partial factor for pull-out resistance of reinforcement = 1.3, from Table 16 BS8006:1995

θ'_{des} = design effective angle of friction (= θ'_{peak})

a = interaction coefficient relating soil reinforcement bond = 1.0 for rough borehole.

σ_n = $\frac{1}{2}(\sigma_v + \sigma_l)$

σ_v = $f_{fs}\,\gamma\, z_i\,(1 - r_u)$ note that the contribution from the live load surcharge W_{LIVE} is not included but the partial factor of 1.5 has been applied to the soil mass in determining the pull-out resistance of the nail

σ_l = $\sigma_v K_l$

K_l = $\frac{1}{2}(1 + K_a)$

z_i = $\frac{1}{2} L_T \sin\delta + \{(H - D_i)\sin\delta.\cos\theta/2\sin(\delta + \theta)\} + D_i$

The total length of the nail may be chosen so that the individual pull-out resistance of the nail equals P_{res}. In this design example a uniform nail length of 7m was selected for ease of site control and installation by the contractor. For the soil nail distribution described above the effective length and pull-out resistance of each nail is shown in Table A3.

Table A3 Determination of effective length and available pull-out resistance

Nail row	*Depth (D_i)*	*L_T (m)*	*l_i (m)*	*P_{res} (kN)*
1	0.5	7	4.67	50.14
2	1.5	7	5.02	78.17
3	2.5	7	5.39	109.64
4	3.5	7	5.75	144.57[1]
5	4.5	7	6.10	182.93[1]
6	5.5	7	6.46	224.74[1]
7	6.5	7	6.82	270.00[1]

[1]*Available design pull-out exceeds the design rupture strength of the nail = 144kN*

The resistance to rupture of the reinforcement in accordance with Section 6.6.4.2.2 of BS8006:1995 is:

$$T_u/(f_m f_n) \qquad \text{(A7)}$$

where: T_u = ultimate tensile strength of reinforcement = $A_s\,\sigma_t$

A_s = area of reinforcing bar

σ_t = tensile strength of reinforcement taken to be 490N/mm² (from Table 6 of BS8006, although strictly this relates to strip not round bar)

f_m = partial material factor for reinforcement = 1.5 for steel

f_n = 1.1 for category 3 structure (from Table 3 of BS8006:1995)

The allowable tension (T_u) for a 25mm diameter bar is calculated to be 144kN. (Based on the ratio of thread diameter to the nominal diameter of the bar and knowing the measured thread rupture loads were about 230kN the unfactored ultimate short-term strength of the nail body was about 360kN.) In Table A3 the maximum allowable pull-out for nail rows 4, 5, 6 and 7 has been reduced to 144kN on the basis that as higher individual nail resistances are mobilized some load shedding to adjacent nails occurs. It has been taken arbitrarily that redistribution occurs before the design rupture strength is reached. While this is not a rigorous approach it is considered reasonable and possibly conservative, particularly as the nail tests suggest a short-term rupture strength of the nail body of 360kN.

From Table A3, $T_{res} = \Sigma P_{des}$ = 814kN per 1.5m run of wall (based on de-rated contribution from nails 4, 5, 6 and 7) and is thus greater than T_{max} = 662kN per 1.5m run of wall and therefore satisfies the requirements for internal stability. If the P_{res} values for nails 4, 5, 6 and 7 are not de-rated a total design resistance, ΣP_{des} = 1060kN per 1.5m run is available.

It is thought that in general with ‘top down’ construction the upper layers of nails will tend to develop tension generated by the construction process. This would be consistent with the typical pattern of wall movements in Figure A.2. Conversely, the distribution of earth pressures behind a wall would usually be taken to provide greater pressures (and nail loads) at the bottom. These two effects, plus any load shedding of higher local loads would tend to lead to generally similar nail loads at different elevations in the wall.

A.6.2 During construction

For construction conditions the calculations for T_{max} are repeated but with short-term soil strength, $\phi'_{des} = 35^o$ and temporary surcharge live loading, $W_{LIVE} = 10kN/m^2$.

From Equation A1, T_{max} = 321kN (short term) at a critical failure plane, θ of 68°. Using Equation A5 the value of T_{res} is calculated using the short-term soil strength and the nail lengths given in Table A4 to check that the first 6 nails are able to resist S_h x T_{max} (short term) = 482kN per 1.5m run of wall as set out in Table A4. The effective nail lengths (l_i) in the two tables are slightly different because the critical wedge angle is 69^o for the long-term case and 68^o for the short term.

Table A4 Value of pull-out resistance available during construction

Nail row	*Depth (D_i)*	*L_T (m)*	*l_i (m)*	*P_{des} (kN)*
1	0.5	7	4.56	58.89
2	1.5	7	4.94	91.94
3	2.5	7	5.31	129.26
4	3.5	7	5.69	170.85[1]
5	4.5	7	6.06	216.71[1]
6	5.5	7	6.44	266.84[1]

[1]*Available pull-out exceeds the design rupture strength of the nail = 144kN*

$T_{res} = \Sigma P_{des}$ = 712kN per 1.5m run, using a downrated values for nails 4, 5 and 6, or 934kN per 1.5m run, using the full values for nails 4, 5 and 6.

Using the lower T_{res} value, the factor of safety against instability is given by:

$$T_{res}/T_{max} = 1.5 \text{ (short term)} \qquad \text{(A8)}$$

This is acceptable for temporary works and the self supporting nature of the soil has been confirmed by the trial excavation of a trench 4.5m deep by 10m long by 1m wide which stood successfully for 24 hours before being backfilled. A report of this trial was included in the original design documents.

A.7 External stability for ULS condition

External stability checks are undertaken for the following, but for the completed structure only:

1 Sliding across the plane of the lowest nail.

2 Bearing capacity and overturning at the base of the wall.

3 Slip failure behind the soil nailed mass.

A.7.1 Sliding

The resistance to sliding along the plane of lowest soil nail has been analysed by considering the block of reinforced soil as a two part wedge, see Figure A1. It was assumed that shear force on the interface between the blocks was zero; this results in an overestimation of the disturbing forces. The resistance to sliding of block 1 (T_1), ignoring surcharge loading but taking account of the component of block weight along the sliding plane, was calculated from the equation:

$$T_1 = [W_1(\sin\delta + \lambda\cos\delta.\tan\theta'_{des}) - \lambda U_1\tan\phi'_{des}]/(\cos\delta - \lambda\sin\delta.\tan\phi'_{des}) \qquad \text{(A9)}$$

where: $\lambda = (ad_{hole})/(f_{s1}S_h) + (1 - d_{hole})/(f_{s2}S_h)$ to allow for reduction in soil area and partial factors for sliding along sliding plane

f_{s1} = partial factor for sliding across reinforcement layers = 1.3 from Table 16 of BS8006:1995

f_{s2} = partial factor for sliding on soil to soil contact = 1.2 from Table 16 of BS8006:1995

$W_1 = \gamma f_{fs} (D_i + L_t\sin\delta/2) L_t\cos\delta$ where f_{fs} = 1 for relieving effect

$U_1 = r_u W_1/\cos\theta$

a = interaction coefficient relating soil reinforcement bond is 1.0 for rough borehole

S_h = 1.5m, assumed from design layout

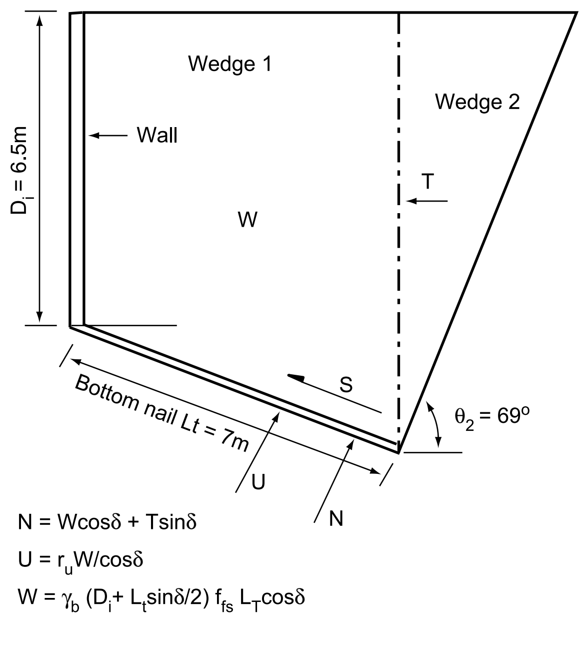

Figure A1 Sliding of structure along bottom row of nails

From Equation A9, for L_t = 7m and D_i = 6.5m, T_1 = 1901kN per 1.5m run of wall. From Figure A1, the sliding force in the horizontal direction exerted by block 2 (T_2) is calculated using Equation A1. For height of soil block, H_b = 8.89, θ = 69°, δ = 0, W_{LIVE} = 16kN/m^2, f_{fs} = 1.5, f_q = 1.5, T_2 = 1.5 x 453 = 680kN per 1.5m run of wall. The factor of safety against sliding is given by:

$$T_1/T_2 = 2.8$$

The minimum factor of safety required is 1.2 (inferred from Table 16, BS8006:1995) and thus is satisfactory.

A.7.2 Bearing capacity and overturning

The bearing pressure along the plane of lowest soil nail is calculated on the basis of factored loading applied to block 1 as follows:

Weight of block 1, W_1 = 1595kN per m, taking f_{fs} = 1.5

Surcharge load on block 1, Q_1 = 158kN per m, taking f_q = 1.5

From Equation A1, the horizontal component of force due to soil weight only T_{2a} = 415kN per m, and horizontal component of force due to surcharge only, T_{2b} = 71kN per m.

Taking moments about the centre of the base of the block:

$$e = [T_{2a}(H_b/3 - L_t\sin\delta/2) + T_{2a}(H_b/3 - L_t\sin\delta/2)]/(W_1 + Q_1) \qquad (A10)$$

Assuming a Meyerhof distribution of bearing pressure, see Section 6.5.2 of BS8006:1995

$$q_r = R_v/(L - 2e) \qquad (A11)$$

where: q_r = factored applied bearing pressure

R_v = resultant of all factored vertical loads

From Equation A10, e = 0.55m indicating that the resultant force acts within the middle third of the base and is satisfactory with regard to overturning. From Equation A11, q_r = 320kN/m^2. The allowable bearing pressure when ϕ'_{des} = 30° and applying the partial factor of safety for foundation bearing capacity, f_{ms} = 1.35, is approximately 750kN/m^2. Therefore the applied bearing pressure (q_r) is less than the allowable.

A.7.3 External slip failure

This condition was checked using a conventional slope stability analysis. BE3 (DMRB 2.1) requires that no potential failure surface exists behind the soil nailed wall with a factor of safety against instability of less than 1.5. For conventional slopes BS6031:1981 requires that the factor of safety is not less than 1.3 to 1.4 for permanent works. Neither document gives guidance on the relevant values for the soil strength parameters and material partial factors to use in the analysis.

The soil strength values for this design have been derived as a worst credible estimate of peak strength taking account of the possible long-term reduction in strength, ie a partial factor f_{ms} = 1.45 has been applied to $\tan\phi'_{peak}$ to obtain $\tan\phi'_{des}$. For this reason it seems reasonable to adopt either:

1 a factor of safety against sliding of 1.5 in conjunction with ϕ'_{peak}

2 a lower factor of safety of 1.3 in conjunction with ϕ'_{des}

For a nail length of 7m the results of the slope stability analysis using the above partial factors of safety and soil strengths (both for r_u = 0) are as follows:

1 with ϕ'_{peak} = 40°, minimum factor of safety = 3.0

2 with ϕ'_{des} = 30°, minimum factor of safety = 2.0

Thus there is an adequate factor of safety against slope failure.

A.8 SLS condition

A.8.1 Settlement of structure

Because the structure is formed by excavation into existing ground there will be a net unloading of the ground resulting in heave in front of the wall. However, in the sandy gravel on site it is thought that resulting movements would not be significant.

A.8.2 Deformation of block of reinforced soil

Information on calculating typical movements of soil nailed structures is given in Murray (1993) and is shown in Figure A2 and Table 10 of this report. Vertical or horizontal deformation at the crest of a wall in coarse sand or gravel can be estimated from:

$$\delta_v = \delta_h = H/1000 \qquad (A12)$$

where: H = height of wall

Deformation at the rear of the soil nailed mass is given by:

$$\delta_0 = 4H/10000 \text{ to } 5H/10000 \qquad (A13)$$

For a 7m high wall in gravel, horizontal and vertical deformation at the crest of the wall would be of the order of 7mm, and the deformation at the rear of the soil nailed mass would be some 2.8mm to 3.5mm. The values of δ_v, δ_h and δ_0 are small in terms of the limits allowed for the SLS condition of the wall and are therefore considered acceptable.

A.9 Corrosion protection

No formal aggressivity assessment was made at the site but the sand and gravel material and the low water table (below the base of the wall) would be likely to be non-aggressive or only slightly aggressive. BS8006:1995 gives advice on corrosion aspects of reinforced earth structures but only limited guidance on corrosion protection for soil nails in natural ground. If the natural material met the requirements of Table 4 of BS8006:1995 then stainless steel or galvanized steel meeting the requirements of Section 3.2 of BS8006:1995 would be suitable.

The nails employed on Scheme A were manufactured from 25mm high yield steel. The mean required design rupture strength of a nail is 95kN (Equation A5). The design rupture strength of a nail is 158kN (Equation A7) and thus some sacrificial thickness of the steel is present. The nails were galvanized, although not to the enhanced zinc thickness required by BD70 (DMRB 2.1) and Section 3.2.2.1 of BS8006(1995), which would provide a sacrificial zinc layer to delay corrosion. The most rigorous component of the corrosion resistance was the corrugated plastic sheath into which the nails were grouted. As this process was carried out off site, in factory conditions, good control of

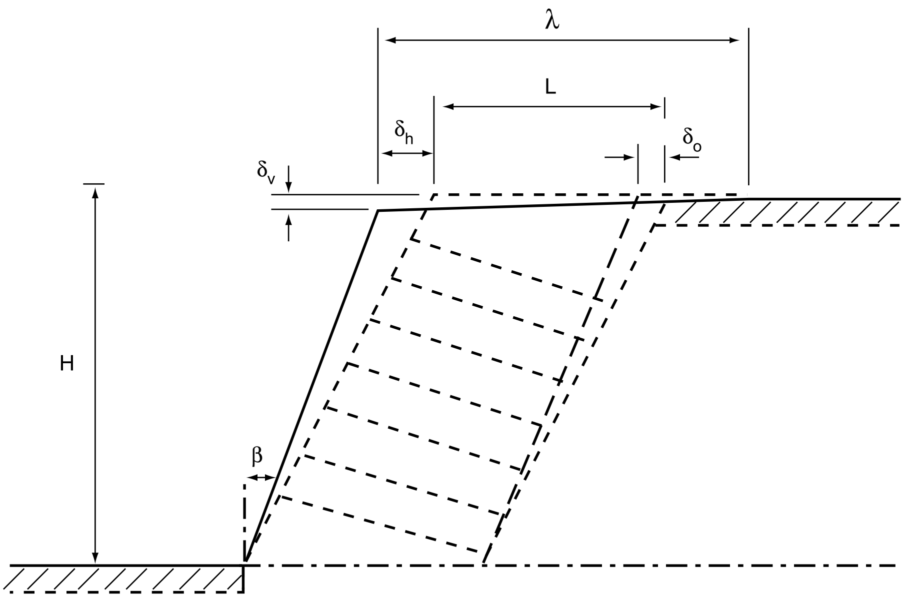

Figure A2 Definitions of variables in soil nailing structure

this operation was possible. The assembly was then grouted into a 140mm borehole as wall construction progressed. Although it is not possible to make a definitive assessment of the adequacy of the corrosion protection on this site, overall the measures taken appear to be reasonable.

Appendix B: Design check of a strengthened retaining wall (Scheme B)

B.1 General design philosophy

The original strengthening design employed a draft version of BS8006 (dated 1991) *Draft code of practice for strengthened/reinforced soils and other fills*. Section 6 of this draft dealt with the general philosophy for soil reinforcement and Section 7 with reinforced walls. (In the final version of BS8006:1995, principles of design are given in Section 5 and wall design is in Section 6.) For soil nails, however, the documents provide limited advice and a number of subjective decisions needed to be made by the designer. For the original design the draft Standard was supplemented by BS8081:1989 *Code of practice for ground anchorages* and HA68 (DMRB 4.1) *Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques*.

This design check can be divided into two parts.

1 Assessment of the current stability using Departmental Advice Notes and Standards for retaining structures. The methodology to be used for assessing retaining structures is set out in BA55 (DMRB 3.4.8) *The assessment of bridge structures and foundations, retaining walls and buried structures*, which summarises the relevant requirements and use of appropriate Departmental Standards and Advice Notes.

2 Design of strengthening work. The required strengthening forces are determined in accordance with BS8002:1994 *Code of practice for earth retaining structures*, together with relevant Departmental Standards and Advice Notes for the design of retaining structures. As discussed in Section B.6.1, some judgement is required in the application of either BS8002:1994 or BS8006:1995 to the design of reinforcement. This depends on the magnitude of required strengthening forces in relation to the available restoring forces. Reference has also been made to BD70 (DMRB2.1) *Use of BS8006:1995*, and TRL Report RR380 *The development of specifications for soil nailing* (Murray 1993).

Details of the wall are given in Figures 4 and 5 of the main report. It is necessary to ensure that the existing wall satisfies the appropriate ULS or SLS conditions. Failure is defined by an inadequate factor of safety for the particular limit state condition. The limit state conditions to be considered are shown in Table B1.

Table B1 Assessment limit states

Limit state	*Internal stability*	*External stability*
Ultimate (ULS)	yes	yes
Serviceability (SLS)	no	yes (visual check on wall deformation)

In this particular example, the degree of bulging of the wall had exceeded the serviceability limit and it was therefore necessary to check the ULS condition for external stability (i.e. sliding, bearing capacity, overturning and slope stability) to determine the most likely mode of failure and thus the requirements for strengthening. In general, however, it would be necessary to check both internal and external stability, particularly where there was no clear visible failure or distress and/or an increase in working loads was planned.

The design of the strengthening works was undertaken in accordance with current design codes for retaining structures. BS8002:1994 was used to calculate the necessary out of balance force to be carried by the soil nails. The resisting force and design of the nails themselves was then undertaken in accordance with Section 6 of BS8006:1995.

B.2 Design soil parameters

Representative soil parameters for the Glacial Till backfill were assumed to be:

$$\phi'_{peak} = 32°,\ c'_{peak} = 0,\ \gamma_{fill} = 18kN/m^3$$

(The value of 32° for ϕ'_{peak} is between the values of $\phi'_{peak} = 35°$ and $\phi'_{des} = 25°$ used in the original analysis. The assumption of $c'_{peak} = 0$ was made in the original design and appears reasonable for the long-term properties of the mixed material.)

The ground conditions below the foundations were not known. However, it was assumed that the retaining wall would be founded on weathered rock. For the purposes of assessment it was assumed that the foundation provided an adequate bearing. With regard to sliding of the wall, the angle of base friction was based on ϕ'_{peak} for the backfill (32°).

The following representative parameters were assumed for the foundation soils:

Weathered rock, surface to 2m depth: $\phi'_{peak} = 32°$, $c'_{peak} = 50\ kN/m^2$, $\gamma_{soil} = 18kN/m^3$

Unweathered rock, below 2m depth: $\phi'_{peak} = 35°$, $c'_{peak} = 50\ kN/m^2$, $\gamma_{soil} = 20kN/m^3$

Groundwater was assumed to be 2m below original ground surface and would not influence the behaviour of the wall: therefore $r_u = 0$ was assumed.

For assessment, the partial material factor (f_{ms}) applied to the soil strength is unity. For the design of the strengthening works a mobilisation factor, M = 1.2, is applied to the peak strength values in accordance with BS8002:1991 Section 3.2.5, such that:

$$\tan \phi'_{des} = (\tan \phi'_{peak})/1.2 \text{ and } c'_{des} = c'_{peak}/1.2$$

B.3 Applied loads

In accordance with Section 4.2 of BD37 (DMRB 1.3), when assessing the stability of existing structures the factors of safety may be relaxed and the assessment performed with nominal loads. The design of the strengthening works was in accordance with BS8002:1991 for which the partial load factors were taken to be unity for the various ULS and SLS conditions because a mobilisation factor, M, was applied to the soil strength resulting in the most severe distribution of horizontal earth pressure on the wall.

In assessment and in accordance with Section 5.8.2.1 of BD37 (DMRB 1.3) the nominal HA loading of 10kN/ m² was applied as the live load surcharge, W_{LIVE}, to the rear of the wall.

In this design check, the nominal HA loading of 10kN/m² was also applied as a live load surcharge. This is because it was found to result in greater force on the wall than the HB loading of 20kN/m² acting beyond a one metre strip adjacent to the wall. The strip represents a footway with a footway loading of 5kN/m².

With regard to the dead weight of the soil, it was assumed that $\gamma_{fill} = 18kN/m^2$ for the Glacial Till backfill.

B.4 Assessment of the existing wall

B.4.1 General

It is necessary to check the stability of the structure at ULS for external stability for the following conditions:

1 sliding
2 bearing capacity
3 overturning
4 overall stability - slip failure through soil mass.

B.4.2 Earth pressure coefficients

From Section 3.2.6 of BS8002:1994 the angle of wall friction and base friction, β is given by:

$$\beta = 2/3\ \phi'_{peak} = 2/3 \times 32° = 21.3°$$

Since the rear face of the wall is vertical and the ground surface behind wall is horizontal the active coefficient of lateral earth pressure, $K_a = 0.26$, from Figure A1 of BS8002:1994.

B.4.3 Applied loadings (Using nominal, unfactored values)

Applied live and dead loads

Figure 4 shows the geometry of the wall.

Soil force from surcharge,

$F_1 = W_{LIVE} K_a H$ where: H = retained height of wall

$F_1 = 10 \times 0.26 \times 2.65 = 6.9kN$ (Lever arm about toe of wall = 2.65/2 = 1.3m)

Soil force from earth fill,

$F_2 = 0.5\ K_a \gamma_{fill} H^2$

$= 0.5 \times 0.26 \times 18 \times 2.65^2 = 16.4kN$ (Lever arm about toe of wall= 2.65/3 = 0.9m)

Total horizontal force, $F = F_1 + F_2 = 23.3kN$

Weight of wall

The unit weight of the masonry wall was assumed to be 24kN/m³. For computation purposes the wall was divided into a rectangular block, W_1, and triangular block, W_2 (Figure 4).

$W_1 = 0.7 \times 2.65 \times 24 = 44.5kN$ (Lever arm about toe = 0.5m)

$W_2 = 0.5 \times (0.85 - 0.7) \times 2.65 \times 24 = 4.8kN$ (Lever arm about toe = 0.1m)

Total weight of wall, $W = W_1 + W_2 = 49.3kN$

B.4.4 Factor of safety against sliding

Resistance of existing wall to sliding, R

$R = W \tan\beta = 49.3 \times \tan 21.3° = 19.2kN$

Factor of safety against sliding is given by:

$R/F = 19.2/23.3 = 0.82$

The factor of safety is less than unity indicating that the wall may fail by sliding at this ULS condition and is therefore not satisfactory.

B.4.5 Overturning

The overturning moment, M_o is due to the horizontal forces generated by the surcharge load and soil mass, given by:

$$M_o = F_1 \times 1.3 + F_2 \times 0.9 = 23.7\text{kNm}$$

The restoring moment, M_r is due to the vertical weight of the wall, given by:

$$M_r = 44.5 \times 0.5 + 4.8 \times 0.1 = 22.7\text{kNm.}$$

The factor of safety against overturning is given by:

$M_r / M_o = 0.95$ This value is less than unity indicating that the wall may overturn at this ULS condition and is therefore inadequate.

B.4.6 Overall stability

The geometry of the slope and ground conditions were not known in detail. A potential circular slip failure could be assumed behind the existing wall. The calculations were performed using Bishop's method of analysis using a commercially available program. Based on the geometry and the values of the soil strength used for assessment, the analysis suggests that overall stability was satisfactory, with factors of safety for potential failure planes generally in the range 1.4 to 2.7. For conventional slopes BS6031:1981 requires that the factor of safety against instability is not less than 1.3 to 1.4 for permanent works: thus the existing wall satisfies the requirements for overall stability.

B.4.7 Conclusion

The above calculations show that the existing retaining wall is in a state of marginal equilibrium with regard to both sliding and overturning. This is reasonably consistent with the evidence that the wall is known to be bulging and therefore strengthening works are required to prevent sliding and overturning.

B.5 Required strengthening of the wall

B.5.1 General

The design of the strengthening works to prevent instability was undertaken in accordance with BS8002:1995 and Departmental Standards and Advice Notes relevant to new structures. In accordance with BS8002:1995, the load factors to be applied for ULS conditions when considering the external stability of the structure are as follows: no load factor (or $\gamma_{fL} = 1.0$) for lateral load from retained soil, and $\gamma_{fL} = 1.5$ in accordance with BD37 (DMRB 1.3) for the lateral load from surcharge loading applied to the retained soil. The wall was analysed in accordance with BS8002:1994 and the nail force to maintain stability was calculated. The vertical component of the soil nail force was assumed to be carried by the foundation and it was therefore necessary to ensure that the foundation bearing capacity was adequate.

The strengthening requirements could be considered to be onerous (ie erring on the high side) since the actual base of the wall (taken to be 0.85m) was used in the calculations. If soil nailing was specified as the preferred technique at an early stage the designer might make the assumption that the whole soil mass (with a base of about 7.85m in this case) acts as the retaining wall (the normal approach to reinforced earth) which would demonstrate a much more stable condition.

B.5.2 Design soil parameters

As described in Section B.4.2 the values of the soil properties were derived from the representative soil parameters in accordance with Section 3.2.6 of BS8002:1994:

$\tan \phi'_{des} =$ representative $\tan\phi'_{peak}/M$, where: $M = 1.2$

ie $\phi'_{des} = \tan^{-1} (\tan 32°/1.2) = 27.5°$

B.5.3 Earth pressure coefficients for the backfill

As set out in Section B.4.2 the wall friction and earth pressures are calculated using ϕ'des = 27.5° The design wall friction, $\beta = 2/3\ \phi'_{des} = 21.3°$

For the rear face of wall which is assumed vertical and a horizontal ground surface,

$K_a = 0.31$ as given in Figure A1 of BS8002:1994.

B.5.4 Applied design loads per metre of wall

As described in Section B.3 the nominal HA loading of 10kN/m^2 was applied as the live load surcharge.

Applied live and dead loads

Soil force from surcharge, $F_1 = \gamma_{fL} W_{LIVE}\ K_a\ H$

$$F_1 = 1.5 \times 10 \times 0.31 \times 2.65 = 12.3\text{kN}$$

Lever arm about toe of wall = 2.65/2 = 1.3m

Soil force from earth fill, $F_2 = 0.5\gamma_{fL} K_a\ \gamma_{fill}\ H^2$

$$F_2 = 0.5 \times 1.0 \times 0.31 \times 18 \times 2.65^2 = 19.6\text{kN}$$

Lever arm about toe = 2.65/3 = 0.9m

Total horizontal force, $F = F_1 + F_2 = 31.9\text{kN}$

Weight of wall

As calculated in Section B.4.3 the total weight of wall is 49.3kN

B.5.5 Force required to resist sliding

As calculated in Section B.2.4 the resistance of existing wall to sliding (R) is

$$R = W \tan\beta = 49.3 \times \tan 21.3° = 19.2\text{kN}$$

Factor of safety against sliding is given by:

$$R/F = 19.2/31.9 = 0.60$$

(This factor of safety has a different value from that calculated in Section B.4.4 because different K_a values are applied and F_1, the soil force from surcharge has a $\gamma_{fL} = 1$ in Section B.4.3 and $\gamma_{fL} = 1.5$ in Section B.5.4.

The horizontal component of the soil nail force, T_h required to maintain stability against sliding is given by:

$$T_h = F - R = 12.7\text{kN}$$

Assume that the force will be supplied by soil nails installed at an inclination δ of 12° from the horizontal and that the resultant soil nail force acts at the one third height of the wall.

Lever arm of horizontal component about toe = 0.9m

Design nail force at δ = 12° is given by:

$$T_{des} = 12.7/\cos 12° = 13.0\text{kN}$$

Vertical component of force, T_v

$$T_v = 13.0 \sin 12° = 2.7\text{kN}$$

Lever arm about toe = 0.85m

Total vertical force, $V = W + T_v = 52.0$kN

Thus nails having a total T_{des} of 13kN per metre run would resist sliding.

B.5.6 Force required to resist overturning

The overturning moment M_o is given by:

$$M_o = F_1 \times 1.3 + F_2 \times 0.9 = 33.6\text{kNm}$$

The restoring moment M_r is given by:

$$M_r = W_1 \times 0.5 + W_2 \times 0.1 + T_h \times 0.9 + T_v \times 0.85 = 36.5\text{kNm}$$

The restoring moment is slightly greater than the overturning moment. The small net moment (36.5 - 33.6 = 2.9kNm) about the toe indicates that the resultant force passes through the base close to the toe. For overturning stability, the resultant must pass through the middle third of the base, ie at least 0.28m from the toe.

Vertical force (from Section B.5.5) = 52kN

Distance of load line from toe = 2.9 / 52 = 0.056m

This is less than the 0.28m required and is unsatisfactory.

A greater soil nail force is therefore necessary to ensure stability, ie to ensure the line of the resultant vertical load falls within the inner third. A number of nail forces are checked and it is found that a nail force of 25kN is satisfactory, see below.

$$M_r = W_1 \times 0.5 + W_2 \times 0.1 + T \times \cos 12° \times 0.9 + T \times \sin 12° \times 0.85$$

$$= 49.1\text{kNm for } T = 25\text{kN}$$

$$V = W + T \times \sin 12° = 54.5\text{kN}$$

Check on eccentricity of vertical load,

$$M_r - M_o = 15.5\text{kNm}$$

Distance of load line from toe = 15.5 / 54.5 = 0.284m

This is within the middle third of the base and is satisfactory. The nail force to resist overturning is greater than that required to resist sliding. Therefore for design, the total soil nail force, T_{max} is 25kN/m run of wall.

B.5.7 Bearing capacity

The maximum bearing pressure, σ_{max} from the wall is given by:

$$\sigma_{max} = V(1+6e/b)/b,$$

where: b is the foundation width = 0.85m

e is the eccentricity from the mid point = 0.14m

Thus σ_{max} = 130kN/m^2. For the purposes of these calculations the foundations are assumed to provide adequate bearing capacity. In a rigorous check the existence of sloping ground in front of the wall should be included.

B.6 Design of soil nails

B.6.1 General philosophy

The overall structure has been designed as an earth retaining structure in accordance with BS8002:1994. The design assumes that the soil nails will resist part of the loading thus improving the factor of safety against overturning and sliding when assessed at the ULS condition. It is intended that the soil nails are designed to BS8006:1995 and it is a matter of engineering judgement as to how the load is shared between the wall and nails. A continuous wall would be capable of redistributing any local out-of-balance forces. However in the case of a natural stone block retaining wall it is considered appropriate to balance local loads with the local resistance of reinforcement. The local stability of each layer was assessed following Section 6.6.4.2.1 of BS8006:1995 but the force to be resisted by each layer was reduced by the proportion of total load carried by the soil nails. The resistance of the soil nails was determined in accordance with Section 6.6.4.2.2 using the tie back wedge method, modified to take account of the variation of radial stress around the soil nail.

In BS8002:1994 a modification factor of 1.2 is to be applied to soil strength, but no load factors are applied to dead or live loads. However in BS8006:1995 a partial factor, f_{ms} = 1.0 is applied to soil strength, f_p = 1.3 applied to pull-out resistance and f_{fs} or f_q = 1.5 applied to soil weight and surcharge. As the majority of the restoring force will be carried by the soil nails, the partial factors given in BS8006:1995 were applied in the design of the nails. If the proportion of load to be carried by the soil nails had been smaller in relation to the retaining wall, it might have been more appropriate to design the soil nails using the partial factors given in BS8002:1994.

BS8006:1995 recommends the use of design strengths based on peak strength parameters for design of walls, therefore:

$$\phi'_{des} = \phi'_{peak} = 32°$$

Furthermore the nails are designed to support a natural stone block wall, therefore the spacing should be such as to effectively distribute the nail load into the wall. Assuming a 45° load spread from the face of the wall, the maximum spacing of nails should be about 1.5m to 2.0m.

The required design total nail force, T is 25kN per metre of wall (see Section B.5.6), with horizontal component 25 x cos12° = 24.5kN. The total horizontal force, F = 31.9kN so that the proportion of the load carried by the soil nails = 24.5/31.9 = 0.76. From BS8006:1995 the inclination of the failure plane to the horizontal can be taken as $(45° + \phi'_{peak}/2)$ = 61° The hole diameter is 68mm and a 25mm diameter stainless steel reinforcement bar is specified as one option with GRP as a second option. The adopted layout of the nails is shown in Figure 5 of the main report.

B.6.2 Corrosion protection

No formal aggressivity assessment was made at the site. One of the acceptable nail materials was stainless steel type 316S33, which is one of the materials listed in Table 6 of BS8006:1995. The calculated rupture strength of the nail in Section B.6.3 is 152 kN and is significantly higher than the approximately 30kN maximum strength required in Table B2. Using these figures the minimum steel nail diameter required is 11mm compared to the 25mm employed in the design calculations. This could be considered as a 7mm sacrificial corrosion allowance on the radius of the nail. If the natural soil is relatively non-aggressive and falls within the limits of Table 4 of BS8006:1995 then a sacrificial corrosion thickness of 0.1mm would be needed (BS8006 Table 7). Thus, although difficult to quantify, it could be argued that the actual 7mm sacrificial thickness would provide protection in significantly more aggressive soils.

Table B2 Calculation of pull-out resistance required

Nail row	*Depth from crest h_j (m)*	*Vertical spacing S_{vj} (m)*	*Required resistance T_{pj} (kN/m)*	*Horizontal spacing S_h (m)*	*Nail force required $0.76T_{pj}S_h$ (kN)*
1	0.50	0.88	6.49	2.0	10.09
2	1.25	0.75	9.56	2.0	14.86
3	2.00	1.03	18.64	2.0	28.96

The alternative nail permitted in the strengthening works, and the one actually employed, was a glass reinforced plastic (GRP) tube with an outside diameter of 22mm and an inside diameter of 12mm. The specification called for a tensile strength of 310kN. Corrosion protection is not discussed in the original design document and it is assumed that the designer either considered GRP to be non-degradable or that the 310kN specified strength was so much greater than the calculated working load of 22.4kN that the residual strength after any degradation would still be satisfactory. As discussed in Section 3.3.3.4, earlier work recommended that the long term working strength of GRP reinforcement should be taken as 10% of its short-term ultimate tensile strength. For the nails employed on Scheme B this would give a working load in the anchor of 31kN and a working bolt head load of 16kN (based on the manufacturer's quoted breaking load of 160kN). This 90% reduction in available strength might be considered overly conservative but it still provides available strengths of a similar value to the calculated requirements in the design.

B.6.3 Local stability of layers

The ultimate tensile force to be resisted by a layer of nails was compared with the local rupture force and pull-out, in accordance with Sections 6.6.4.2.1 and 6.6.4.2.2 of BS8006:1995.

The partial load factors applicable to soil weight and surcharge, f_{fs} and f_q, are taken from Table 17 of BS8006:1995, and each combination A, B and C, should be checked. However in these calculations, combination A has been found to be the most critical condition, so calculations for the other combinations are not shown.

For each combination the same load factor should be used for determination of pull-out resistance as is used for determining the required resistance (f_{fs} in Section 6.6.4.4.2b in BS8006:1995). Thus in combination A the partial factors f_{fs} and f_{fq} = 1.5 is applied to the soil weight, any dead load surcharge and any live load surcharge in calculating the disturbing force. When considering pull-out resistance the same factor f_{fs} = 1.5 is used, but live load surcharges are not included in the calculation (only dead load surcharge (w_s) Section 6.6.4.4.2b).

The maximum force in the direction of the reinforcements inclined at δ, to be resisted by the j^{th} layer of reinforcement is:

$$T_{pj} = K_a (f_{fs}\gamma h_j + f_q w_s) S_{vj} / [\cos\delta \{\{1 - K_a (f_{fs}\gamma h_j + 3f_q w_s)\} (h_j/L)^2 / \{3 (f_{fs}\gamma h_j + f_q w_s)\}\}]$$

where: (From BS8006 Section 6.6.4.2.1)

K_a = 0.26, based on unfactored soil strength (ϕ'_{des} = 32°, Section B.4.2 of this Appendix)

f_{fs} = partial load factor applicable to soil weight = 1.5

f_q = partial load factor applicable to surcharge = 1.5

δ = nail inclination = 12°

γ = unit weight of backfill = 18kN/m^3

h_j = depth of elements below top of structure

S_{vj} = vertical spacing of reinforcement

L = length of reinforced zone, take as nail length.

For the soil nail distribution described above the required force to be resisted by each row of soil nails is given in Table B2. The force required is reduced by the proportion of load carried by the soil nails, ie 0.76.

B.6.3.1 Resistance to rupture

In accordance with Sections 5.3.3.2 and 6.6.4.2.2 of BS8006:1995 the design tensile strength of the soil nail (T_j) is given by:

$$T_j = T_u/(f_m f_n)$$

where: T_u = ultimate tensile strength of reinforcement $= A_s \sigma_t$

A_s = area of reinforcing bar, diameter of bar = 25mm

σ_t = tensile strength of reinforcement = 510N/mm^2, from Table 6 of BS8006:1995 (assuming stainless steel nails as one of the permitted options)

f_m = partial material factor for steel reinforcement = 1.5, from Annex A.2 (BS8006)

f_n = 1.1 for category 3 structure, from Table 3 and Figure 13 (BS8006)

Therefore T_j = 151.7kN, which is greater than the nail force required.

B.6.3.2 Resistance to pull-out

The pull-out resistance (P_j) of a soil nail, based on BS8006:1995 Section 6.6.4.2.2 but modified in accordance with HA68 (DMRB 4.1) to account for radial pressure on an inclined nail, is

$$P_j = \pi\, d\, l_i\, (a\, \sigma_n \tan\phi'_{des}/f_{ms}) / (f_p\, f_n)$$

where: d = diameter of nail = 68mm

l_i = effective length of the i_{th} nail behind failure plane

f_p = partial factor for pull-out resistance of reinforcement = 1.3 (from Table 16 of BS8006: 1995)

f_{ms} = 1.0, partial material factor applied to $\tan\phi'_{des}$ (from Table 16 of BS8006, $\phi'_{des} = 32°$)

a = interaction coefficient relating soil/ reinforcement bond = 1.0 for rough borehole

σ_n = $½ (\sigma_v + \sigma_l)$

σ_v = $f_{fs}\, \gamma\, z_i\, (1 - r_u)$ note that the contribution from the live load surcharge W_{LIVE} is not included in determining the pull-out resistance of the nail, but the partial factor applicable to soil weight is included

σ_l = $\sigma_v K_l$

K_l = $½ (1 + K_a)$ where $K_a = 0.26$

z_i = average depth of embedment over effective length

For the soil nail distribution described above the pull-out resistance of each nail is given in Table B3.

Table B3 Calculation of pull-out resistance available

Nail row	*Effective length l_i (m)*	*Average cover z_i (m)*	*Pull-out resistance of nail P_j (kN)*
1	5.11	1.26	13.20
2	5.49	1.97	22.20
3	5.77	2.67	31.63

The pull-out resistance of each row is greater than the pull-out resistance required as shown in Table B2. Furthermore the total pull-out resistance of 67.03kN is greater than the total resistance required of 25kN/m over a 2m width ie 50kN (Section B.6.1).

B.6.4 Connection with wall

The connection of the nails with the wall is not considered in these calculations. Table 25 of BS8006:1995 indicates that the connection with the wall should be able to resist the maximum tensile load required (as given in the final column of Table B2). The design of the connection is more problematic for reinforced earth where fixings are generally confined to the rear of the facing. The method of construction on this scheme, employing a 130mm spreader plate against the larger, more solid blocks in the wall is judged to provide a robust connection detail. It is probable that the weakest feature of this detail would be the rupture strength of the stainless steel or GRP thread. These strengths are discussed in Section B.6.2: generally they exceed the required loads significantly. For GRP, downrating the strength of the nail body by 90% provides a safe working load of 31kN. A decision has to be made for the long-term strength of the moulded plastic thread and its connection to the GRP body of the nail. The manufacturer claimed an ultimate breaking load of the nail thread of 160kN and a working load of 100kN. This is significantly higher than the maximum nail forces given in both Table B2 and Table B3. A 90% downrating of the measured rupture strength of the thread (Section 3.3.3.4 of the main report) gives a working load of only 13kN. However, because the exact failure mode of the plastic thread is not known, and this large strength reduction of 90% is not normally applied to plastics, it is assumed that the thread detail is reasonable.

Abstract

This report is intended to encourage the use of soil nailing where technical or economic benefits would result. Soil nails may be used for the construction of new walls or the strengthening of existing ones, but the lack of a definitive, published design method may be inhibiting their use. The report describes and discusses two highway schemes where soil nails were used. It also attempts to draw together the relevant parts of various British Standards, DETR (Department of the Environment, Transport and the Regions) documents and other publications. Where clear guidance is not available discussion and advice is provided which should be of value to prospective designers and clients.

Related publications

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

CR274 *A new design procedure for the nailing of slopes: the PROSPER program* by P Delmas, J C Berche, G Cantier and A Abdelhei. 1991 (price £20, code C)

CR239 *Soil nailing - experimental laboratory study of soil-nail interaction* by J Marchal. 1990 (price £20, code B)

CR214 *Examples of the use of nailing for stabilising unstable slopes* by G Cartier. 1990 (price £20, code B)

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

SR583 *Pull-out tests on reinforcements embedded in uniformly graded sand subject to vibration* by R T Murray, D R Carder and J V Krawczyk. 1980. (price £20, code AA)

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