Soil Nailing: Application and Practice – part 1

by D.A. BRUCE[††], BSc, PhD, CEng, MICE, MIWES, MASCE, MHKIE, FGS
and R.A. JEWELL[†], MA, PhD, MICE

Preamble

For well over a decade now, engineers in France, Germany and N. America have been exploiting the special advantages of the technique of soil nailing. This geotechnical engineering process comprises the insitu reinforcement of soils and has a wide range of applications for stabilising excavations and slopes. It has been researched with large budgets since 1975 by collaborations of contractors, universities and government organisations. It has been the subject of International Conferences, Symposia and Seminars since 1979, and has given rise to a rapidly expanding literature of technical papers and articles. There are abundant successful case histories to cite in a wide variety of ground conditions and applications.

And yet, as far as the authors are aware, engineers in Britain seem to have either ignored these developments or have remained unaware of the pedigree the technique has now established over the years. This review, compiled with the co-operation of researchers and practitioners in Europe and the United States, is intended to reveal both the potential benefits which the system can provide, and the means to realise them. It summarises the major features, and historical evolution of soil nailing, it illustrates the popular applications by brief accounts of the more significant projects executed, and it describes the construction methods now being applied.

In a companion Paper*, the results obtained from measurements on full-scale trials and model tests are introduced, and the current understanding of soil nailing behaviour and existing design methods are described.

††Now Technical Director/Director Special Projects, Nicon Corp., Bridgeville, PA, U.S.A.
†Department of Engineering Science, University of Oxford

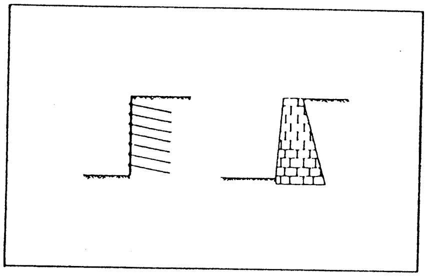


Fig. 1. The analogy between a gravity wall and nailed soil structure (Stocker et al *1979)*

1. General introduction

Soil nailing is a practical and proven technique used in constructing excavations and stabilising slopes (Photographs 1 and 2) by reinforcing the ground insitu with relatively small, fully bonded inclusions, usually steel bars. These are introduced into the soil mass, the face of which has been locally stabilised by sprayed concrete, and act to produce a zone of reinforced ground. This zone then performs as a homogeneous and resistant unit to support the unreinforced ground behind, in a manner similar to a conventional gravity retaining wall (Fig. 1).

1.1 Piling and insitu reinforcement

There is a wide and growing use of metal inclusions installed in soil, and it is important to distinguish between them. The two basic groupings are *piling* and *insitu reinforcement*, and the main distinction is as follows:

Piling refers to inclusions placed in the soil to support external loads applied directly to them.

Insitu reinforcement refers to inclusions placed in the soil to maintain equilibrium

*"Soil nailing: performance and design" by Drs. Jewell and Bruce will follow this Paper in later editions of *Ground Engineering*. Each Paper will be published in two parts; the final instalment will be followed by the references for both Papers.

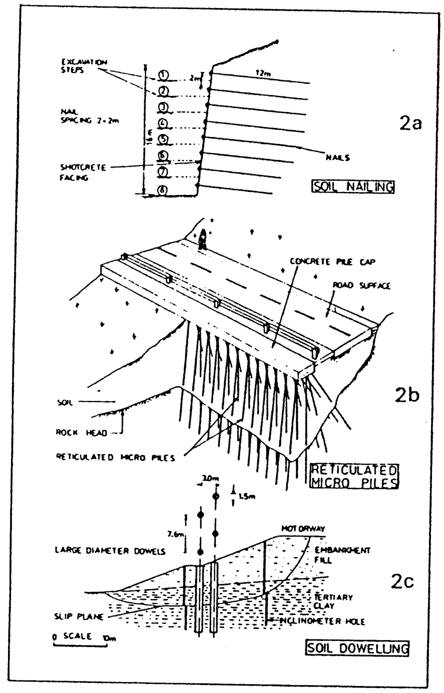


Fig. 2. The family of insitu soil reinforcement techniques (a) soil nailing (after Schlosser, 1982) (b) reticulated micro piling (after Boley and Crayne, 1985) and (c) soil dowelling (after Gudehus and Schwarz, 1984)

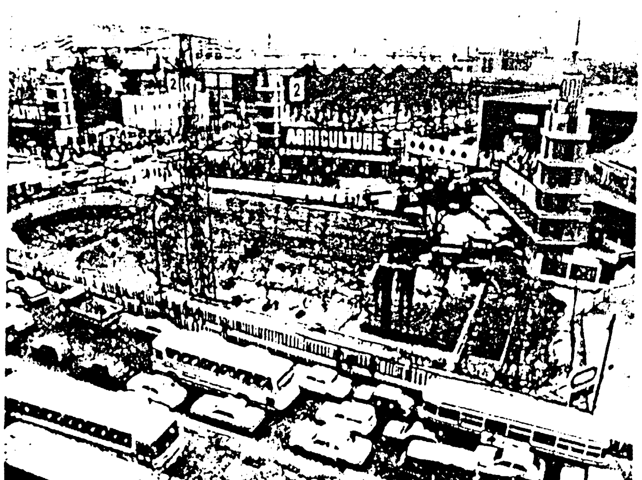


Photo. 1. Excavation for the underground car park, Boulevard Victor, Paris, 1978. (Medio et al, *1983)*

Photo. 2. Detail of the soil nailing and rig at Boulevard Victor, executed by the Hurpinoise system. (Medio et al, *1983)*

under the soil self-weight loading, and surcharge loading on the soil.

The latter grouping includes the subject of this Paper.

1.2 Insitu reinforcement techniques

There are three main categories of insitu reinforcement techniques used to stabilise soil slopes and excavations. These are *nailing, reticulated micro-piling,* and *dowelling.*

In *soil nailing,* the reinforcement is installed horizontally or subhorizontally so that it improves the shearing resistance of the soil by acting in tension, (Fig. 2a).

Reticulated micro-piles are steeply inclined in the soil at various angles both perpendicular and parallel to the face, (Fig. 2b). The overall aim is similar to soil nailing, namely to provide a stable block of reinforced soil which supports the unreinforced soil by acting like a gravity retaining structure. In this technique the soil is held together by the multiplicity of reinforcement members acting to resist bending and shearing forces. Fondedile's *Pali Radice* system is the best known form of this construction (Lizzi, 1982) whilst more recently Nicholson Construction has applied the technique in the USA under the name *Insert wall* (Nicholson & Boley 1985).

Soil dowelling is applied to reduce or halt downslope movements on well defined shear surfaces, (Fig. 2c). The slopes treated by dowelling are typically much flatter than those in soil nailing or reticulated micro-pile applications. Gudehus has (1983) shown that the most efficient way to improve mechanically the shearing resistance on a weakened shear surface through the soil is to use relatively large diameter piles which combine a large surface area with high bending stiffness. Thus the diameter of a soil dowel is generally far greater than that of a soil nail or micro-pile.

1.3 Selecting insitu reinforcement

Although there are fundamental differences in the mechanical action of these three insitu reinforcement techniques, there are circumstances where more than one may be applied to slope stabilisation as illustrated in Figure 3. The following points merit consideration when choosing the appropriate insitu reinforcement technique.

Laboratory experiments have shown the influence of the inclination and properties of reinforcing members on the shearing resistance of reinforced soil, for example Jewell (1980). These indicate that the reinforcement gives the best increase in strength when it is angled across the potential rupture surface in soil so that the reinforcement is loaded in tension. At other orientations in the soil the reinforcement provides less benefit, and can even reduce the shearing resistance of the soil mass if it acts in compression.

The conclusion, therefore, is that in applications *where a steep slope is to be excavated* in a homogeneous granular soil, it is most efficient to place the reinforcement through the face in a direction close to the horizontal, as in Figure 3b. To stabilise the soil with reinforcement placed in substantially vertical directions (Fig. 3c) will require a much higher density of reinforcement. For this type of application soil nailing is likely to be more cost-effective than reticulated micro piling.

In marginally stable granular or scree slopes when stability must be improved, but where *excavation is not foreseen,* then either soil nailing or reticulated micro-piling would

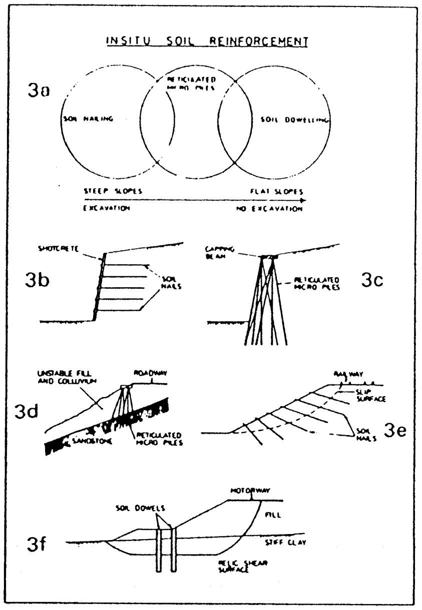


Fig. 3. Overlap of insitu soil reinforcement applications (b) and (c) in excavations, (d) and (e) for general slope stabilisation and (f) to stabilise residual slips in clay

be appropriate. Where drilling equipment cannot be placed on the slope, reticulated micro-piling would be best (Fig. 3d). Where access is not problematical either technique could be applied (Figs. 3d and 3e), with economic considerations being decisive.

In flatter clay slopes where stability is governed by a well defined shear zone, larger diameter soil dowels would be most appropriate (Fig. 3f).

Reticulated micro-piling and *soil dowelling* are not described further in this Paper. The former are described in publications by Lizzi (1970, 1982), Dash and Jovino (1980), Berardi and La Magna (1984), and Boley and Crayne (1985). Soil dowelling is described by Baker and Yoder (1958), Verrier and Merlette (1981), Winter *et al* (1983), and Gudehus (1983).

1.4 Fundamental design considerations

Just as in the design of a gravity retaining wall, the stability of a nailed structure must be checked against both external and internal forces. Regarding *external* forces:

- the reinforced zone must be able to resist the outward thrust from the unreinforced interior, without sliding,
- the combined loading from the reinforced zone self weight and the lateral soil thrust it is resisting must not cause a foundation bearing failure, and
- the stability of the retaining structure must be checked against the deeper seated overall failure mechanisms.

With respect to *internal* stability, the reinforcing elements must be installed in a pattern dense enough to ensure an effective interaction with the soil in the reinforced zone. The reinforcement elements must also have sufficient length and capacity to ensure a stable reinforced zone. In particular:

- each individual reinforcement should be capable of holding the soil immediately surrounding it in equilibrium. This local stability aspect dictates the spacing of the reinforcement, and
- overall slip failure in the reinforced zone must also be considered to ensure against failure by insufficient bond, or breaking of the reinforcement. These criteria govern the required length of the reinforcement.

Each of these aspects of design is detailed and illustrated in the companion Paper*.

1.5 Comparison with prestressed ground anchorages

Superficially, there would appear to be a number of similarities between nails and prestressed ground anchorages when used for slope or excavation stability. Indeed it is tempting to regard nails merely as "passive" small scale anchorages. However, there are major functional distinctions to be made, which will favour the choice of the one over the other. The following comparisons and contrasts may be drawn:

- Ground anchorages are stressed after installation so that in service they ideally prevent any structural movement occurring. In contrast, soil nails are not

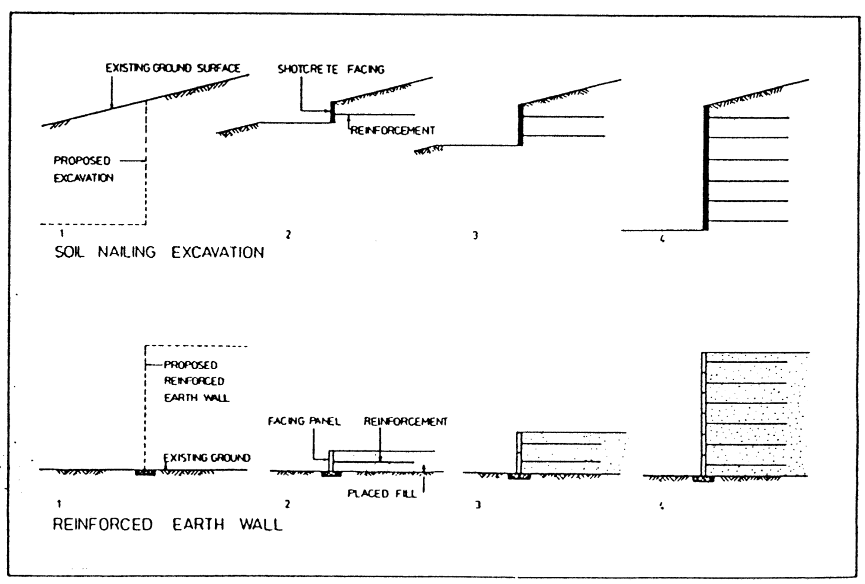


Fig. 4. Contrast of the construction sequence (a) "top down" in soil nailing and (b) "bottom up" for reinforced soil

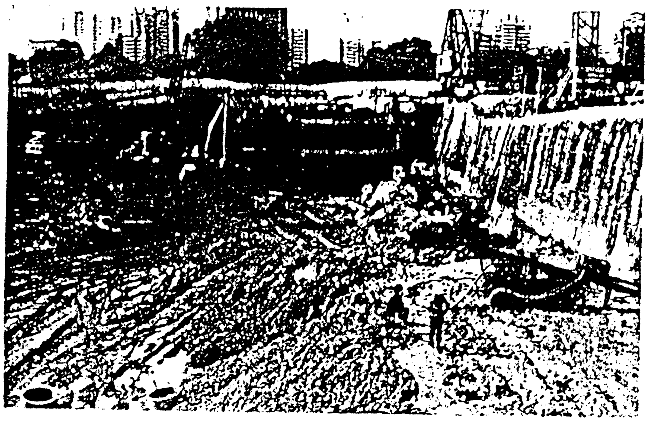


Photo. 3. General view of the equipment and the construction sequence in a soil nailing excavation

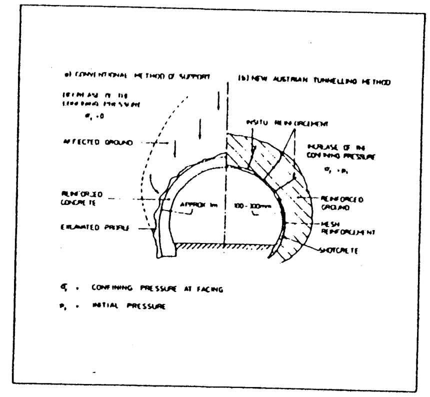


Fig. 6. Schematic comparison of the New Austrian Tunnelling Method and a conventional method of support

prestressed* and require a finite (albeit very small) soil deformation to cause them to work.

- Nails are in contact with the ground over most of their length (typically 3-10m), whereas ground anchorages transfer load only along the distal, fixed anchorage length. A direct consequence of this is that the distribution of stresses in the retained mass is different for each type.
- Since nails are installed at a far higher density (typically 1 per 0.5 to 5m^2) the consequences of a one unit failure are not necessarily so severe. In addition, the constructional tolerances of installation need not be so high, given their overall, interactive mode of operation.
- As high loads have to be applied to anchorages, appropriate bearing facilities must be provided at the head to eliminate the possibility of "punching" through the facing of the retained structure. Substantial bearing arrangements are not necessary with nails whose low individual head loadings are easily accommodated on small steel bearing plates placed on the shotcreted surface.
- Individual anchorages tend to be longer (say 15-45m) and so may necessitate larger scale installation equipment. Also an anchorage system is often provided to stabilise a substantial retaining structure, such as a diaphragm wall or bored pile wall, which will itself necessitate large scale construction equipment.

In general, if the overall stability calculations show the problem to be deep seated, then ground anchorages will most probably be required. Conversely, for vertical excavations, soil nailing has frequently proved preferable to other methods of lateral support incorporating prestressed ground anchorages (such as Berlin or diaphragm walls).

1.6 Comparison with reinforced earth walls

Although soil nailing shares certain features with the older and more widely known technique of reinforced earth for retaining wall construction (Vidal 1966) there are also some fundamental differences which are important to note (Schlosser, 1982).

The main similarities are:

- The reinforcement is placed in the soil unstressed; the reinforcement forces are mobilised by subsequent deformation of the soil.
- The reinforcement forces are sustained by frictional bond between the soil and the reinforcing element. The reinforced zone is stable and resists the thrust from the unreinforced soil it supports, like a gravity retaining structure.
- The facing of the retained structure is thin – prefabricated elements in the case of reinforced earth, and, usually, shotcrete in soil nailing – and does not play a major role in the overall structural stability.

The main dissimilarities are:

- Although at the end of construction the two structures may look similar, the construction sequence is radically different. Soil nailing is constructed by staged excavations from "top down" while reinforced earth is constructed "bottom up", (Fig. 4). This has an important influence on the distribution of the forces which develop in the reinforcement, particularly during the construction period:
- Soil nailing is an insitu reinforcement technique exploiting natural ground, the properties of which cannot be preselected and controlled as they are for reinforced earth fills.
- Grouting techniques are usually employed to bond the reinforcement to the surrounding ground: load is transferred along the grout to soil interface. In reinforced earth, friction is generated directly along the strip to soil interface.

1.7 Benefits and limitations of soil nailing

Several factors have contributed to the growing popularity of soil nailing as a construction technique, and these include:

- *Economic advantage* – based on discussions with specialists in Europe, it would seem that the cost saving for excavations of the order of 10m deep is 10% to 30% relative to an anchored diaphragm or Berlin wall alternative. This is supported by a claimed saving of 30% on a soil-nailed excavation in Portland, Oregon (ENR, 1976).
- *Construction equipment* – drilling rigs for reinforcement installation and guns for shotcrete application are relatively small scale, mobile and quiet (Photograph 3). This is highly advantageous in urban environments where noise, vibration or access may pose problems. Equally in remote rural areas it may prove impossible to deploy large scale equipment for piling or diaphragm walling.
- *Construction flexibility* – soil nailing can proceed rapidly and the excavation can be shaped easily. It is a flexible technique, readily accommodating variations in soil conditions and work programmes as excavation progresses.
- *Performance* – field measurements indicate that the overall movements required to mobilise the reinforcement forces are surprisingly small. These generally correspond to the movements to be expected for well braced systems (Category I) in Peck's (1969) classification (Fig. 5). Furthermore, nailing is applied at the earliest possible time after excavation, and in intimate contact with the cut soil surface. This minimises the disturbance to the ground and so the possibility of damage being caused to adjacent structures.

Naturally, the technique has certain

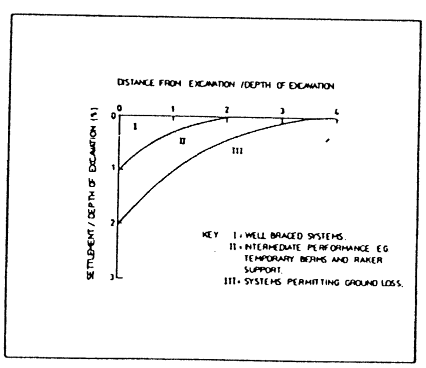


Fig. 5. Field performance of open excavation systems (Peck, 1969)

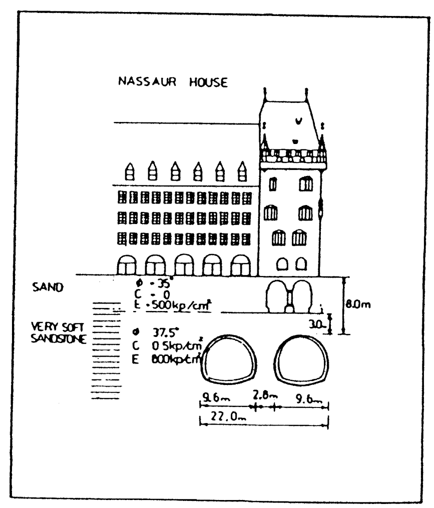


Fig. 7. Subway underground station formed using the NATM, Nurnberg, W. Germany (Bauernfeind et al, 1977)

As is noted later, a small prestress is commonly applied to "drilled and grouted" nails. However, this is done to provide proper seating of the nail/shotcrete/soil structural interface.

practical limitations to its application. These are principally:

- Soil nail construction requires the formation of cuts generally 1-2m high in the soil. These must then stand up unsupported for at least a few hours, prior to shotcreting and nailing. The soil must therefore have some natural degree of "cohesion" or cementing. Otherwise a pretreatment such as grouting may be necessary to stabilise the face, but this will add both complication and cost.
- A dewatered face in the excavation is desirable for soil nailing. If the groundwater percolates through the face the unreinforced soil will slump locally on initial excavation, making it impossible to establish a satisfactory shotcrete skin.
- Excavations in soft clay are also unsuited to stabilisation by soil nailing. The low frictional resistance of soft clay would require a very high density of insitu reinforcement of considerable length to ensure adequate levels of stability. Bored pile or diaphragm walls with anchorages are more suited to these conditions.

2. History and evolution

The principles and techniques of stabilising excavations in rock by in-situ reinforcement have long been applied by mining engineers. Beveredge (1973) noted that the use of mechanical rock bolts grew immediately after World War II, whilst by 1959 the first fully bonded reinforcements (by resin) were being installed in Germany. The New Austrian Tunnelling method, (Fig. 6) evolved in the early 1960's primarily as a hard rock tunnelling system using a combination of shotcrete and fully bonded steel inclusions to provide early, efficient

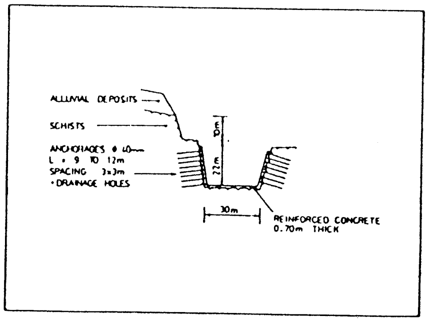


Fig. 8. Stabilisation of rock slopes at Notre-Dame de Commier Dam, France (Bonazzi and Colombet, 1984)

excavation stability.

Following observations and experiences with the system in many hard rock applications, it was adapted successfully to less competent formations comprising graphitic shales as in the Massenberg Tunnel (Rabcewicz 1964/5) and Keuper Marl as in the Schwaikhem Tunnel (Sattler, 1965).

This latter project confirmed the viability of the technique in less competent materials, and soon trials were conducted in soils such as silts, gravels and sands. The earliest applications were in small cross section metro tunnels in Frankfurt in 1970. Soon after, the technique again proved successful in the construction of a double tube of a subway station with cross passages adjacent to delicate and historic buildings in Nuremberg (Fig. 7).

By this time, the use of dowels and bolts to stabilise rock slopes was also well established. For example Bonazzi and Colombet (1984) have described the

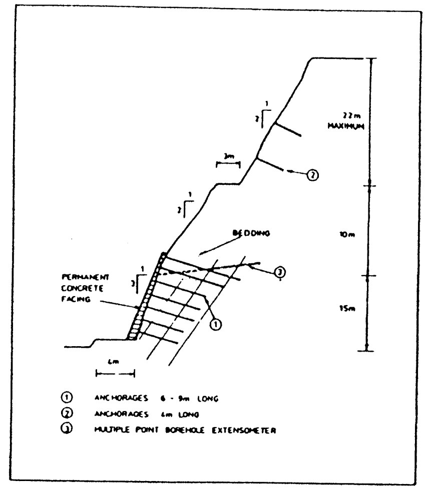


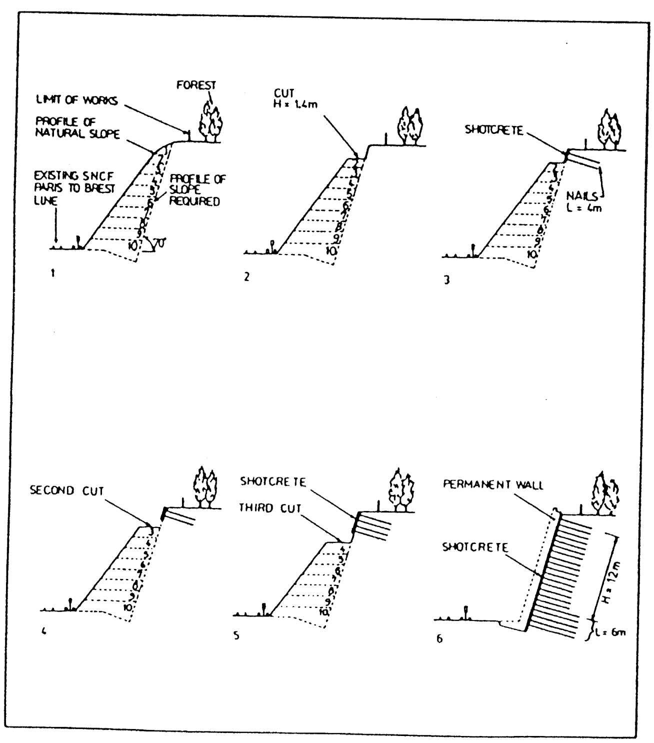
Fig. 9. Retaining wall construction, A9 Autoroute, France (Bonazzi and Colombet, 1984)

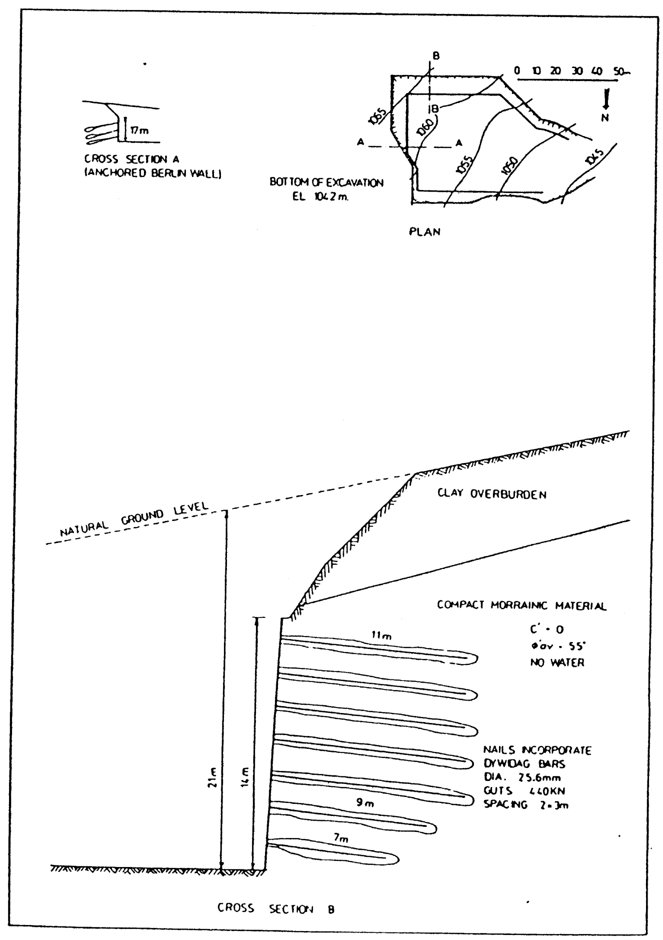
stabilisation of a rock slope in schists (Fig. 8) at the Notre Dame de Commier Dam, France in 1961 by "ancrages passifs" as being one of the first major rock slopes stabilised in that way. They also reviewed applications in other civil engineering projects such as the 45m slope on the A9 autoroute (Fig. 9).

The French contractor Bouygues gained experience in France with the New Austrian Tunnelling Method. They saw that similar techniques could be applied for the temporary support of soft rock and soil slopes, and in 1972, in Joint Venture with the specialist contractor Soletanche, started

Fig. 10 (Below). Cut slope stabilisation construction sequence, Versailles, France (Hovart and Rami, 1975)

Fig. 11 (Right). Retaining wall for an underground car park at La Clusaz, France (After Guilloux et al, 1983)





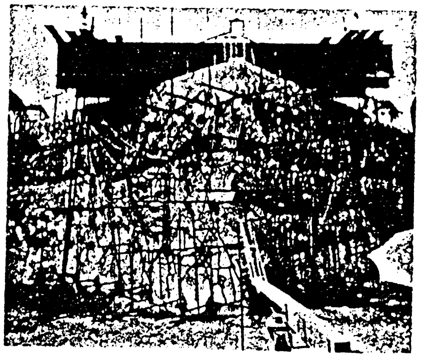


Photo. 4. Large scale field test in Germany of soil nailing on a protruding edge in an excavation, (Stocker et al, *1979)*

work on a 70° cut slope in heavily cemented Fountainbleu Sand for a railway widening scheme near Versailles (Fig. 10). A total of 12 000m² of face was stabilised by over 25 000 steel bars grouted into predrilled holes up to 6m long. This is the first recorded application of soil nailing and the case history is described in detail later in the Paper.

Thereafter, soil nailing as a technique to stabilise slopes and deep excavations grew in popularity in France. Work in West Germany and North America had also started by the mid 1970's. However, engineers in these three areas appear to have proceeded independently until the Paris conference on soil reinforcement in 1979 provided an international forum for the exchange of information. From that time the number of contracts executed with soil nailing has increased rapidly, particularly in France and W. Germany. Based on our interpretation of published Papers and discussions with specialists in Europe and the USA the development in each of the three main areas appears to have occurred as follows:

— France

Following the outstanding and well published success of the Versailles contract, Bouygues executed several other projects, both permanent and temporary, in rural areas for slope stability, and in urban areas for foundation excavations (Medio *et al* (1983)). For the excavation at Les Invalides Metro Station in 1974, Bouygues chose to drive the reinforcement directly into the soil, rather than grout it into predrilled holes. This feature of construction has become central to the Bouygues system of soil nailing called "Hurpinoise" after M. Hurpin, a prime figure in its development.

At the same time, other French specialists including Bachy, Intrafor Cofor, SEFI and Soletanche began to execute projects and to develop proprietary variations. These companies use conventional drilling and grouting techniques to install and bond the reinforcement into the soil. Most recently, a new installation method using simultaneous driving and high pressure grouting has been developed by Dr. Louis in conjunction with the company Solrenfor (Louis (1984)).

The application of nailing in remedial works, such as the repair of failed slopes, reinforced earth walls, or anchored structures, has also grown in France.

These applications of soil nailing have led to fundamental research programmes being carried out in France.

- CERMES the research group at the Ecole National des Ponts et Chaussées under the direction of Prof. Schlosser has carried out model tests and finite element analysis of soil nailing, supervised by Dr. Juran.
- The Centre d'Études et de Recherches du Bâtiment et des Travaux Public (CEBTP) built and tested a 7m high experimental nailed wall during 1984-85, under the direction of M. Plumelle. The work was 75% financed by the Fédération National des Travaux Publics (FNTP) and 25% by the Direction des Affaires Economiques et International (DAEI), a section of the French Ministry of Transport, (Plumelle (1986)).
- Research in France on soil nailing is being continued under a nationally organised project "Programme Clouterre".

Independent assessments of the current level of activity in France suggest a total of around 50 soil nailing projects per year, about 10% of which are in permanent applications. Contracts of up to 24 000m² have been executed, although the average contract size is considerably lower — 1 000 to 2 000m² — with a typical client cost of £50 to £100 per m² (prices at June, 1986).

— West Germany

Developments in W. Germany have been led by the specialist contractor Karl Bauer AG in association with the Institut fur Bodenmechanik und Felsmechanik (IBF), of the University of Karlsruhe, with financial support by the Federal Ministry of Research and Technology, Stocker (1976). In a four year programme commencing in 1975, eight carefully instrumented large scale field trials (Photograph 4) were conducted and analysed, Stocker *et al* (1979).

Model testing and theoretical research into soil nailing has also been conducted in parallel at IBF, University of Karlsruhe, under the direction of Prof. Gudehus.

There is a relative shortage of published information on soil nailing projects in West Germany, but according to Gassler and Gudehus (1981) over 20 projects had been successfully completed by 1981, confirming that "the technique has again and again been found to be safe and economic". The current estimated level of activity is about 25% that of France.

— North America

The excavation for the foundation of the extension to the Good Samaritan Hospital in Portland Oregon (Photograph 5) executed in 1976 by a consortium including Kulchin was the first published application of soil nailing in the United States (ENR, 1976). However, Shen *et al* (1981) refer to the execution of "several hundred thousand square feet of excavation to depths of up to 60ft", by soil

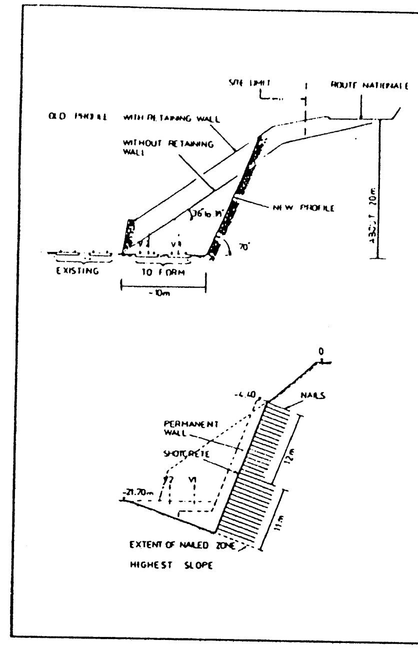


Fig. 12. Cut slope stabilisation at Versailles, France. (After Hovart and Rami, 1975)

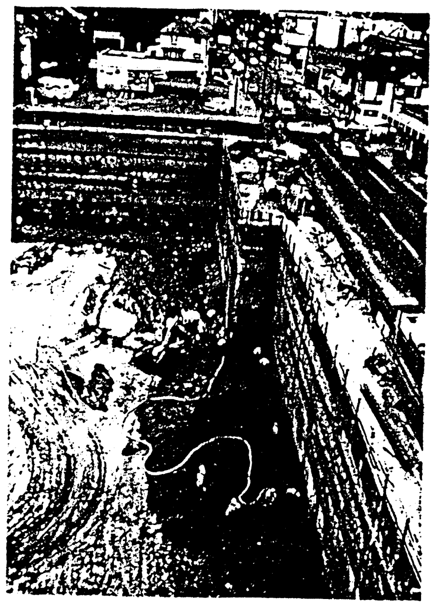


Photo. 5. General view of the excavation at the Good Samaritan Hospital, Portland, Oregon. (Shen et al, *1981)*

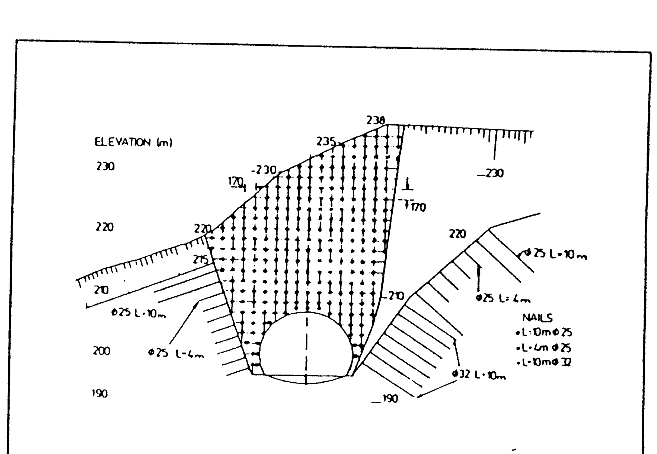


Fig. 13. Stabilisation of tunnel portals and adjacent cut slopes (Louis 1981)

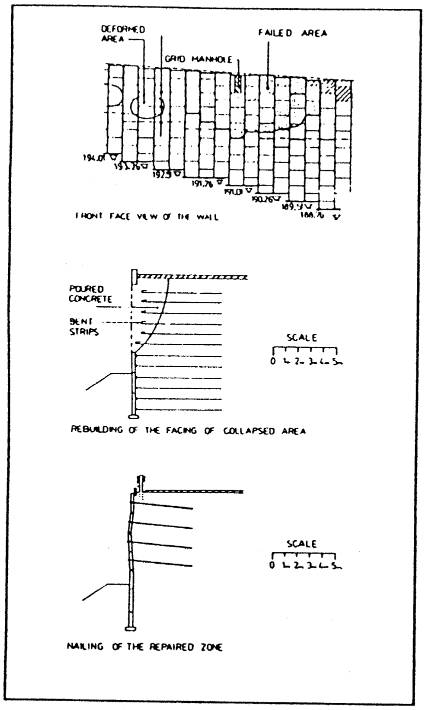


Fig. 14. Repair of reinforced earth wall, Fréjus, France (After Long et al *1984)*

nailing, prior to 1976, in a variety of ground conditions in Western Canada. No other published reference to these projects has been found by the current authors.

Prof. Shen's team of researchers at the University of California at Davis monitored the Portland contract but remained concerned about design methods and the shortage of information on performance and design. They therefore carried out a programme of research including centrifuge testing, an instrumented full scale trial and finite element analyses. The research was funded principally by the National Science Foundation, and the final report published in 1981 (Shen *et al*).

The next significant published development was the excavation for the foundations of the PPG Industries headquarters in Pittsburgh, executed by Nicholson Construction, Nicholson (1986). This was completed in 1982 and is remarkable for the combination of soil nailing with both pre-treatment by grouting of the soil near the face, and underpinning by micropiles of certain critical foundations through the nailed zone.

Although the volume of work currently being conducted in the USA is estimated to be similar to that in West Germany, the potential is far greater, and the level of activity may be expected to increase substantially in the next few years.

Elsewhere in the world, development has been much slower, for reasons which range from lack of applications, or unsuitable soils, to lack of knowledge, or even protectionism of alternative techniques. One may compare experience in Hungary where several contracts are reported by Banyai (1984) with that in Britain where the authors are aware of only three small soil nailing contracts to date.

3. Applications

Soil nailing has been used successfully in temporary and permanent applications, in new and remedial construction, and in rural and urban settings. The following categories of applications can be identified, and selected references are given for each.

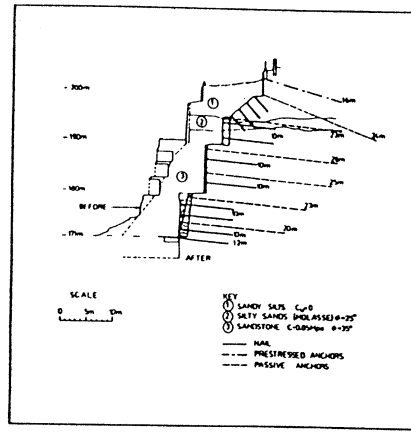


Fig. 16. Repair of a failed soil slope at Herbouville, Lyons, France (After Gauset, 1985)

A. New construction

1. Retaining walls: for excavations associated with foundations of buildings, underground car parks and cut and cover constructions for transportation systems (Fig. 11) (Goulesco & Medio (1981), Stocker *et al*, (1979) and Shen *et al*, (1981).

2. Slope stabilisation: for cuts required for new or widened railway lines or roads (Fig. 12) (Hovart and Rami (1975), Gassler and Gudehus (1981), and Nicholson (1986)).

3. Stabilising tunnel portals: to provide excavation stability to tunnel portals and adjacent slopes (Fig. 13). (Louis (1981)).

B. Remedial works

1. Repair of reinforced earth walls: to replace the effect of the reinforcing strips or fasteners damaged by overloading or corrosion (Fig. 14). (Goytia and Guitton (1979), Long *et al* (1984).

2. Repair of masonry gravity retaining walls: after or just before failure caused by long term decay of wall, or movements behind. (Fig. 15) (See "Case histories" section).

3. Stabilisation of failed soil slopes: after collapse of slope due to failure or inadequacy of preexisting support methods, or catastrophic movements due to hydrogeological reasons (Fig. 16). (Gausset, 1985).

4. Repair of anchored walls: after failure of the prestressed rock anchorages by structural overloading or by corrosion of tendon (Fig. 17). (Corte and Garnier, 1984).

(To be continued)

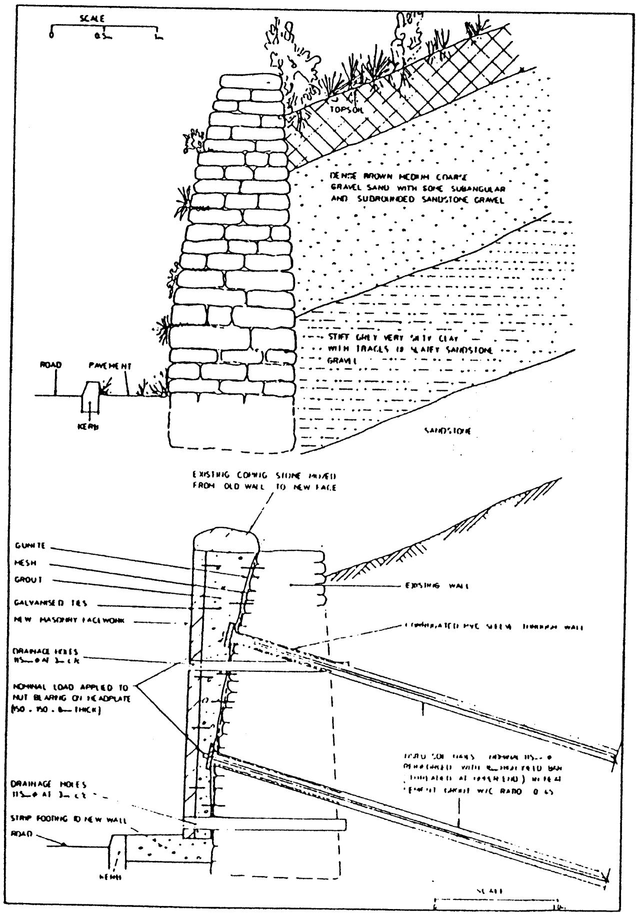


Fig. 15 (Left). Repair of a masonry gravity retaining wall at Bradford, UK

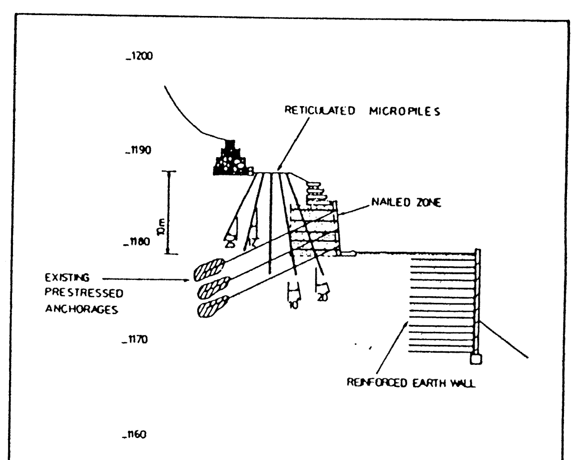


Fig. 17 (Right). Repair of an anchored wall at Fréjus, France (After Corte and Garnier, 1984)

Soil Nailing: Application and Practice – part 2

by D.A. BRUCE[††], BSc, PhD, CEng, MICE, MIWES, MASCE, MHKIE, FGS
and R.A. JEWELL[†], MA, PhD, CEng, MICE

THE INSITU GROUND REINFORCEMENT TECHNIQUE of soil nailing was introduced in Part 1 of the Paper and contrasted with other methods of ground reinforcement and stabilisation. The history and evolution of the technique were reviewed and the main applications described.

The aim of the second half of the Paper is to describe current good practice for soil nailing. Firstly, seven case histories are reviewed illustrating the range of applications for soil nailing and giving construction details. Tables summarising details from published case histories worldwide are presented. Then, on the basis of current experience, each aspect of soil nailing construction is examined to establish guidelines for good practice.

††Now Technical Director/Director Special Projects, Nicon Corp., Bridgeville, PA, U.S.A.

†Department of Engineering Science, University of Oxford.

4. Review of illustrative case histories

The seven case histories have been selected for review on the basis of scale and historical significance. Of the five summaries of nail installations in new construction, three are from France, with two from the United States. This emphasises both the pioneering role of engineers in these two countries, and, regrettably, the shortage of published data on case histories from West Germany. The important work in West Germany on field trials is described in the companion Paper.* One of the two case histories of applications in remedial work is from France, and the British example has importance given the domestic trends for structural repair and maintenance engineering as opposed to new construction.

Summary data for all the following examples are included in Tables 1 to 4.

NEW CONSTRUCTION

4.1 Retaining wall for the French railways (SNCF) at Versailles-Chantiers, France (1972)

(Rabejac and Toudic 1974, Toudic 1975, Hovart and Rami 1975, Medio *et al* 1983.)

In order to improve the railway services to the Versailles-Chantiers Station, the SNCF decided to construct two new lines parallel to the existing Paris – Brest Line, at the west side of the station. For a distance of 965m, cuts had to be created into the existing slope,

*"Soil Nailing: Performance and Design" by Drs. Jewell and Bruce will follow this Paper in later editions of *Ground Engineering*. It will also be published in two parts, with the appropriate references at the end of the second part.

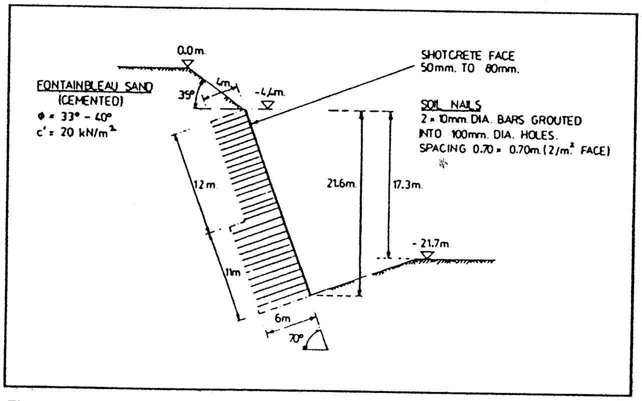


Fig. 18. Cross-section of the highest cut at the Versailles-Chantier excavation (After Toudic, 1975)

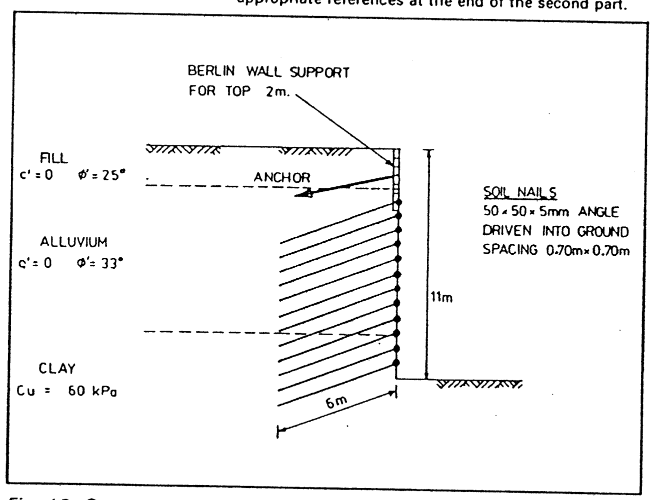


Fig. 19. Cross-section of the Boulevard Victor, Paris excavation (After Medio et al, 1983)

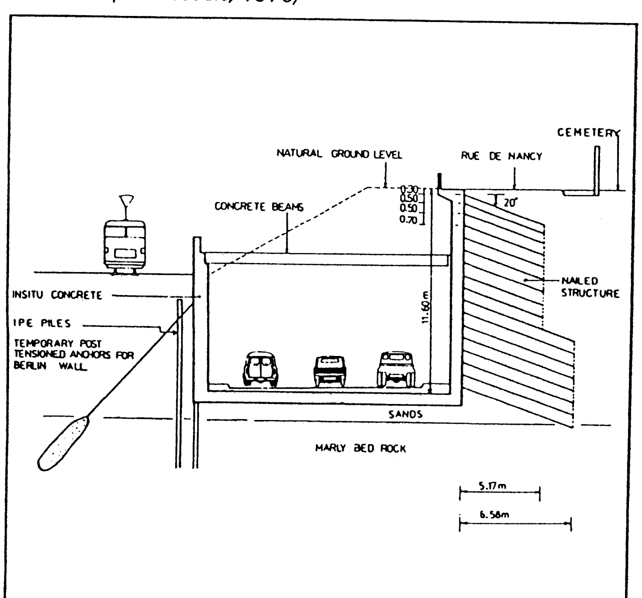


Fig. 20. Cross-section of the Nogent-sur-Marne A86 excavation (After Goulesco and Medio, 1981)

Photo. 6. View showing the nailed excavation and the reinforced concrete wall for permanent support at Versailles-Chantier (Medio et al, *1983)*

TABLE 3: Soil nailing failure case histories; comparison of original and repair designs

PROJECT NAME Description and Scale / DRILLED AND GROUTED	DATE	MAIN REFERENCES	GROUND CONDITIONS Cohesion/Friction/-Unit Weight (kN/m2/-/kN/m3)	SLOPE ANGLE degrees	HEIGHT (m)	NAIL LENGTH (m)	NAIL Diameter (mm)	NAIL HOLE Diameter (mm)	NAIL SPACING Horizontal/-Vertical/Area (m/m/1 per m2)	NAIL ANGLE (-Horiz) degs
FAILURE										
A41 LES EPARRIS, France 70 degree clay cutting 4.2m high in ground sloping at 15 degrees	1981	Schlosser, 1982 Gigan, 1986	Plastic Clay (18 > PI > 15) 0 28 20	70	4.2	4.5	28.0 (eq.)	100 (approx)	3.00 1.40 4.20	20
REPAIR										
Repaired to a 60 degree slope 5.2m high	1982	Schlosser, 1986 Gigan, 1986	0 28 20	60	5.2	10.0	26.0	105	2.00 1.73 3.46	30
FAILURE										
PARIS GARE DU NORD, France Excavation of 10m overall in Marls	1979	Gigan, 1986	Fill overlying 0 30 * Heterogeneous Marls 50 20 *	75 overall	10.0 overall	6.5	32.0	100 (est.)	2.50 1.60 4.00	20 overall
REPAIR										
Repaired as a vertical wall 8.5m high	1979	Gigan, 1986	Assumed strength 0 25 *	90	8.5	10.0	32.0	100	1.50 1.25 1.88	15

length construction was executed using diaphragm walls in the highly variable sediments. However in one particular zone the proximity of a live railway line to the west and existing structures (including a cemetery) on the east (Fig. 20) demanded the use of alternative methods to form the cut. Here the contractor, a consortium of Bouygues and Chantiers Modernes, proposed the use of an anchored Berlin wall construction for 1 900m² (near the railway and where the cemetery prevented nailing being executed), but nailing for the 900m² balance where it was possible to drive the reinforcing elements.

Although not particularly remarkable for its scale, this contract was important because of the test walls and monitoring which were carried out to convince and reassure the Engineer that soil nailing was a sound technique. The testing was carried out before, during and after the construction of the main wall.

The soil was mainly sandy alluvial deposits, ($\phi' = 33°$, $c' = 10kN/m^2$, $\gamma = 20kN/m^3$) but with frequent pockets or lenses of clay, silt and fine sand. In general the nailed wall was constructed in ground regarded as more difficult than that encountered by the Berlin Wall.

A test wall was built 200m north of the main wall in predominantly compact sand and gravels. The wall was 5.6m high and 27.5m long, reinforced with 50 x 50 x 5mm steel angles, 5.5m long driven directly into the soil at an inclination 20° down and at 0.7m centres. Two vertical inclinometers were installed, and eight hollow tube reinforcements (49mm in diameter) were instrumented with strain gauges before being installed in the wall. Tests were conducted to monitor structural movements, investigate reinforcement forces and force distributions, and to obtain pull-out test data. The tests are described in detail in the companion Paper. The test results were so encouraging (e.g. only 6mm of horizontal movement was recorded after excavation, and acceptable, predictable stress distributions were measured) that the Engineer approved the construction of the main wall.

The main wall consisted of driven 50 x 50 x 5mm angles in the upper 9 rows (5.5m long), and 60 x 60 x 6mm angles in the lower eight rows (7m long). Cuts were generally 1.2m high with the nails spaced at two per m². The angle steel reinforcement was bonded to the face mesh and then 50-80mm of sprayed concrete placed (Photograph 7). The main wall was instrumented and showed excellent performance, including:

- maximum vertical displacement 14mm at the crest, completely stabilised within 4 months
- pullout bond of 15 to 20kN/m length of reinforcement, largely independent of depth
- mobilisation of the maximum pullout force at displacements of a few millimetres.

In addition, the wall was subjected to static and dynamic surcharge loading. This confirmed that soil nailing can act as well as a Berlin wall or other type of anchored support.

4.4 Foundation excavation for the extension to the Good Samaritan Hospital, Portland, Oregon (1976)

(Shen *et al* 1981 a, b).

This contract was executed in the summer of 1976 and represents a significant milestone, being the first recorded application of soil nails in the USA. A joint venture of Kulchin and Associates Inc. and Albert K. Leung and Associates contracted to provide 2 140m² of nailed wall around three sides of a foundation excavation of maximum depth 13.7m (Photograph 8). The longest wall was 76.3m and up to 11.3m deep.

The ground consisted of medium to dense silty fine lacustrine sands with a friction angle about 36° and cementing giving a "cohesion"

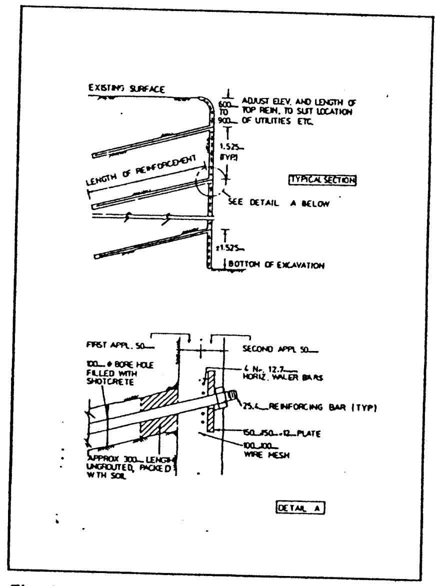


Fig. 21. Details of the soil nailing at the Good Samaritan Hospital site (Shen et al, *1981a)*

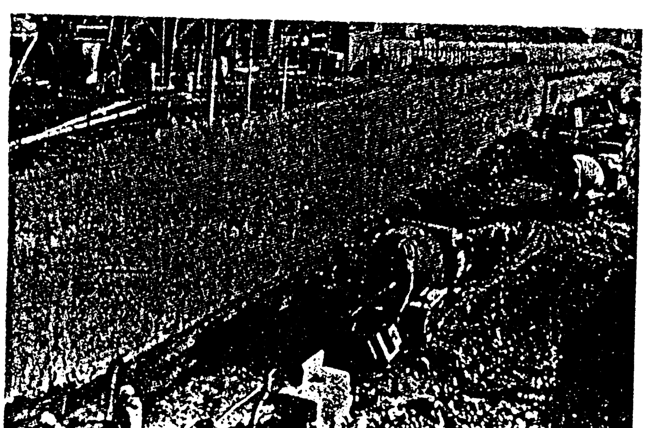


Photo. 7. General view of the Nogent-sur-Marne A86 excavation (Medio et al, *1983)*

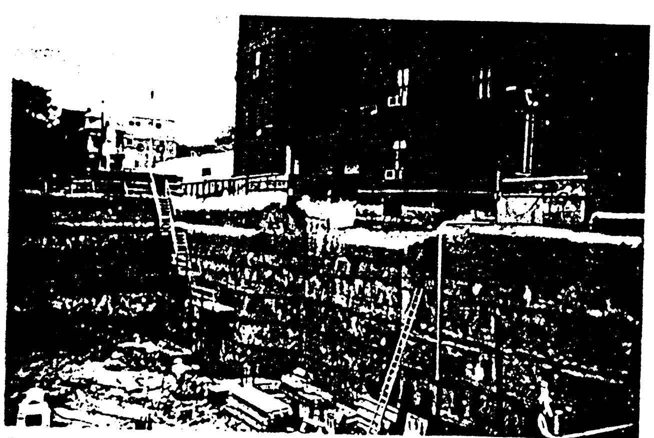


Photo. 8. View of the excavation next to the Marshall Street Annexe at the Good Samaritan Hospital, Portland (Shen et al, *1981a)*

of 20kN/m². No ground water was encountered.

Excavation proceeded in 1.5m cuts, at an average of 100m² per day. Mesh was placed 50mm off the face and sprayed with dry mix shotcrete (plus additive) to set in a few minutes and cure in 24 hours. An auger was used to drill the 683 holes 7-8.5m long and at an inclination 15° down. Centralised bars 25 and 38mm in diameter (Fig. 21) were inserted and grouted to within 0.3m of the face, before rows of four 12.7mm rebars were placed to form horizontal wales at each nail level. Following stressing of the bars to 50% to 80% of their design load, (bearing on 150mm square plates), a further 50mm layer of shotcrete was applied. For the lowermost cut, excavation by backhoe or by hand was limited to 6-12m runs, with 6m intervals (excavated and treated afterwards).

It was noted that the work was conducted in 50 to 70% of the period required for conventional support and at about 85% of the cost. Given the time saving, and the facility to cast the final wall straight on, the overall support and wall system costs were claimed (ENR, 1976) to have been reduced by about 30%.

Since the system was new and unfamiliar to local engineers and contractors, a limited amount of field instrumentation was installed by research workers from the University of California at Davis. An inclinometer placed halfway along the longest wall and 3m behind the face gave the results of Fig. 22: a measured maximum lateral movement of 33mm (= 0.30% of excavation depth, but later corrected for bottom movement to 0.32%). In addition, a crack survey on foundations of the adjacent Marshall St. Annexe showed tensile cracks of up to 8mm wide as far as 7.6m back from the face. However, it was noted that this building "remained fully occupied and functionable throughout the construction period". (Shen *et al*, 1981a).

4.5 Deep foundation for the PPG Industries HQ, Pittsburgh, USA (1982) (Nicholson and Wycliffe-Jones, 1984, Nicholson and Boley 1985).

The PPG Industries headquarters is located in the centre of Pittsburgh. About two thirds of the site required excavations 12-13m deep for parking facilities; the balance of the site had basements 3.6 to 5m deep. The traditional method of driven H-beams with timber lagging, supported by ground anchorages, was used around most of the site (6 000m²). However, in three separate areas, the proximity of existing buildings

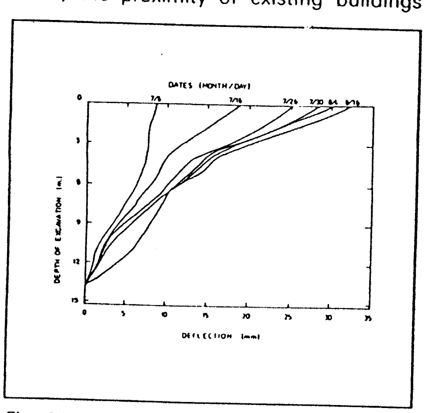


Fig. 22. Horizontal deflections measured on the western boundary wall at the Good Samaritan Hospital excavation (Shene et al, 1981a)

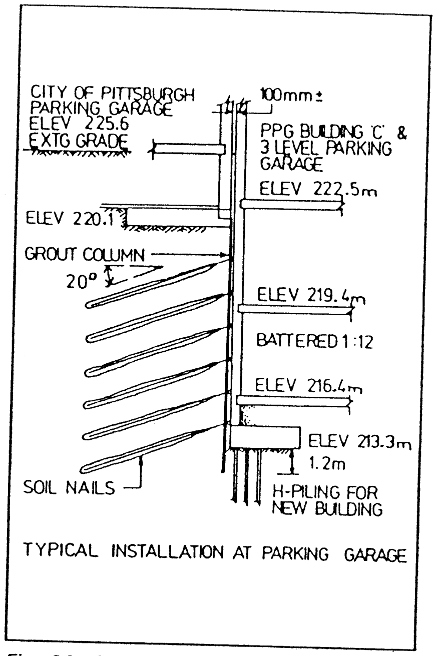


Fig. 23. Cross-section at the PPG Building Parking Garage excavation, showing the face pretreated by grouting (Nicholson and Wycliffe-Jones, 1984)

caused particular concern where the excavation level had to reach 3.3-10m below their foundations:

(i) The building of the Catholic Diocese of Pittsburgh main office; built in the 1950's, of masonry and brick construction, on spread footings on alluvial sand, silt and gravel.
(ii) Third Avenue Parking Garage: five stories high on spread footings.
(iii) Landmark Buildings: built around 1900, brick bearing wall, three stories high, with strip masonry and rubble footings.

The proximity of the new construction to the existing buildings precluded placing conventional H-beams and timber lagging for support. The beams would have interfered with the new columns and walls and, in some cases, would have actually been inside the parking garage. Conventional hand-dug underpinning was also considered but rejected on grounds of programme and costs. A solution based on soil nailing was selected modelled on the University of California, Davis studies.

The soil nailing technique was amended and adapted to local conditions and requirements as the work proceeded. In some areas, vibrations from the Main Contractor's excavation equipment induced local failures in the cohesionless granular soils. Local stability for the face of the soil nailing excavations was therefore improved by drilling fully cased holes of 127mm dia at 250-300mm centres along the line of the foundations. These holes were inclined at about 8° from the vertical to avoid interference with the new structure, were taken to 1.2m below final excavation level, and pressure grouted after the insertion of a steel reinforcing bar. Excavation in 1m cuts showed the grout penetration to be sufficient to prevent further ravelling or sloughing of the face. The installation of the nails then proceeded in the standard fashion (Fig. 23).

The particularly sensitive nature of the Diocese Building foundations required another level of protection against movement. Groups of vertical 450kN mini-piles were installed to support about half of the total vertical load of the foundation directly, before the excavation proceeded (Fig. 24).

Hydraulic rotary drilling with water flush was used throughout the contract with control of drill hole alignment being a critical aspect. Pressure grouting to 2bar was conducted with cement grouts. The 6-8m long nails had a plastic sheath over the upper 3m to debond the bar and facilitate nominal stressing. The average nail spacing was a 1.2m square grid, with each horizontal row connected to the grout columns by a steel waling. The maximum depth supported was over 9m, involving six rows of nails.

After full excavation, the foundation piling for the new building was commenced with 350mm H-piles driven within 0.5m of the grout columns. Neither the excavation itself, nor the piling activities caused any detectable damage or movement to adjacent structures.

REMEDIAL CONSTRUCTION

4.6 Repair of a reinforced earth wall, Fréjus Tunnel France (1981) (Long *et al*, 1984)

The 5km long access road to the Fréjus tunnel involves over 10 000m² of Reinforced Earth retaining walls and was completed in 1978. In March 1981 a sudden collapse of about 60m² of a 7-8m high section of the wall was recorded (Fig. 14).

The failure involved a section 9-10.5m wide and 6m high near the top of the wall. It resulted from the breakage of the reinforcement strips at their connection with the facing panels. A saturated zone of backfill 1m to 2m deep failed with the facing, but behind this the structure appeared perfectly stable.

In the centre top of the collapse there was a grid manhole, 2m deep, which conducted the running water towards the main sewer of the road. It was observed that no rebar had been placed to tie the wall of the manhole to

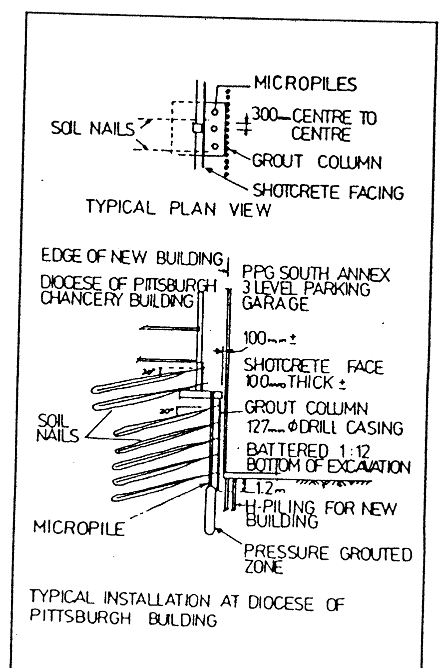


Fig. 24. Excavation adjacent to the Diocese Building showing direct underpinning through the nailed zone to minimise settlement (Nicholson and Wycliffe-Jones, 1984)

the bottom slab. Near the failed area, bulges could be observed in several areas of the facing.

Two types of repair were conducted:

1. In the failed zone, the facing was rebuilt by placing a grid reinforced concrete panel and connecting it to the reinforcing strips left in the backfill.
2. The facing in the deformed areas was stabilised by soil nails drilled through the centre of the facing panels and grouted into the fill beyond (each nail equated to four strips). The soil nails were 28mm diameter Diwidag bars 5m long, torqued to a nominal stress of 20kN. The grout was almost 1:1 sand cement (w = 0.58) chosen to be compatible with the granular fill material. The grouting was executed in two phases, the second at pressures of up to 5bar, to ensure uniform filling of any voids near or behind the bulged facings.

Monitoring of the wall after completion of the remedial work led to the conclusion that the original failure had occurred due to the expansion of the backfill under frost action near the facing, the backfill being supplied with water from the defective manhole.

Although examples of failure in reinforced earth structures are few, Garcia-Goytia and Guitton (1979) had reported a full-scale test to demonstrate the suitability of soil nailing to improve the internal stability of reinforced earth walls threatened by corrosion or other damage to the metal strips. These tests formed the basis of the design for the repair at Fréjus.

4.7 Restoration of a drystone retaining wall, Bradford UK (1985)

(Previously unpublished).

A 125m long drystone masonry retaining wall, at Denholme Clough, near Bradford, is typical of many thousands of kilometres of structures built in West Yorkshire and neighbouring counties, during the last 200 years. The wall is 2-3m high, retains sloping soil and fill, and bears on a weathered sandstone bedrock. The progressive failure and collapse of such walls represents a major problem to the highway authorities since there may be (i) physical danger resulting from a collapse, (ii) disruption to traffic, and (iii) high cost for clearance and rebuilding works.

At Denholme Clough the wall had actually collapsed in places whilst in other areas there was significant bulging indicative of imminent failure. In order to assess the practical and economic benefits of using soil nailing to repair drystone walls the then West Yorkshire Metropolitan C.C. let a contract to repair the wall using this technique.

The following sequence of operations was conducted (Fig. 15):

Drainage. A series of 50mm diameter temporary drain holes were drilled 300mm above the road level at 2m horizontal spacings. After shotcreting and nail installation, two rows of permanent 115mm dia. drain holes were drilled 300mm and 1 500mm above road level extending to the back of the drystone wall at 3m horizontal spacings.

Repairing wall facing. Sections of wall that had collapsed were rebuilt in a single skin brickwork with strengthening piers. The void behind was backfilled with a sand cement grout with Conbex 653 additive to restrict flow. A layer of light steel mesh was fixed to the drystone wall and 50mm thickness of concrete was sprayed to hold the drystones in place during the subsequent nailing. Vertical construction joints were formed at 15m centres.

Nailing. A track-mounted drilling rig was used to drill the 115mm diameter nail holes, with one nail for 2.5m² face. The reinforcement was 16mm diameter high yield bar with a threaded end, and the grout was a neat cement mix (w = 0.45). A corrugated PVC sleeve was placed through the upper section of each hole to restrict lateral travel of grout in the masonry. After setting of the grout, a headplate was fitted and a nominal stress applied to each nail. Pull-out tests were conducted on three test nails, and confirmed that the bond was well in excess of the required working conditions.

New masonry face. A new stone facing was then erected as a finish to the stabilised wall.

This solution for the repair of drystone walls without demolition was found technically and commercially attractive. Construction details could be varied; for example the old facing could be retained by setting the nail heads into the drystone wall itself.

5. Tabulated data from published case histories

Published information on case histories which contain construction details are presented in Table 1 (projects in *granular soils*) and Table 2 (projects in *overconsolidated clays and mixed soils*, mostly Marls and Moraines). The tables identify separately nails installed by drilling and grouting, and nails installed by being driven directly into the ground.

Table 3 summarises two soil nailing projects which failed, and the table allows direct comparison between the failed nail cross-section and the repair cross-section. Three cases where soil nailing has been used in remedial works are given in Table 4 (two of these are described in detail in the previous section).

5.1 Derived parameters

Four derived parameters or ratios are also given in the tables for each reported project. The detailed justification for selecting these parameters is provided in the companion Paper. The aim of the derived parameters is to allow comparison between the design of different projects. The parameters describe the following features of a soil nailing design:

1. The overall geometry of the structure,

$$\textbf{Length Ratio} = \frac{\text{Maximum nail length}}{\text{Excavation height}} = \frac{L}{H}$$

2. The nail surface area available to bond with the soil,

$$\textbf{Bond Ratio} = \frac{\text{Hole diameter x Nail length}}{\text{Nail spacing}} = \frac{(d_{hole})L}{\text{spacing}}$$

where the spacing is the nominal vertical area of face supported by each nail.

3. The strength of the nail arrangement. For steel reinforcement this can be expressed as the ratio of the area of steel to the area of soil. For bar reinforcement, this may be represented by the parameter:

$$\textbf{Strength Ratio} = \frac{(\text{Nail diameter})^2}{\text{Nail spacing}} = \frac{(d)^2_{bar}}{\text{spacing}}$$

4. The performance of the nailed structure. The most frequently made measurement is the outward movement of the top of the excavation, leading to:

$$\textbf{Performance Ratio} = \frac{\text{Outward movement}}{\text{Excavation height}} = \frac{\delta_{horizontal}}{H}$$

Where the nail is not a solid circular bar, equivalent values for the nail diameter and the hole diameter have been calculated and entered in the tables. The equivalent nail diameter gives an equal steel area, and the equivalent hole diameter gives an equal surface area for bonding with the soil. The conversion factors for the "Hurpinoise" angle steel reinforcement are noted at the foot of the tables.

There are many other references to soil nailing projects in the literature, and some of these contain interesting information. For this Paper, however, case histories have only been tabulated where the soil and nailing cross-sections have been fully described.

Details of the field trials and experimental soil nailing structures which have been built are presented and discussed in the companion Paper on performance and design, where the observations made below are also considered in more detail.

5.2 Observations on case histories

A few general observations may be made based on the tabulated data:

5.2.1 Steep granular slopes

For steep slope (80° or more) projects in granular soils there is a reasonable correlation of the derived parameters as shown in Table 5.

TABLE 5: Comparison of drilled and grouted, and driven nails for steep slope case histories in granular soil. (Data from Table 1)

	Drilled and grouted	*Driven*
Length Ratio	0.5-0.8	0.5-0.6
Bond Ratio	0.3-0.6	0.6-1.1
Strength Ratio (10^{-3})	0.4-0.8	1.3-1.9
Performance Ratio	.001-.003	No data

Overall, for projects in granular soils, the driven nails are slightly shorter than those which have been drilled and grouted. Probably to compensate for the relatively smooth surface of driven nails about twice as much surface area is provided for bonding with the soil than is the case with the drilled and grouted nails.

The most striking difference, however, is in the strength ratio which shows that about three times as much cross sectional area of steel is used with driven nails compared to drilled and grouted nails. At least part of this, however, must be caused by providing more surface area for bonding with the driven nails.

The performance ratio for drilled and grouted nails show consistently an outward movement of up to 0.3% of the excavation depth. Similar excellent performance would be expected for the driven nails although no measurements were reported on the commercial projects, (but see the

TABLE 6: Comparison for drilled and grouted nails for steep slope case histories in granular soils and Moraine or Marls. (Data from Tables 1 and 2)

	Granular soils	*Moraine and Marl*
Length Ratio	0.5-0.8	0.5-1.0
Bond Ratio	0.3-0.6	0.15-0.20
Strength Ratio (10^{-3})	0.4-0.8	0.1-0.25

experimental projects in the companion Paper).

5.2.2 Comparison between projects in granular soils and stiff clays

For drilled and grouted nail projects, less bond and less strength are provided for the excavations in Moraine or Marl than for the excavations in granular soil. The results are shown in Table 6.

Although the length ratio is similar for projects in the two types of soil, about two or three times less surface area for bonding is provided in the Moraine and Marl projects. The cross-sectional area of steel used to stabilise the Moraine and Marl excavations is about four times less than was the case for granular soils.

By comparison the one project in Moraine using driven nails had a similar bond ratio and strength ratio to the typical values for driven nails in granular soil.

5.2.3 Comments on reported failures

The failure at Les Eparris (Table 3) is well-documented and the slip was due to lack of available bond between the reinforcement and the clay. This is reflected in the repair cross section where the bond ratio is increased by a factor of three but the strength ratio is little changed.

Much less information is available for the Gare du Nord failure, but both the bond ratio and the strength ratio were increased in the repair cross section by a factor of two to three.

6. Construction

The purpose of this section is to highlight aspects of soil nailing construction which may be considered as being good practice, or which are regarded as having potential for future application.

6.1 Excavation and facing

The maximum cut depth at each level of excavation is dictated by the ability of the exposed face to "stand up". In addition, where deformation must be minimised, the cut depth may be reduced to the smallest value consistent with site practicalities and commercial considerations. Cut depths of more than 2.0m or less than 0.5m are rare in granular soils. Greater cut depths have been used in overconsolidated clays.

A level working bench at least 6m wide should be provided for the nailing equipment. Usually the length of a single cut is dictated by the area of face which can be stabilised in the course of a working shift. Where deformations must be minimised, the nailing may be executed in alternative primary and secondary cut lengths, which might typically be 10m long.

The excavation equipment should minimise the disturbance of the ground to be retained and must provide a reasonably smooth and regular slope profile. Any loosened areas on the face should be removed prior to the facing support being applied. Pretreatment in the form of grouting (such as at PPG, Pittsburgh) may be necessary in loose or dry soils without natural cohesion, especially where the face is subjected to external vibrations. In this context, the possible effects of blasting in adjacent areas must be evaluated.

As a rule, the face support must be placed at the earliest time to prevent relaxation or ravelling of the ground. Typically this involves pinning a reinforcement mesh to the face and spraying a concrete cover before drilling the nail holes. In the "Hurpinoise" system the angle steel reinforcement is often driven before placing the mesh and sprayed concrete.

The final face thickness varies from 50-150mm for temporary applications to 150-250mm for permanent projects. The face may be built up in one, two or more layers, depending on the nail type, and the construction and stressing sequence. Short bars may be driven into the face before spraying to serve as a depth gauge for the sprayed concrete. Architectural finishes may be applied with a final layer of sprayed concrete say 50mm thick to blend colour, or with larger aggregate to give a rugged finish.

Both "wet" and "dry" sprayed concrete may be used depending on the scale of the project and the availability of equipment and materials. Maximum aggregate sizes of 10-15mm are usually specified and admixtures are often incorporated to accelerate set, or less commonly, to reduce creep of the hardened concrete. Minimum cement contents of 300kg/m^3 are typical. Control "panels" or boxes are recommended for on site quality assurance, at frequencies of about one per 100m^2. Accelerated shotcrete should give an unconfined compressive strength of around 5N/mm^2 in 8 hours, whilst it is best to let it cure for 24 hours prior to further works. The proper curing of the sprayed concrete face is important if surface cracking is to be avoided.

Spraying is often discontinued about 300mm above the bottom of the cut. This facilitates the fastening of the mesh for the next lower cut, and an overlapped construction joint for the sprayed concrete, which is further aided by chamfering at 45°.

6.2 Drainage

An early aid is to excavate a drainage ditch along the crest of the excavation to lead away surface water. The ditch may be lined with concrete during the spraying of the first cut. Thereafter, there are three main types of drainage for the retained soil mass:

1. *Shallow drains:* tubes 300-400mm long, to release water immediately behind the facing. These drains are usually about 100mm in diameter and their spacing depends on the groundwater conditions and the likelihood of frost damage.
2. *Deep drains:* slotted tubes, usually longer than the nails, about 50mm in diameter and inclined upwards at 5 to 10°. Their spacing depends on the soil and groundwater conditions but is typically less than one per 3m^2 of face.
3. *Face drains:* these are placed vertically against the cut slope at regular horizontal intervals before spraying the concrete face. The spacing depends on groundwater conditions and the threat of frost or ice action, but may typically be between 1 to 5m. These drains are extended continually over the full height of the excavation and are connected by overlaps at the bottom of each successive cut. At the base they discharge into a collector system with weep holes. The drains may be prefabricated from geotextiles and need protection against impregnation by the sprayed concrete with, for example, a polyethylene sheet backing. Face drains are an alternative to the shallow drains described above.

6.3 Installation of nails

In many respects the "good practice" recommendations or stipulations of codes of practice covering ground anchorages would be applicable for soil nails. The recommendations of DD81 (1982) would therefore apply to works in the United Kingdom.

Drilling techniques and methods vary with the ground conditions, the geometry of the installation, and the resources and experience of the contractor. The most common systems (excluding simple "open hole" methods such as uncased rotary, or down-the-hole hammer) are;

Duplex drilling: this rotary or rotary percussive method involves the simultaneous advancement of a temporary outer casing and an inner drill rod (Bruce 1984). Water or air flush is usually employed, although care is needed with air flush in urban environments. Diamond drilling may be necessary initially when installing nails in remedial works through existing rock or concrete-faced structures.

Auger drilling: this rotary method is commonly used in clay soils without boulders or in cemented sands. In unstable conditions the reinforcement and grout can be introduced through a "hollow stem" auger during withdrawal of the string.

Based on the experience from ground anchorages, the temporary support or boreholes by bentonite or other mud suspensions is not recommended as "smear" on the borehole walls may reduce the subsequent grout-to-ground bond.

Clearly, with the percussively-driven reinforcements of the Hurpinoise system, no predrilling is required. In favourable conditions, therefore, the rate of installation can be very high. Directly driven nails may be less suitable in boulder clays or very dense, cemented soils. Also, care must be taken in loose, and weakly-cemented, granular soils to ensure that driving does not cause local destructuring of the soil around the nail which could result in low values of bond stress.

Most recently Louis (1984, 1986) has reported patented* systems of nail installation for which very high rates of production are claimed. In the "Jet Bolting" system (Fig. 25) very high pressures, over 200bar, are used to inject cement grout through small apertures at the tip of the nail whilst it is being installed or percussed into the soil. This jet grout lubricates the penetration of the nail, and upon setting is claimed to provide an enhanced bond capacity for the nail. An improvement of the performance of loose sand or soft clay between nail locations is also claimed, but is as yet unquantified. Jet bolting systems have not yet had significant commercial application outside Southern France, to the authors' knowledge.

In general, borehole diameters range from 76 to 150mm for drilled and grouted nails. This usually permits a grout annulus of at least 20mm thickness around the reinforcement providing a degree of corrosion protection. As nails are relatively quite short and close together, the drilling tolerance does not have to be as precise as it is for ground anchorages, and this allows higher production rates. Holes inclined downwards (even as little as 10-15°) are easier to grout effectively than those which

There are no proprietary restrictions on the use of soil nailing (although the Laws of Copyright will, of course, apply). Certain specific construction elements, such as some special prefabricated facings, and special reinforcement installation techniques are, however, subject to proprietary restrictions.

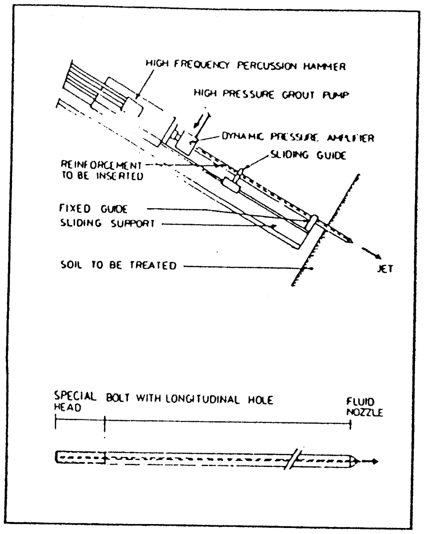


Fig. 25. The "Jet Bolting" technique for soil nail installation which combines vibro-percussion and high grout pressure at the nail tip (Louis, 1984)

TABLE 8: Typical nail dimensions and properties in North American practice

Bar size Designation Number	Diameter Inches	Diameter mm	Weight Kg per metre
5	0.63	15.9	1.55
6	0.75	19.1	2.24
7	0.88	22.2	3.05
8*	1.00	25.4	3.98
9*	1.13	28.7	5.07
10*	1.25	31.8	6.41
11*	1.38	35.0	7.92
14	1.75	44.5	11.4
18	2.26	57.2	20.3

Minimum yield stress = 415N/mm^2
*These numbers most commonly used in soil nailing

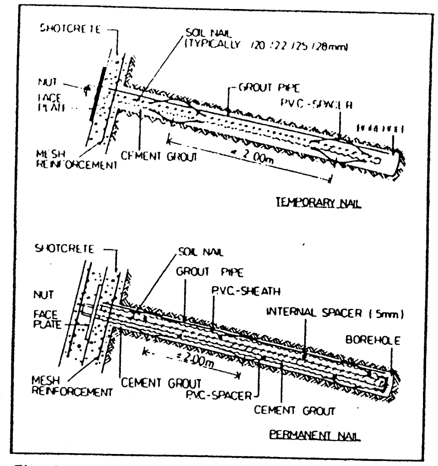


Fig. 26. Examples of good practice for drilled and grouted nail installation (a) for temporary applications and (b) for permanent applications (Based on West German experience, reproduced with permission of Bauer, AG)

are horizontal or inclined upwards. However, Jet bolting can operate equally well over a range of inclinations.

Grouting is usually carried out with stable, cement-based grouts (w = 0.4-0.5) under gravity or very low excess (less than 5bar) pressure. The use of higher pressure is often restricted by the risk of hydrofracture or leakage. Also, the potentially beneficial effects on bond of higher grout pressures (Ostermayer, 1974) would probably not justify the higher costs for most soil nailing installations. The reinforcement should be placed and the grouting completed with the minimum delay after drilling.

6.4 Reinforcement and corrosion protection

Although polymer-based reinforcements such as plastic rods or fibreglass could be used for soil nailing, steel reinforcement has been applied in practice to date.

The driven elements in the Hurpinoise system are generally mild steel angles with tensile capacities 50-150kN, and dimensions 50 x 50 x 5mm or 60 x 60 x 6mm. Higher strength solid steel bars (Tables 7 and 8) are used for nails grouted into predrilled holes.

For temporary applications in standard environments, corrosion protection is usually provided only by the grout, and sometimes with an epoxy coating to the steel surface (Fig. 26a). For permanent works, the degree of protection may be increased by providing

TABLE 7: Typical nail dimensions and properties in European practice

	Diameter (mm)	Yield Stress (N/mm^2)	Ultimate Stress (N/mm^2)
DYWIDAG	26.5 32.0 36.0	835	1 030
DYWIDAG	26.5 32.0 36.0	1 080	1 230
GEWI	22.0 25.0 28.0 40.0	420	500

an outer sheath of plastic material, ensuring an inner annulus of at least 5mm thickness (Fig. 26b). Other proprietary systems have also been developed to overcome the potential problems arising from microfissuring of the grout under tension (e.g. the "Intrapac" nail of Intrafor-Cofor). In all cases, centralisers are placed at regular intervals (say 2m) along the reinforcement to ensure concentricity with the borehole.

It is interesting to compare the approach in codes of practice dealing with ground anchorages and with reinforced earth. All international codes on ground anchorages require protection by at least one sheathing over the tendon. Conversely, for permanent installations in codes for reinforced earth the galvanised steel strips are allowed to remain in direct contact with the soil (SETRA, 1979, DTP, 1978).

One of the most recent studies on corrosion in reinforced earth (FHA, 1985) has demonstrated once more that the understanding of corrosion mechanisms for metals is incomplete and that long-term problems can occur. This would suggest that good practice for permanent soil nailing installations should require direct protection by at least one sheathing along the lines developed, and codified, for ground anchors. This type of permanent nail installation is illustrated in Figure 26b.

A thorough summary of corrosion mechanisms and protection has most recently been provided by the FIP state-of-the-art review "Corrosion and corrosion protection of prestressed ground anchorages" (1986), prepared by a working group under the chairmanship of Prof. Littlejohn.

Because of the way soil nails work – virtually their whole length is bonded to the soil and available for load transfer – it is unnecessary to apply significant degrees of post-tension after installation. Typically a load of about 10% of the working load would be locked in, with a torque wrench and lock nut arrangement. This tension is applied to "seat" the soil/facing/nail system so that it acts in direct response to soil deformation. Since the "lock off" loads are relatively low the steel bearing plates are quite light (150 x 150 x 10mm or 200 x 200 x 10mm), and stiff wales are not required. The nominal post-tensioning is normally applied during or just after the installation of nails in the cut immediately below.

6.5 Slope claddings

To date most applications of soil nailing have been temporary and so the appearance of the nailed structure has not been a significant consideration. However, there are an increasing number of applications where precast or prefabricated facing units are being used to facilitate construction, improve appearance, provide better long term durability, or to enhance noise absorption.

Facing panels may be placed directly in contact with the slope face during construction. Alternatively, the excavation may be completed with a normal sprayed concrete facing and then covered by precast panels. Drainage arrangements are often attached directly to the back of facing panels.

Exactly as for reinforced earth, the benefits of a prefabricated facing include fabrication under controlled conditions to ensure high quality, and the wide range of shapes and materials which may be used to give an attractive, individual finish.

The combination of vegetation with an open or terraced structural facing is also used, and this has great potential for providing an environmentally sensitive finish to a permanent cutting or excavation.

6.6 Instrumentation and monitoring

In contrast to ground anchorages, it is not necessary to check each individual soil nail. This reflects the fact that it is the overall performance of the soil nails which is important. Selected nails should be subject to pull-out tests during each level of excavation to verify the design assumptions on bond capacity. Louis (1986) recommends for good practice that 4 or 5 short bars should be installed and tested for pull-out capacity in each type of soil to be excavated at a site, before the main contract starts.

By strain gauging individual nails, the development and distribution of the nail forces may be measured, which provides vital feedback to designers. Load cells at the nail head also provide useful data, particularly where near surface effects are important, such as freezing.

The most significant measurement of overall performance of the system is the deformation of the wall or slope during and after construction. Slope inclinometers at various distances back from the face provide the most comprehensive data on ground deformations. The face movements can be measured directly by surveying, and prisms attached to selected nails permit electronic distance measurements to be made.

Continual monitoring of the ground during the progress of the works allows the actual performance to be checked against the design assumptions. It also provides a continuous record of performance, thereby allowing modification of the construction details in response to changed conditions, most importantly if poorer soils are encountered.

7. Conclusions

In continental Western Europe and North America, soil nailing is embraced by practicing engineers as a highly competitive, well proven construction technique. The origins of soil nailing lie in the New Austrian Tunnelling Method, which is still a popular and efficient system for soft ground tunnelling as it enters its third decade of application.

Soil nailing has certain similarities to both reinforced earth and anchoring, although its particular operating principles and construction methods give it a firm and distinct identity. Similar considerations distinguish it from allied insitu soil reinforcing techniques such as reticulated root piles and soil dowelling.

Most applications of soil nailing to date have been associated with new construction projects such as foundation excavations and slope stabilisation, for both temporary and permanent works. The system has equal facility in a wide range of remedial projects, and indeed it is most likely that nailing will find its earlier applications in the United Kingdom in this field, bearing in mind the prevailing economic trends.

It is to be hoped that the growth of the technique in the United Kingdom can be fostered by practical research collaborations between industry, the universities and government, in the manner of France, West Germany, and the United States of America, the current leaders in the field.

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