11 In-situ earth reinforcing by soil nailing

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11.1 Introduction

Since the late 1960s, engineers in Europe, Japan and North America have been exploiting the special advantages of the technique of soil nailing.[1] This geotechnical engineering process comprises the in-situ reinforcement of soils and has a wide range of applications for stabilizing excavations such as are associated with deep foundations or cut and cover tunnelling schemes. It has been researched with large budgets since 1975 by collaborations of contractors, universities and government organizations. 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 worldwide. There are abundant successful case histories to cite in a wide variety of ground conditions and applications, and 'first uses' have been reported recently in such diverse locations as South Africa,[2] New Zealand,[3] and Hungary.[4]

11.2 Characteristic features

Soil nailing is a practical and proven technique used in constructing excavations by reinforcing the ground in-situ with relatively short, fully bonded inclusions—usually steel bars. These are introduced into the soil mass as staged excavation proceeds, 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 (Figure 11.1).

11.2.1 In-situ reinforcement techniques

There are three main categories of in-situ reinforcement used to stabilize slopes, namely nailing, reticulated micropiling, 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 (Figure 11.2(a)). One major variant, the 'Hurpinoise' method,[7] involves simply driving an angle section into the soil, without predrilling and grouting.

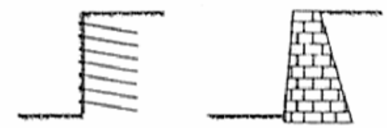

Figure 11.1 The analogy between a gravity retaining wall and a soil-nailed wall.[5]

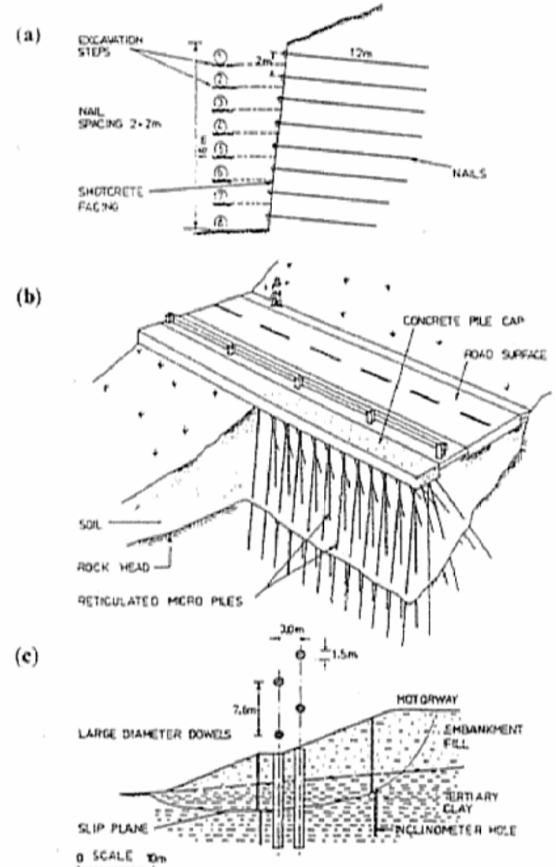

Figure 11.2 The family of in-situ soil reinforcement techniques: (*a*) Soil nailing (after Schlosser[6]); (*b*) reticulated micropiling (after Boley and Crayne[8]); and (*c*) soil dowelling (after Gudehus and Schwarz[11]).

This chapter, however, focuses on the drilled and grouted version—by far the most popular, widely used, and usable version, given practical concerns with installation and corrosion protection.

Reticulated micropiles are steeply inclined in the soil at various angles both perpendicular and parallel to the face (Figure 11.2(b)). 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 compression and tension, bending, and shearing forces. Fondedile's pali radice system was the original form of this construction[9] whilst, more recently, Nicholson Construction has applied a similar technique in the USA under the trade name *Type A INSERT Wall.*[10]

Soil dowelling is applied to reduce or halt downslope movements on well-defined shear surfaces (Figure 11.2(c)). Slopes treated by dowelling are typically much flatter than those in soil nailing or reticulated micropile applications. Gudehus[12] has 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 micropile.

11.2.2 Selecting in-situ reinforcement

Although there are fundamental differences in the mechanical action of these three in-situ reinforcement techniques, there are circumstances where more than one may be applied to slope stabilization as illustrated in Figure 11.3. The following points merit consideration when choosing the appropriate in-situ reinforcement technique.

Laboratory experiments (e.g. Jewell[13]) have shown the influence of the inclination and properties of reinforcing members on the shearing resistance of reinforced soil. 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 11.3(a). To stabilize the soil with reinforcement placed in substantially vertical directions (Figure 11.3(b)) will require a much higher density of reinforcement. For this type of application soil nailing is likely to be more cost-effective than reticulated micropiling.

In marginally stable granular or scree slopes when stability must be improved, but where excavation is not foreseen, then either soil nailing or Type A INSERT Walls would be appropriate. Where drilling equipment cannot be placed on the slope, Type A walls would be best (Figure 11.3(c)). Where access is not problematical, either technique could be applied (Figures 11.3(c) and 11.3(d)), with economic considerations being decisive.

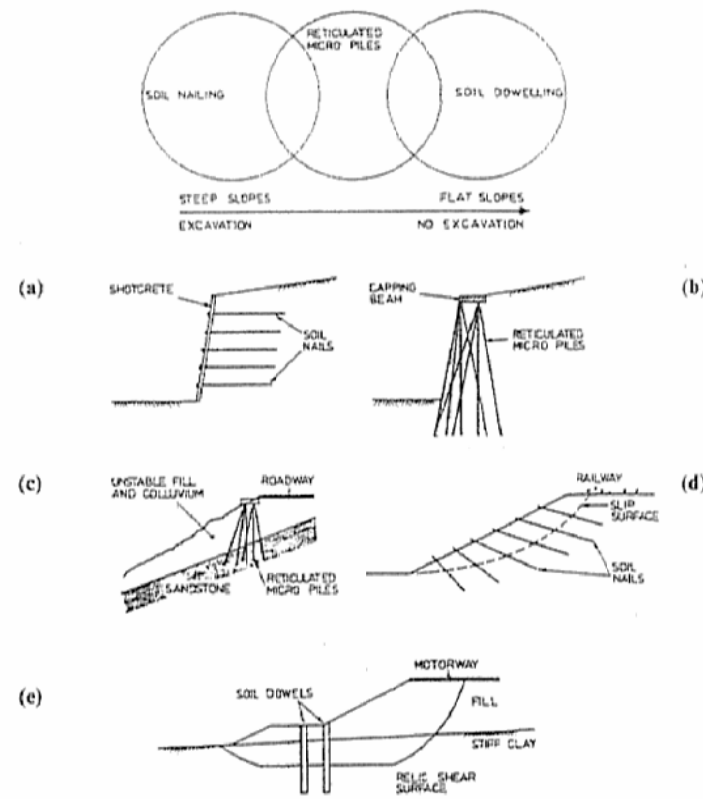

Figure 11.3 Overlap of in-situ soil reinforcement applications: (*a*) and (*b*) in excavations; (*c*) and (*d*) for general slope stabilization; and (*e*) to stabilize residual slips in clay.[1]

In flatter clay slopes where stability is governed by a well-defined shear zone, larger diameter soil dowels would be most appropriate (Figure 11.3(e)), possibly in concert with a RODREN[14] type deep well concept (Figure 11.4).

Type A walls and soil dowelling are not described further in this chapter. The former are described in various publications.[8–10,15–17] Soil dowelling is described in references 12 and 18–20.

11.2.3 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:

(i) the reinforced zone must be able to resist the outward thrust from the unreinforced interior, without sliding;
(ii) the combined loading from the reinforced zone self weight, and the

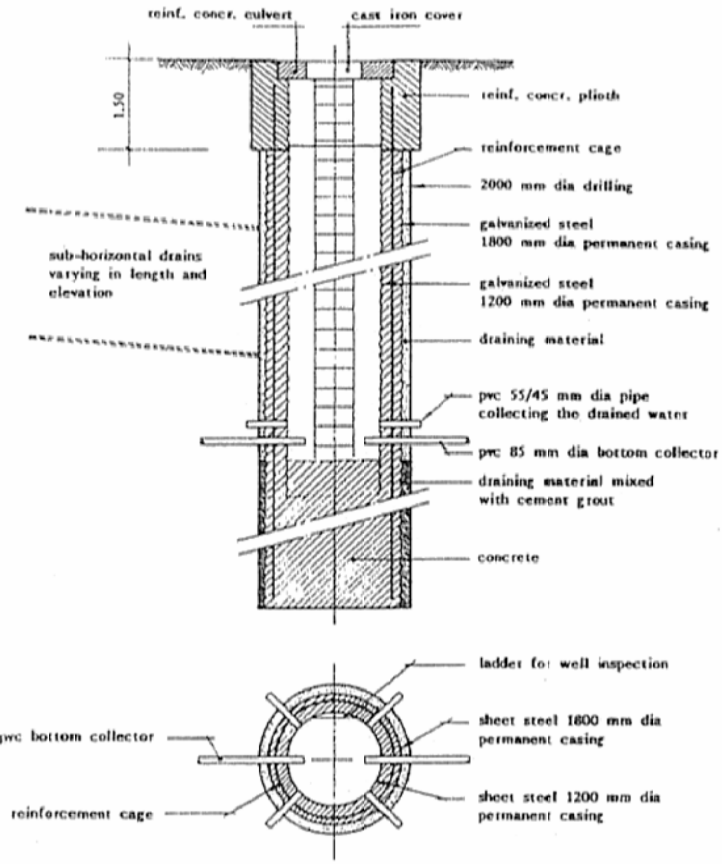

Figure 11.4 Structural RODREN drainage well.[14]

lateral soil thrust it is resisting must not cause a foundation bearing failure; and

(iii) the stability of the retaining structure must be checked against the deeper seated overall failure machanisms.

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:

(i) 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

(ii) 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 referred to in subsequent sections.

11.2.4 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 drawn, which will favor the choice of the one over the other:

(i) Ground anchorages are stressed after installation so that in service they ideally prevent any structural movement occurring. In contrast, soil nails are not prestressed and require a finite (albeit very small) soil deformation to cause them to work.

(ii) Nails are in direct, bonded contact with the ground over most of their length (typically 3–10 m), 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 soil mass is different for each type.

(iii) Since nails are installed at a far higher density (typically one per 2–3 m^2 of face) 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.

(iv) As high loads have to be preapplied 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, as the low individual head loadings are easily accommodated on small steel-bearing plates placed on the shotcreted surface.

(v) Individual anchorages tend to be longer (say 15–45 m) and so may necessitate larger scale construction equipment. Also, an anchorage system is often provided to stabilize a substantial retaining structure, such as a diaphragm wall or bored pile wall, which will itself necessitate large-scale equipment.

As is noted in section 11.2.6, certain soil conditions are not suited to nailing, whereas anchoring is applied more widely. In addition, nailing appears more limited in terms of excavated depth potential. If the overall stability calculations show the problem to be deep seated, then ground anchorages will most probably be required in place of, or together with, nails. 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), in appropriate geological, geometrical and performance conditions.

11.2.5 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,[21] there are also some fundamental differences.[6]

The main similarities are:

(i) The reinforcement is placed in the soil unstressed and is not then prestressed: the reinforcement forces are mobilized by subsequent deformation of the soil.
(ii) 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.
(iii) 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:

(i) 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' (Figure 11.5). This has an important influence on the distribution of the forces that develop in the reinforcement, particularly during the construction period.
(ii) Soil nailing is an in-situ reinforcement technique exploiting natural ground, the properties of which cannot be preselected and controlled as they are for reinforced earth fills.

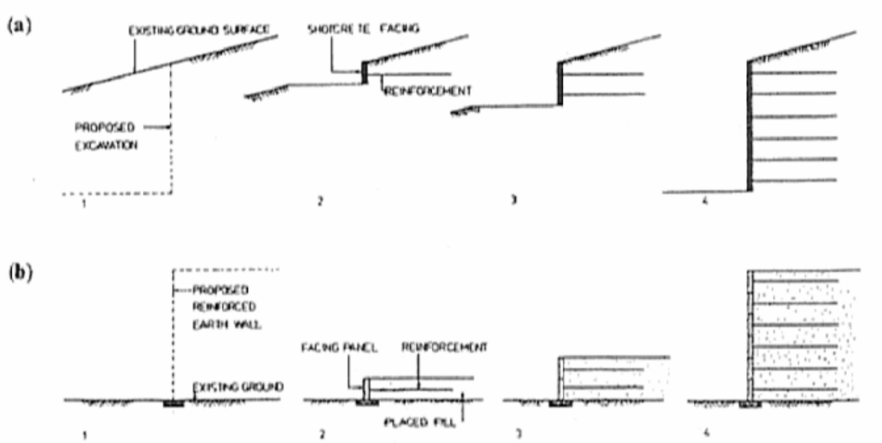

Figure 11.5 Contrast of the construction sequence: (*a*) 'top down' in soil nailing; and (*b*) 'bottom up' for reinforced soil.[1]

(iii) 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 itself.

11.2.6 Benefits and limitations of soil nailing

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

(i) Economic advantage—it would seem that the cost saving for excavations of the order of 10 m deep can be 10 to 30% relative to conventional underpinning, or an anchored diaphragm or Berlin wall alternative in appropriate conditions. This is supported by a claimed saving of 30% on a soil-nailed excavation in Portland, Oregon,[22] and 35% in a more recent San Francisco excavation[23] featuring an underpinning application.
(ii) Construction equipment—drilling rigs for reinforcement installation, and guns for shotcrete application are relatively small-scale, mobile and quiet. This is highly advantageous in urban environments where noise, vibration or access restraints may pose problems. Equally, in remote rural areas it may prove impossible to deploy large-scale equipment such as is needed for piling or diaphragm walling.
(iii) Construction flexibility—soil nailing can proceed rapidly (150–200 m^2 per shift) and the excavation can be shaped easily. It is a flexible technique, readily accommodating variations in soil conditions and work programs as excavation progresses.
(iv) Performance—field measurements indicate that the overall movements required to mobilize the reinforcement forces are surprisingly small. These generally correspond to the movements to be expected for well-braced systems (Category I) in Peck's[24] classification (Figure 11.6). Furthermore, nailing is applied at the earliest possible time after excavation, and in intimate contact with the cut soil surface. This minimizes the disturbance to the ground, and so the possibility of damage being caused to adjacent structures.

Naturally, the technique has certain practical limitations to its application:

(i) Soil nail construction requires the formation of cuts generally 1–2 m high in the soil. These may then have to remain 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 stabilize the face, but this will add both complication and cost.

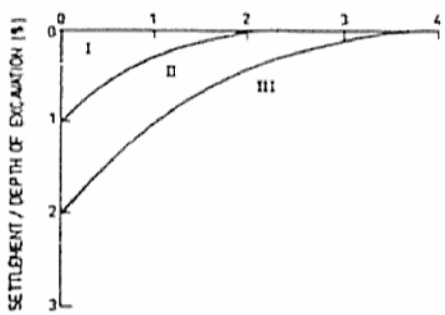

Figure 11.6 Field performance of open excavation systems.[24] I: well-braced systems; II: intermediate performance, e.g. temporary berms and raker support; III: systems permitting ground loss.

(ii) A dry face in the excavation is desirable for soil nailing. If groundwater percolates through the face, the unreinforced soil will slump locally on initial excavation, making it very difficult to establish a satisfactory shotcrete skin.

(iii) Excavations in soft clay are also unsuited to stabilization by soil nailing. The low frictional resistance of soft clay would require a very high density of in-situ reinforcement, of considerable length to ensure adequate levels of stability. Bored pile or diaphragm walls with anchorages are more suited to these conditions.

11.3 History and evolution

The principles and techniques of stabilizing excavations in rock by in-situ reinforcement have long been applied by mining engineers. Beveredge[25] noted that the use of mechanical rock bolts grew immediately after World War II, whilst by 1959 the first fully bonded (resin) reinforcements were being installed in German mines. The New Austrian Tunnelling Method,[26] evolved in the early 1960s primarily as a hard rock tunnelling system using a combination of shotcrete and fully bonded steel inclusions to provide early, efficient excavation stability with minimal movement. It was later adapted successfully to less-competent formations comprising graphitic shales—as in the Massenberg Tunnel[27]—and Keuper Marl—as in the Schwaikhem Tunnel.[28]

This latter project confirmed the viability of the technique in less-competent materials, and soon trials were conducted in soil such as silts, gravels and sands, and in very weak rocks. The earliest applications were in small cross-section metro tunnels in Frankfurt in 1970. Soon after, in Nürnberg, the technique again proved successful in the construction of a double tube of

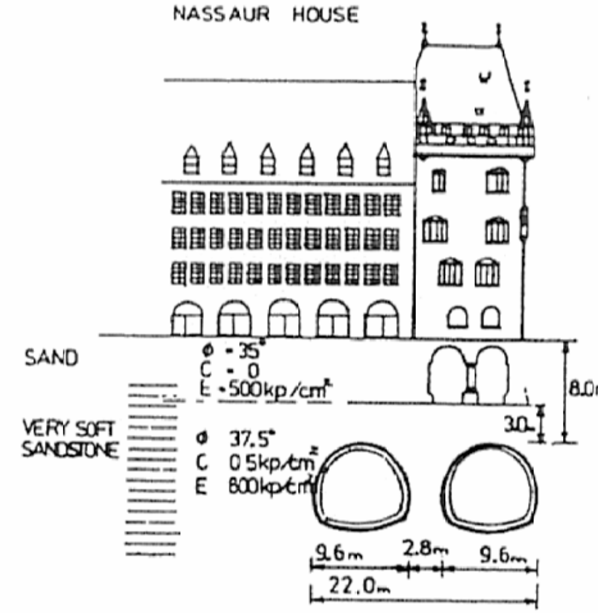

Figure 11.7 Subway station in Nürnberg, Germany formed using the New Austrian Tunnelling Method.[28]

a subway station with cross-passages adjacent to delicate and historic buildings (Figure 11.7).

By this time, the use of dowels and bolts to stabilize rock slopes was also well established. For example, Bonazzi and Colombet[29] described the stabilization, by 'ancrages passifs,' of a rock slope in schists at the Notre Dame de Commier Dam, France in 1961 as being one of the first major rock slopes stabilized in that way. They also reviewed applications in other civil engineering projects such as the 45 m high slope on the A9 Autoroute, in France.

The French contractor Bouygues had gained experience with the New Austrian Tunnelling Method and saw that similar techniques could be applied for the temporary support of soft rock and soil slopes. In 1972, in joint venture with the specialist contractor Soletanche, they started work on a 70° cut slope in heavily cemented Fountainbleau Sand for a railway widening scheme near Versailles. A total of 12 000 m² of face was stabilized by over 25 000 nails grouted into predrilled holes up to 6 m long. This appears to be the first published case history of a true soil nailing project, although it would seem that similar techniques were by then being used for tunnel portal support, and deep excavation stability in Western Canada. Indeed, Shen *et al.*[30] refer to the execution by soil nailing of 'several hundred thousand square feet of excavation, to depths of up to 60 ft,' in a variety of ground conditions in that region prior to 1976. It is likely that nailing also began to be used in California and West Germany about the same time for similar applications.

However, engineers in each of these three distinct areas appear to have proceeded independently—especially with respect to design methodology—until a Paris conference on soil reinforcement in 1979 provided an international forum for the exchange of information. Despite—or even because of—these often heated debates, soil nailing has continued to expand and is still one of the fastest growing specialty geotechnical techniques, described recently[31] as a "solution looking for problems."

First applications continue to be reported from different countries, as already noted, while the scale of the market in older established areas was eloquently described by Condon[32]: over 100 000 square meters of nailed excavations in over 200 different sites—some seismic and as deep as 20 m—over the last 10 years or so in Southern California alone. Nicholson[33] described the growth of the technique from 1982 in the Eastern states, where the current total of completed projects is now approaching Californian levels. Worldwide, about 15–20% of all applications are for permanent structures.

Fundamental research programs have been conducted since the late 1970s with federal and private funds in France,[34] Germany[5] and the United States,[35,36] designed to improve understanding of nail and structure performance, and to evolve design manuals. Demonstration projects have also been sponsored by interested parties, including one of the few contracts so far completed in Britain.[1]

11.4 Applications

Soil nailing has been used successfully in temporary and permanent application, 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.

11.4.1 New construction

11.4.1.1 Retaining walls. For excavations associated with foundations of buildings, underground car parks, and cut and cover constructions for transportation systems (Figure 11.8).[5,35,37,38]

11.4.1.2 Slope stabilization. For abutment and embankment cuts required for new or widened railway lines or roads (Figure 11.9).[33,39,40]

11.4.1.3 Stabilizing tunnel portals. To provide excavation stability to tunnel portals and adjacent slopes (Figure 11.10).

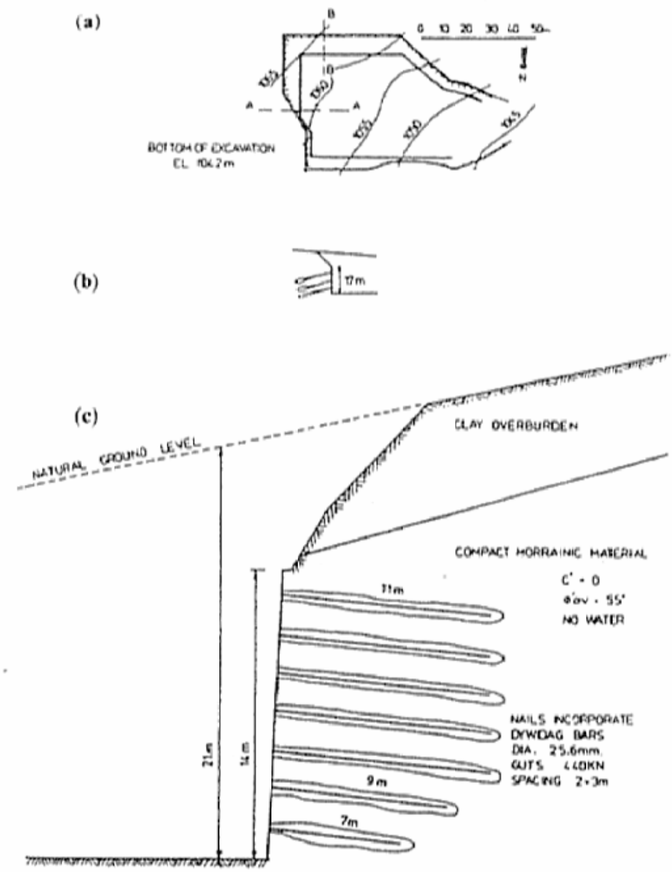

Figure 11.8 Retaining wall for an underground car park at La Clusaz, France: (*a*) plan; (*b*) cross-section A (anchored Berlin wall); (*c*) cross-section B. After Guilloux *et al.*[37]

11.4.2 Remedial works

11.4.2.1 Repair of reinforced earth walls. To replace the effect of the reinforcing strips or fasteners damaged by overloading or corrosion (Figure 11.11).[42, 43]

11.4.2.2 Repair of masonry gravity retaining walls. After or just before failure caused by long-term decay of wall, or movements behind (Figure 11.12).

11.4.2.3 Stabilization of failed soil slopes. After collapse of slope due to failure or inadequacy of pre-existing support methods, or catastrophic movements due to changed hydrogeological factors (Figure 11.13).

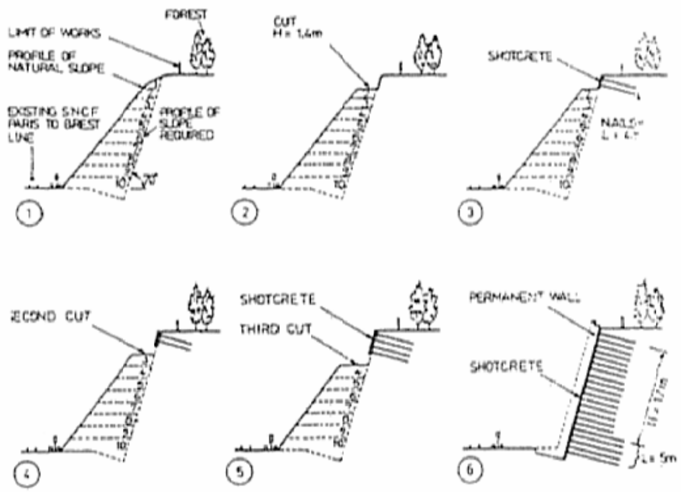

Figure 11.9 Slope stabilization construction sequence at Versailles, France.[39] Fontainbleau Sand (cemented). $\phi = 33$–$40°$; $c' = 20\,\text{kN/m}^2$. Soil nails: 2×10 mm diameter bars grouted into 100 mm diameter holes, spacing 0.70×0.70 m (two per square meter of face).

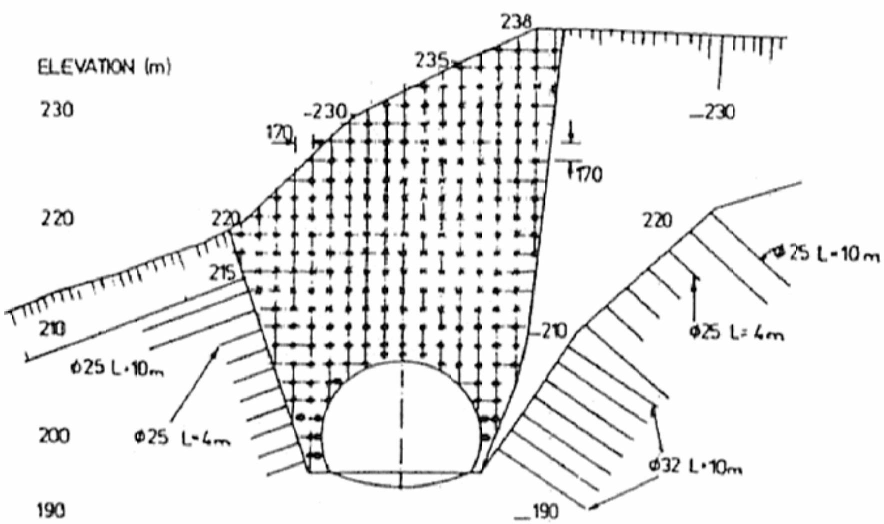

Figure 11.10 Stabilization of tunnel portal and adjacent cut slopes.[41] Nails: (●) $L = 10$ m, $\phi = 25°$; (×) $L = 4$ m, $\phi = 25°$; (○) $L = 10$ m, $\phi = 32°$.

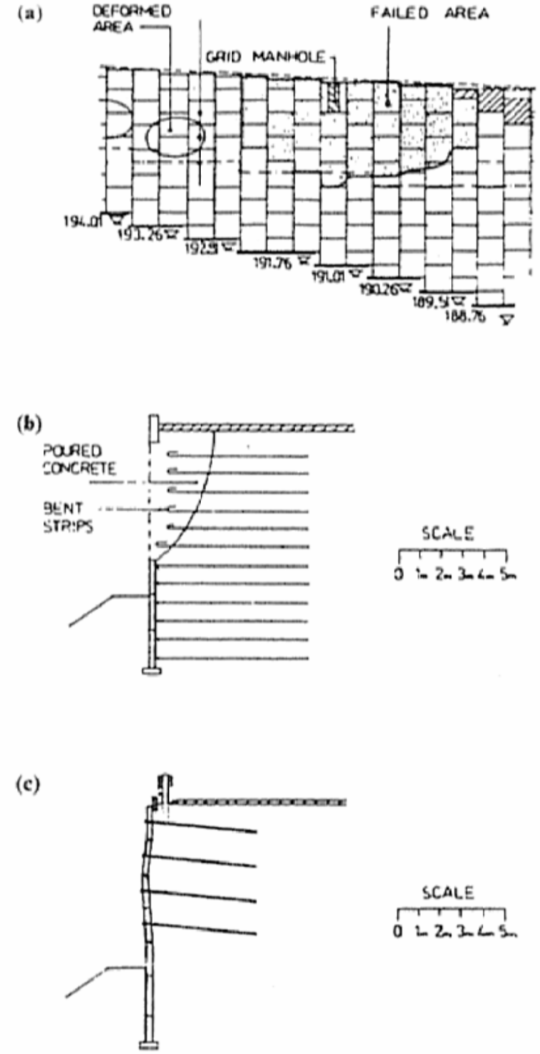

Figure 11.11 Repair of reinforced earth wall at Fréjus, France. After Long *et al.*[43] (*a*) Front face view of wall; (*b*) rebuilding of facing of collapsed area; (*c*) nailing of repaired zone.

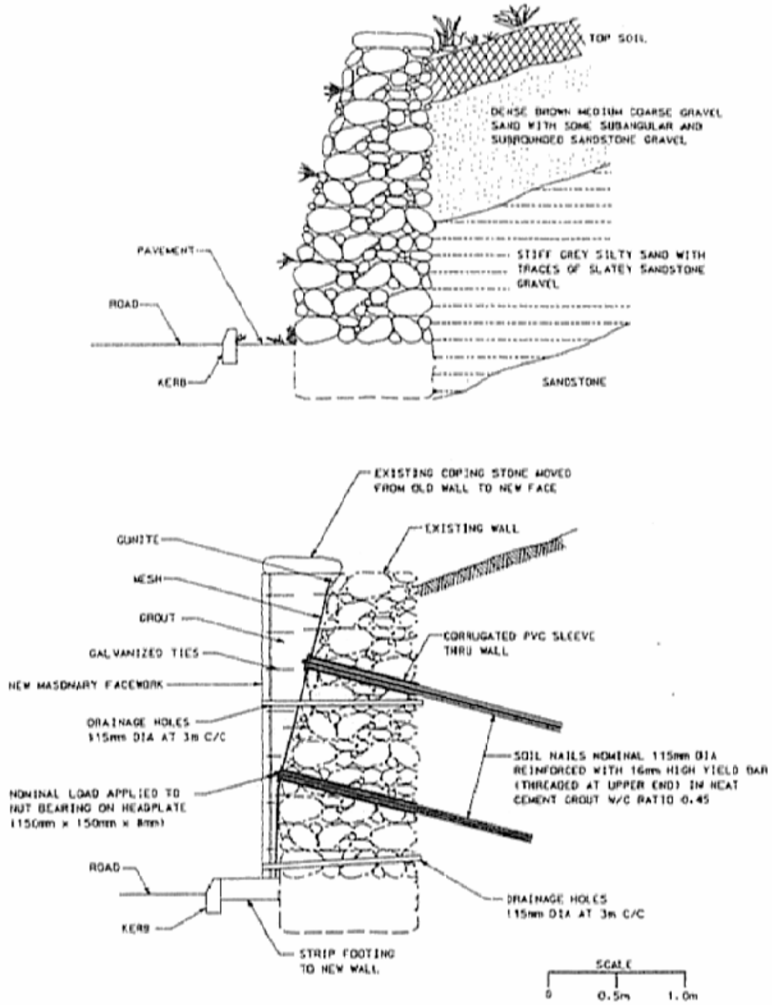

Figure 11.12 Repair of masonry gravity retaining wall at Bradford, UK.[1]

11.4.4.4 Repair of anchored walls. After failure of the prestressed rock anchorages by structural overloading or by corrosion of tendon (Figure 11.14).

11.5 Construction

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

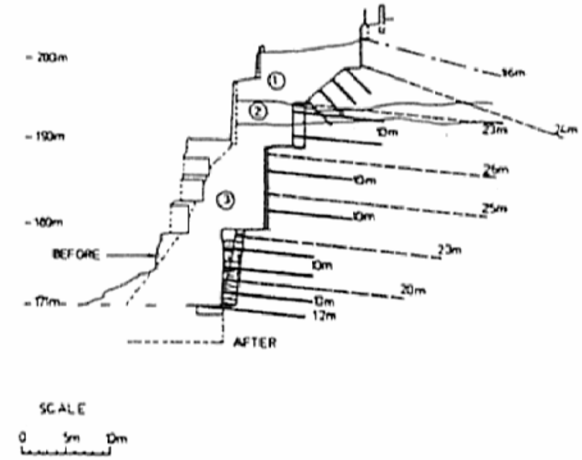

Figure 11.13 Repair of a failed soil slope at Herbouville, Lyons, France. After Gausset.[44] ① = Sandy silts, $C_u = 0$; ② = silty sands (molasse), $\phi = 25°$; ③ = sandstone, $c = 0.05$ MPa, $\phi = 35°$. (—) nail; (———) prestressed anchors (----) passive anchors.

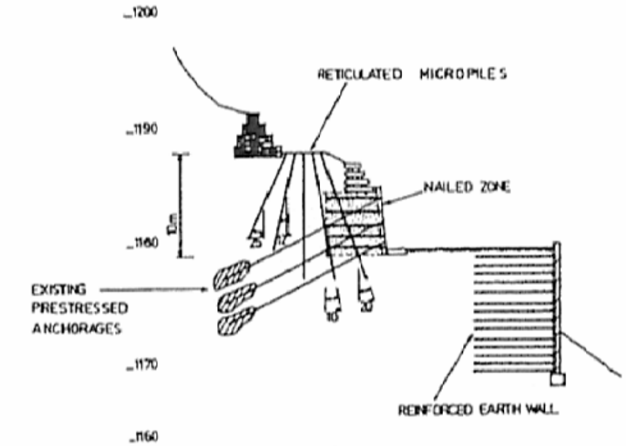

Figure 11.14 Repair of an anchored wall at Fréjus, France. After Corte and Garnier.[45]

11.5.1 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 minimized, the cut depth may be reduced to the smallest value consistent with site practicalities and commercial considerations. Cut depths of more than

2.0 m or less than 0.5 m are rare in granular soils. Greater, single cut depths have been used in heavily overconsolidated clays.

A level working bench at least 5 m wide should be provided for the nailing equipment. Usually the length of a single cut is dictated by the area of face that can be stabilized in the course of a working shift. Where deformations must be minimized, and especially in wet or very sensitive soils, the nailing may be executed in alternative primary and secondary panels, typically 10 m long, but occasionally as short as 2 m.

The excavation equipment must minimize 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 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. Grout columns may also be necessary to provide early support where the soil is cohesive and will not 'take' shotcrete.[46]

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 shotcrete cover before drilling the nail holes. The final face thickness varies from 50–150 mm for temporary applications, to 150–250 mm for permanent projects. The face may be built up in one, two or more layers, depending on the nail type, the construction and stressing sequence, and the longevity of the structure. Short bars may be driven into the face before spraying to serve as a depth gauge for the sprayed concrete, and a final screeding can be achieved using a piano wire as a guide. Architectural finishes may be applied with a final layer of sprayed concrete—say 50 mm thick—to blend color, 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 10–15 mm are usually specified, and admixtures are often incorporated to accelerate set in very wet conditions or, less commonly, to reduce early creep of the hardened concrete. Minimum cement contents of 300 kg/m^3 are typical, and 400 kg/m^3 is common. Control 'panels' or boxes are recommended for on-site quality assurance, at frequencies of about one per 100 m^2. Accelerated shotcrete should give an unconfined compressive strength of around 5 N/mm^2 in 8 h, whilst it is best to let it cure for 24 h prior to further works. The proper curing of the sprayed concrete face is important if surface cracking is to be avoided. Steel or plastic fibers can also be added to the shotcrete to enhance performance during and after spraying.

Spraying is often discontinued about 300 mm above the bottom of the cut. This facilitates both 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°. Careful screeding can eliminate, visually, this interface.

Final pointers on good construction practice were provided by Condon.[32] Special care is warranted with the top row of nails—'the key to a good wall'—while the soil above this row should be sloped back at about 2 : 1 wherever possible to reduce nailed area, and avoid near-surface utilities.

11.5.2 Drainage

An early aid is to excavate a drainage ditch parallel to, and 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, three types of drainage can be applied to the retained soil mass:

1. Shallow drains: pipes 300–400 mm long, to release water immediately behind the facing. These drains are usually about 100 mm in diameter. Their spacing depends on the groundwater conditions and the likelihood of frost damage.
2. Deep drains: slotted tubes, usually longer than the nails, about 50 mm 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 3 m^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 and 5 m. 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 shallow drains.

11.5.3 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.

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—suitable for strong, cohesive soils, rock or concrete) are:

1. Duplex drilling. This rotary or rotary percussive method involves the simultaneous advancement of a temporary outer casing and an inner drill

rod.[47] Water or air flush is usually employed, although care is needed with air flush in urban environments.

2. 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, although this system does mean that drill hole diameters are considerably larger than average. This method is particularly common in California.

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

Most recently Louis[48, 49] has reported patented systems of nail installation for which very high rates of production are claimed. In the 'jet bolting' system (Figure 11.15) very high pressures—over 200 bar—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, on 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. To the author's knowledge, jet bolting systems have not yet had significant commercial application outside southern France.

In general, borehole diameters range from 76 to 150 mm for drilled and grouted nails, although the use of the hollow stem auger usually raises the upper limit to 200 mm. Such diameters usually permit a grout annulus of at least 20 mm thickness around the reinforcement, so providing a degree of

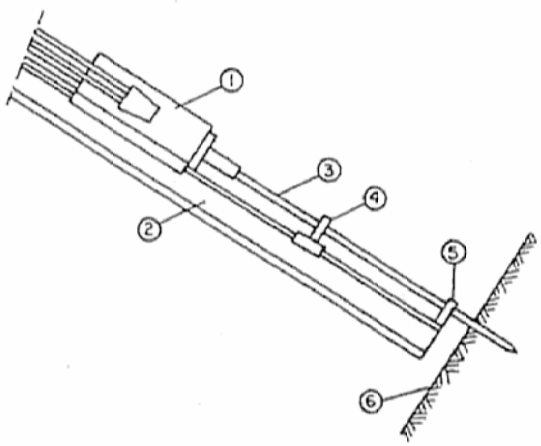

Figure 11.15 The 'jet bolting' technique for soil installation—combines vibropercussion and high grout pressure at the nail tip.[49] ① = Vibropercussion hammer; ② = sliding support; ③ = reinforcement to be inserted; ④ = sliding guide; ⑤ = fixed guide; ⑥ = soil to be treated.

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 gound anchorages, and this allows higher production rates. Holes inclined downwards (even as little as 10–15°) are easier to grout effectively than those that 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 5 bar) 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 do not justify the higher grouting costs for most soil nailing applications. The reinforcement should be placed and the grouting completed with the minimum delay after drilling.

11.5.4 Reinforcement and corrosion protection

Although polymer-based reinforcements such as plastic rods or fiberglass, becoming common in underground works, are being promoted for soil nailing, it would seem that steel bars are still used universally. High yield bars of 25–32 mm diameter are the most typical choice.

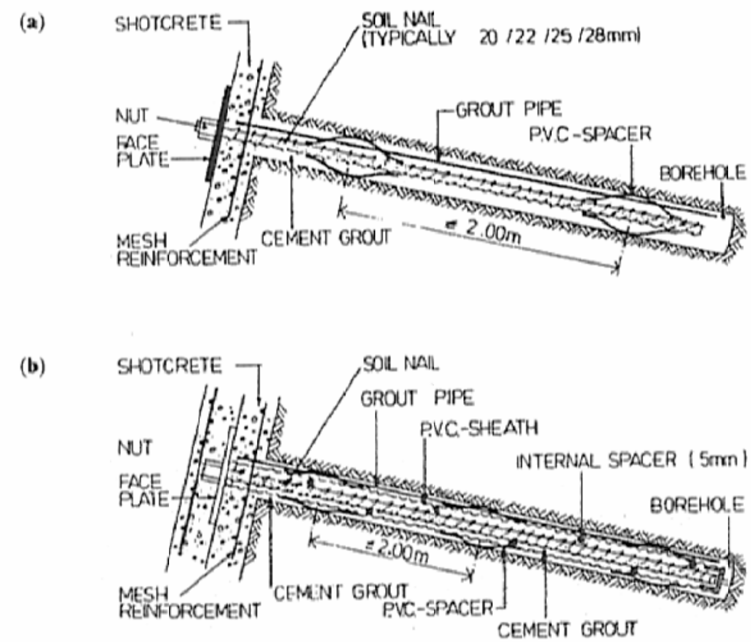

Figure 11.16 Examples of good practice for drilled and grouted nail installation: (*a*) for temporary applications; and (*b*) for permanent applications. (Based on (formerly) West German experience, and reproduced with permission of Bauer, AG.)

For temporary applications in standard environments, corrosion protection is usually provided only by the grout, but occasionally also with an epoxy coating to the steel surface (Figure 11.16(a)). For permanent works, the degree of protection may be increased by providing an outer sheath of plastic material, ensuring an inner grout annulus of at least 5 mm thickness (Figure 11.16(b)). 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).[7] In all cases, centralizers are placed at regular intervals (say 2 m) along the reinforcement to encourage 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, in codes for reinforced earth the galvanized steel strips for permanent installations are allowed to remain in direct contact with the soil.[50,51]

One of the most recent studies on corrosion in reinforced earth[52] 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 (e.g. Figure 11.16(b)). A thorough summary of corrosion mechanisms and protection has been provided by the FIP state-of-the-art review *Corrosion and Corrosion Protection of Prestressed Ground Anchorages*, prepared by a working group under the chairmanship of Professor G.S. Littlejohn[53].

The way in which soil nails work—virtually their whole length is bonded to the soil and available for load transfer—means that it is unnecessary and impractical to apply significant degrees of post-tension after installation. However, a load of about 5–10% of the working load is usually 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 immediate response to subsequent soil deformation . Since the 'lock off' loads are relatively low, the steel bearing plates are quite light (150 × 150 × 10 mm or 200 × 200 × 10 mm), and stiff wales are generally not required—although some US contractors provide 4 × 12.7 mm bar wales. The nominal post-tensioning is normally applied during or just after the installation of nails in the cut immediately below.

11.5.5 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 is 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 enhance noise absorp-on. Such panels may be placed directly in contact with the slope face during construction and the nails placed at the centers or corners of each panel. Alternatively, the excavation may be completed with a normal sprayed concrete facing and then covered later by precast panels. Drainage arrangements are often attached directly to the back of facing panels, or the gap between shotcrete and panel filled with suitable filter material.

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 that 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 face.

11.5.6 Instrumentation and monitoring

In contrast to ground anchorages, it is not routine or necessary to test each individual nail. This reflects the fact that it is the overall performance of the soil-nailed mass which is paramount. Selected nails should, however, be subject to pull-out tests during each level of excavation, to verify the design assumptions on bond capacity. Louis[49] recommends that, for good practice, four or five 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, while Condon[32] tests 5% to some test load.

By strain gauging individual nails at regular intervals along the bar, the development and distribution of the nail forces may be measured, and this can provide vital feedback to designers, assuming the data are consistent and truly reflective of in-situ conditions. Load cells at the nail head also provide useful data, particularly where near-surface effects, such as freezing, may be significant. Pressure cells placed under the shotcrete cladding can help in determining soil pressures on the face of the excavation.

Arguably the most useful 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 continuous record of performance, thereby allowing modification of the construction details in response to changed conditions—most importantly if poorer soils are encountered. Readings maintained after construction can be equally informative in gauging time-dependent movements.

11.6 Design

11.6.1 Background

The most controversial aspect of soil nail technology is undoubtedly their design and, in particular, the choice of method to compute stability and the role of shear in nail capacity. Recent publications by the respective antagonists have reached a level of stridency not commonly observed in the technical press.[54,55] As in all such matters there is probably no one 'best' method, as illustrated in several papers in a Technical Session at the recent ASCE Conference at Cornell University.[56]

To put the issue into perspective, it is instructive to review the published data to fundamentally determine the major factors influencing the behavior of soil-nailed structures. It would appear that the key design-related elements are:

1. the ultimate capacities of the individual nails;
2. the forces mobilized in the nails under service
3. the various equilibrium methods used to compute stability.

A fourth design consideration, namely the deformation performance of the walls, is discussed in section 11.7.

11.6.2 Ultimate nail capacities

A major conclusion from the wealth of studies, tests and observations is that the ultimate pull-out capacities of nails are strongly influenced by their method of installation. It also seems likely that the noted variations in capacity are due in large part to variations in the normal pressure (σ_o) between the nail and the surrounding ground, as well as the geotechnical categorization of the soil itself (Figure 11.17), and the depth of cover. Another possible source of difference in capacity is the angle of interfacial friction between the nails and the surrounding ground (δ), which is larger for grouted nails than for driven nails.

Schlosser[6] indicated that, in the case of driven nails, the stress (σ_o) should be equal to the overburden pressure. In the case of grouted nails (presumably gravity grouted) he indicated that the normal stress may be very low and approximately constant with depth.

Nicholson[58] suggested that, for grouted nails, the pull-out capacities can be estimated using the same techniques used for grouted anchorages. This suggests that the value of (σ_o) would be assumed equal to the grout pressure in the case of pressure grouted nails.

11.6.3 Mobilized nail forces

Juran and Elias[59] summarized nail forces measured by others in walls constructed in silty sand, poorly graded sands, and clayey sand and residual soils. They converted the measured nail forces to apparent pressure diagrams—as Terzaghi and Peck[60] had done for measured strut forces in braced excavations. The distributions of apparent pressure (shown in Figure 11.18) are very similar to those developed by Terzaghi and Peck, increasing with depth in the upper half of the wall, reaching a maximum near mid-height, and decreasing again below mid-height. It seems likely that the similarity in apparent pressures results from the fact that nailed walls—like conventional braced excava-

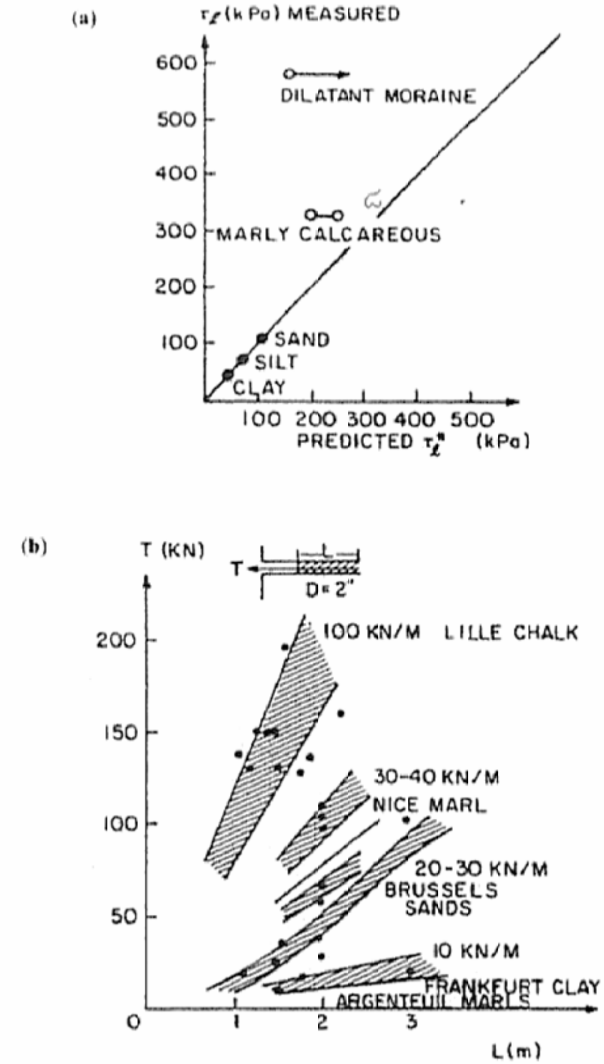

Figure 11.17 (*a*) Comparison of measured soil bar lateral friction and design guidelines for friction piles using pressuremeter test results;[57] (*b*) variation of pull-out resistance of reinforcing elements with depth of embedment (cement or resin grout) for different soils (C. Louis, unpublished results). (●) driven bars; (○) grouted bars in borehole.

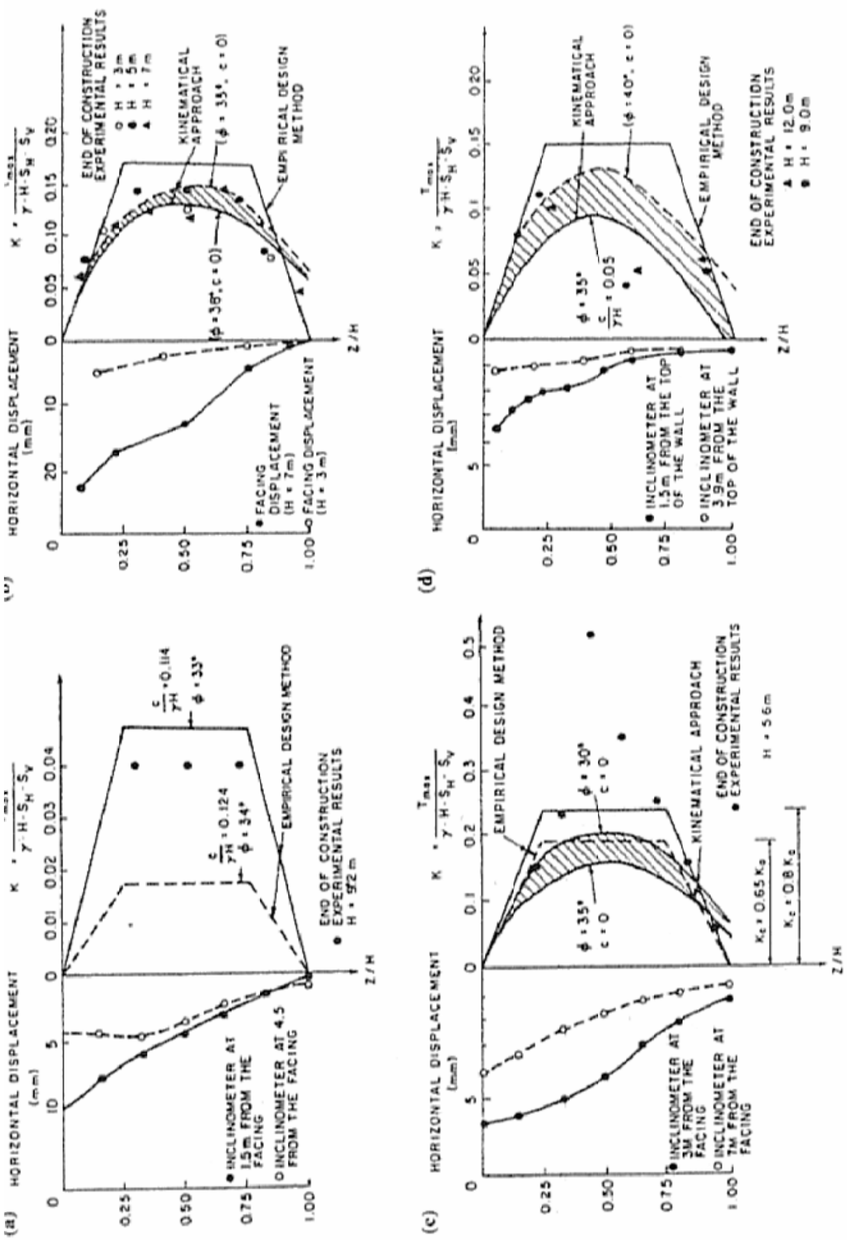

Figure 11.18 Apparent lateral earth pressure for soil-nailed walls—experimental data and theoretical predictions of tension forces. After Juran and Elias.[59] (*a*) Davis wall; (*b*) full-scale experiment CEBTP; (*c*) Parisian wall; (*d*) Cumberland Gap wall.

tions—are constructed from the top down, with the nails being installed at the top first. The stress distributions are therefore likewise different from those for reinforced earth.

Juran and Elias[59] also found that nail forces increased by 15 to 30% with time after construction in a wall in poorly graded sand. The nail forces measured in the Cumberland Gap wall in residual soils increased by 50 to 70% after construction. During the first winter the nail forces in the Cumberland Gap wall reached values as large as 100% more than the values at the end of construction, and did not decrease afterward. These findings indicate that some allowance should be made for increase in nail forces after the end of construction (assuming these data were not overly influenced by ice build-up at the face).

Experimental studies have shown that the displacement required to mobilize the ultimate pull-out capacities of soil nails are small, typically of the order of 4 to 7 mm.[61] Although the displacements required to mobilize tension in the nails are small, considerably larger deformations are required to mobilize shear forces in the nails.[62]

Shewbridge and Sitar[63] found that displacements of 20 to 40 mm were required to mobilize shear capacities in model tests, and it would be expected that the displacements required to mobilize shear resistance would be considerably larger than 40 mm for full-scale nails in the field.

Since larger displacements are required to mobilize shear resistance, it appears that the principal mode of behavior of soil nails in practice is tensile reinforcement and that the stabilization they impart results mostly from tension forces[6,54,56,64]. It seems reasonable, therefore, to ignore shear forces in soil nails as a stabilizing mechanism and to rely only on their tensile action in design analyses although it must be noted that this conclusion is strongly disputed in certain quarters.

11.6.4 Stability computation procedures

Review of the literature shows that most analyses of soil-nailed walls have been performed using computational procedures based on static equilibrium approaches, similar to those used for conventional slope stability analyses. At least five distinctly different equilibrium stability analysis procedures have been developed for nailed walls. The features of each of these can be summarized as follows.

Gassler and Gudehus.[40] This procedure has been called a 'kinematical' procedure. It is based on a two-part wedge mechanism with a bilinear shear surface. Nail forces are assumed to be known. The equations of horizontal and vertical force equilibrium are used to compute the magnitude of the forces in the soil on the bases of the wedges, and between the two wedges. The factor of safety is then defined as a *ratio of work* done by external and internal forces rather than as a ratio of forces or shear strengths, as is done in other procedures. A simple bilinear shear surface and only two wedges ('slices') are considered. Accordingly, the procedure is limited with regard to the types of soil conditions that can be considered, since the base of each slice must be entirely within one type of soil.

Beech and Juran.[65] This procedure assumes a curved shear surface that appears to be a logarithmic spiral. The shear surface is assumed to be perpendicular to a line extending back into the slope from the crest of the slope, parallel to the nails, i.e. the shear surface intersects the uppermost nail at approximately 90°. Kotter's equation is integrated to obtain the distribution of normal stresses along the shear surface, and the shear strength is assumed to be fully mobilized along the shear surface. Thus, the shear stresses can be expressed in terms of the normal stress and shear strength parameters (c and ϕ). Forces are resolved in the horizontal direction for segments of the shear surface—corresponding to the bases of inclined 'slices'—to compute both shear and axial forces in the nails where they cross the assumed shear surface. Internal forces are ignored when forces are resolved in the horizontal direction. Unlike other equilibrium analysis procedures, Beech and Juran's procedure is intended for computation of forces in individual nails, rather than as a procedure for computing overall stability and safety factors. It appears that the nail forces calculated by this procedure are used subsequently to compute a factor of safety against failure of the nails using an estimated ultimate capacity for the nails. Juran and Elias[59] have referred to Beech and Juran's procedure as also being a 'kinematical' method.

Schlosser.[6,66] Schlosser's procedure is based on what appears to be conventional limit equilibrium approaches employing circular shear surfaces. Reference is made to Fellenius's (Ordinary Method of Slices) procedure and Bishop's procedure; however, the specific limit equilibrium procedure employed by Schlosser is not specified. Considerable attention is paid to the nail forces, including particularly the shear forces in the nails. Four failure criteria are considered in arriving at suitable nail forces, and these criteria consider:

1. Structural failure of the nailed inclusion (nail and grout) under combined shear and tension at the point where the nail crosses the shear surface.
2. Structural failure of the nail by bending at the point of maximum moment, which occurs some distance away from the shear surface.
3. Pull-out failure of the nails.
4. Failure due to 'flowing' of the soil around the nail, much like what occurs around a laterally loaded pile as it undergoes excessive deflection.

The lowest of the nail forces found by considering these failure criteria is used in the stability calculations. Separate factors of safety may be applied to each of the failure criteria to compute the nail forces. The governing nail forces may vary along any given nail and from nail to nail. Failure due to shear in the soil itself is accounted for in the normal way by the factor of safety with respect to shear strength, which is computed from the limit equilibrium analyses. A computer program called TALREN performs the computations. The procedure is applicable to non-homogeneous soil conditions, provided that the critical shear surface can be approximated accurately by a circle.

Bangratz and Gigan.[67] Bangratz and Gigan described a procedure for computing the factor of safety based on the Ordinary Method of Slices. Accordingly, the procedure employs a circular shear surface. Nail forces are estimated and are included in the equilibrium equations. The component of the nail forces perpendicular to the base of the slice contributes to the normal force on the base of the slice, and thus the forces in the nails are considered to contribute to the available shear strength. The component of the nail forces acting tangential to the circular shear surface contributes to the resisting moment. The factor of safety is defined with respect to shear strength in the conventional manner used in limit equilibrium slope stability analyses.

Shen et al.[30] This method, sometimes referred to as the 'Davis' method, is based on the assumption that the shear surface is parabolic. However, only two slices are used, and failure mechanism is therefore actually a two-part wedge with a bilinear shear surface. The requirements of equilibrium in the horizontal and vertical directions are satisfied by the equations used to compute the factor of safety. Nail forces are estimated independently of the limit equilibrium computations. Only the axial components of the nail forces are considered and included in the equilibrium equations. The factor of safety is defined with respect to the shear strength of the soil. Unlike most limit equilibrium procedures, this procedure may actually correspond to different values of the factor of safety for each slice. However, only an 'average' factor of safety is actually computed. If a factor of safety is applied to the nail forces, it must be included in the values of nail force used in the analyses. Since this procedure employs only two slices, it is limited with regard to the types of soil conditions that can be considered, as the base of each slice must be entirely within one type of soil. Some engineers (e.g. reference 68) state that this method 'overpredicts the width of the active zone.'

In summary, different methods exist because: (i) different shapes of shear surface are assumed; (ii) different equilibrium conditions are satisfied; and (iii) different definitions of the factor of safety are used. None of these procedures of analysis satisfies all of the conditions of static equilibrium, and none is capable of considering non-circular shear surfaces, as might be appropriate if a layer of distinctly weaker soil exists in the cross-section. In addition, three of the procedures (Gassler and Gudehus, Beech and Juran, Shen *et al.*) use so few slices that, for practical purposes, they are restricted to homogeneous soil conditions.

There does not appear to be any good reason why procedures that: (i) satisfy complete static equilibrium; (ii) are applicable to non-homogenous soils; and (iii) can be used to analyze any shape of shear surface, should not be used for analysis of soil-nailed walls. In fact, considerable advantages would result from their use. Such procedures can be used to consider stability along shear surfaces passing through the nails (internal stability). They can also be used to examine the possibility of bearing and sliding failures of the nail-stabilized soil mass (external stability). Efforts to develop these procedures are currently underway, albeit often on a proprietary basis, in different academic and commercial centers; a detailed example is provided by Bridle.[69]

11.6.5 Development of design methodology

The studies described in the previous section can provide a basis for understanding the behavior of soil-nailed walls, and for deciding what factors should be considered in their design. A reliable design methodology can, simplistically, follow the following steps.

1. Estimate the forces that will be mobilized in the nails. The nail forces can be estimated using apparent pressure diagrams of the type that have been used for many years for estimating loads on excavation bracing systems. Apparent pressure diagrams suitable for nailed walls will need to be developed using the results of experiments and field measurements.
2. Calculate the minimum nail lengths required to develop these forces, with suitable factors of safety against pull-out. The minimum length is equal to the required nail force divided by the allowable load per unit length that can be applied to the nail. Procedures for estimating these allowable loads need to be developed and checked against the available information regarding results of pull-out tests on nails.
3. Perform stability analyses using a method that satisfies all conditions of equilibrium, and that can be used to analyze slip surfaces of any shape. Analyses should be performed for all stages during construction, as well as at the end of construction. The wall should be stable for the conditions existing prior to installation of each row of nails, as well as after. The soil strength parameters used in the analyses should be either undrained or drained, consistent with the permeabilities of the soils and the time available for drainage. The long-term condition should be analyzed using drained strength parameters and the most adverse groundwater conditions that could develop during the life of the wall.
4. If necessary, increase the lengths of the nails, in order to increase the value of the factor of safety to an acceptable value.
5. Estimate the deformations that will develop during and following construction. These deformations can best be estimated based on the results of previous experimental and analytical studies described in the literature. If the estimated displacements are larger than those that can be tolerated, the design can be modified to reduce their magnitudes.

11.7 Data from published case histories

Published information on case histories, containing construction details for soil-nailed structures in a variety of soil conditions, was tabulated by Bruce and Jewell.[1]

11.7.1 Derived parameters

Four derived parameters or ratios were also calculated for each project:

1. The overall geometry of the structure.

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

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

$$\text{Bond ratio} = \frac{\text{Hole diameter} \times \text{Nail length}}{\text{Nail spacing}} = \frac{(d_{\text{hole}})L}{\text{spacing}}$$

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:

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

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

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

Where the nail is not a circular bar, equivalent values for the nail diameter and the hole diameter were 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.

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

11.7.2 Observations on case histories

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

11.7.2.1 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 11.1. Overall, for projects in granular soils, the driven nails are slightly shorter than those that 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,

Table 11.1 Comparison of drilled and grouted, and driven nails for steep slope case histories in granular soil.[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	0.001–0.003	No data

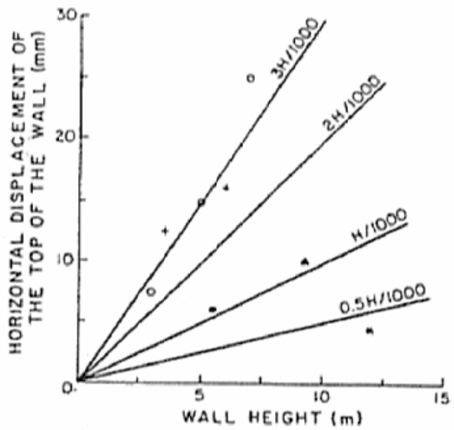

Figure 11.19 Horizontal displacement of nailed walls.[7] (+) medium sand, driven nails (Gassler *et al.*, 1981); (▲) silty sand (SM), grouted nails (Shen *et al.*, 1981); (●) fine sand (SP) to clayey sand (SC), driven nails (Cartier and Gigan, 1983); (•) residual clayey silt weathered shale, sandstone, grouted nails (Juran and Elias, 1986); (○) Fontainbleau Sand (SP), grouted nails (Plumelle, 1986).

Table 11.2 Comparison for drilled and grouted nails for steep slope case histories in granular soils and Moraine or Marls.[1]

	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

must be caused by providing more surface area for bonding with the driven nails.

The performance ratios for drilled and grouted nails show consistently an outward movement of up to 0.3% of the excavation depth (Figure 11.19). Similar excellent performance would be expected for the driven nails, although no measurements were reported on the commercial projects.

11.7.2.2 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 11.2. 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 stabilize 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.

11.7.2.3 Comments on reported failures. The failure at Les Eparris (Table 11.3) 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 was increased by a factor of three but the strength ratio is little changed.

Much less information is available for the Gard 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.

Participants are normally reticent to discuss failures, but various personal communications confirm that several unpublished failures have occurred throughout the world. Lack of bond, leading to nail pull-out, is the most common cause, but it is clear that other construction-related factors have often contributed. These include—individually or in combinations:

(i) inefficient and late excavation face stability prior to nailing, including the exposure of too-large face areas in one pass;
(ii) use of too-high pressure air during drilling, causing fracturing of the soil mass into vertical slices;

Table 11.3 Case histories of failures of drilled and grouted soil nail systems.[1]

Project name Description and scale	Date	Main references	Ground conditions: Cohesion (kN/m²)	Ground conditions: Friction (degrees)	Ground conditions: Unit weight (kN/m³)	Slope angle (degrees)	Height (m)	Nail length (m)	Nail diameter (mm)	Nail hole diameter (mm)	Nail spacing: Horizontal (m)	Nail spacing: Vertical (m)	Nail spacing: Area (1 per m²)	Nail angle (Horiz.) degs
Les Epparis, France 70 degrees clay cutting 4.2 m high in ground sloping at 15 degrees	1981	6, 70	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
Repaired to a 60 degree slope 5.2 m high	1982	70, 71	0	28	20	60	5.2	10.0	26.0	105	2.00	1.73	3.46	30
Paris Gard du Nord, France	1979	70	Fill overlying 0	30	–									
Excavation of 10 m vertical in Marls			Heterogeneous Marls 50	20		75 overall	10.0 overall	6.5	32.0	100 (est.)	2.50	1.60	4.00	20 overall
Repaired as a vertical wall 8.5 m high	1979	70	Assumed strength 0	25	–	90	8.5	10.0	32.0	100	1.50	1.25	1.88	15

(iii) lines of pre-installed dewatering or instrumentation holes, parallel to, and at some distance back from the face. These act as a vertical 'presplit' line in the soil mass; and

(iv) insufficient attention paid to the shape of the wall in plan (especially where it includes a corner or bend), or the profile of the slope in section (major backslope giving additional surcharge).

11.8 Two recent case histories

The following two North American case histories have been selected to illustrate many of the points raised in the chapter to date. The first—from Edmonton, Canada—focuses on nail application, construction and performance. The second—from Seattle, Washington—provides one view of the design process, as supported by on-site measurements of nail strains.

11.8.1 Excavation for tunnel portal, Edmonton, Alberta, Canada[72]

11.8.1.1 Background. The city of Edmonton recently commissioned Phase II of the South Light Rail Tansit (SLRT) Extension. This involved the construction of the South Tunnel and Portal, from the south bank of the North Saskatchewan River to the University of Alberta campus (Figure 11.20). As shown in Figure 11.21, the initial 80 m of the tunnel alignment was marked by unfavorable ground conditions—created by slump debris from an ancient landslide comprising saturated sand deposits. For economic and technical reasons, this initial portion was designed as a cut-and-cover section.

The original concept featured soldier piles, prestressed ground anchorages, and struts to secure the excavation (160 m long, 18 m wide to a maximum depth of 19 m), in concert with peripheral dewatering wells. The successful contractor proposed an alternate shoring system featuring soil nails—known locally as the 'Ground Control Method.' This was approved, but with the

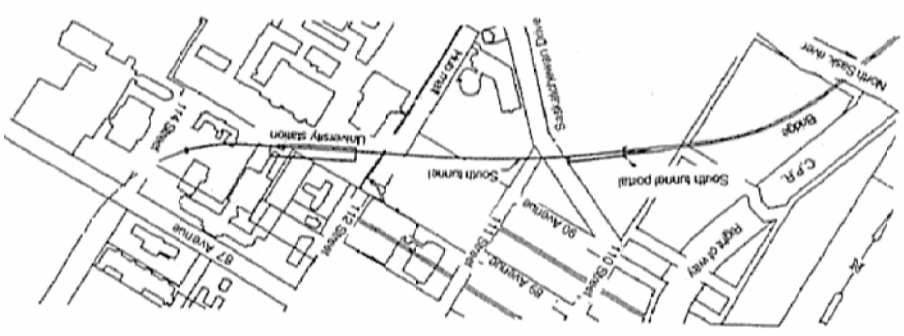

Figure 11.20 Excavation for tunnel portal at Edmonton, Canada—general site plan.[72]

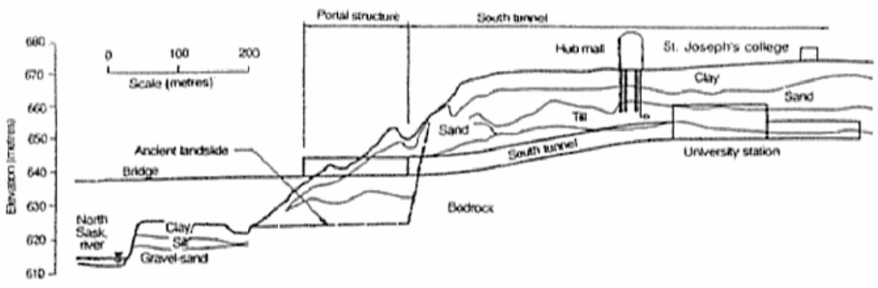

Figure 11.21 Excavation for tunnel portal at Edmonton, Canada—simplified geological cross-section.[72]

caveat that the whole system was appropriately monitored to confirm acceptable performance.

11.8.1.2 Site and geological conditions. The portal was located at about mid-height in the valley slope, immediately below an intermediate terrace at an elevation of 650 m. From this terrace, the ground surface rose at a slope of about 20° to the prairie level at an elevation of 670 m.

The detailed stratigraphy is shown in Figure 11.22. The site investigation confirmed a complex and variable stratigraphy—a direct result of the landslide processes. The general stratigraphy consisted of clay over sand over Cretaceous bedrock. The superficial lacustrine clay was stiff and highly plastic. The sand was fine-grained but variable in silt/clay content (7–46%, average 23%). It was generally medium-dense with SPT values ranging from 5 to 30 (per 300 mm penetration). Additionally, layers of clay and clay till were found in the sand. The bedrock comprised claystones, siltstones and sandstones, with occasional bentonitic and coal seams.

Piezometers confirmed the water level as varying from an elevation of 646.5 m (north) to 654 m (south), about 12-20 m above the base elevation, while pump tests indicated a sand permeability of 8×10^{-5} m/s. Dewatering

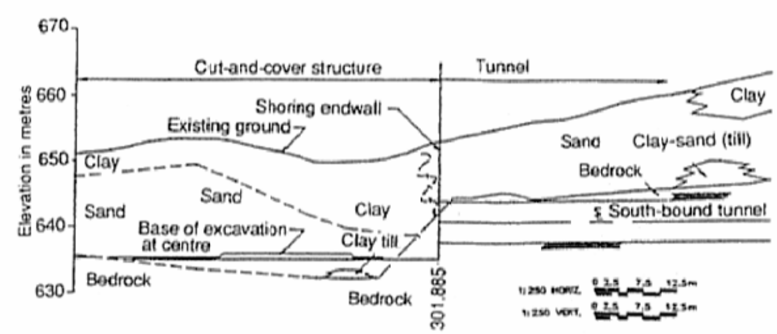

Figure 11.22 Excavation for tunnel portal at Edmonton, Canada—detailed stratigraphy of the portal area.[72]

was therefore a prerequisite for construction, and a contractual obligation was for the piezometric surface to be 2 m below the excavation elevation.

11.8.1.3 Design considerations. The inherent variability of the stratigraphy and groundwater conditions caused serious concerns for the project engineers. In addition, this alternate and locally untried technique could in no way be allowed to impact the overall slope stability. No precedent could be found in the literature for a 19 m high, soil-nailed wall in cohesionless landslide debris. The design was therefore conservatively derived and meticulously reviewed. The following major criteria were employed:

(i) A triangular soil pressure distribution, using $K_a = 0.3$, plus an allowance of 9.6 kPa for live load, was adopted to determine 'the total driving force' for the design of the nails. Practical experience was used to determine the level by level distributions of nail loads.
(ii) The safety factor for stability (both overturning and shear movement) varied from 6 to 9, considering shearing through the sand only.
(iii) The safety factor for designing the nails 'and the ground confinement' ranged from 2 to 3.

The excavation was broken down into eight different zones, based on typical wall height, nail spacing and stratigraphy. Details for each zone are shown in Table 11.4. An example of the shoring design for Zone No. 2, including shotcrete requirements, nail spacings, loads and lengths, is shown in Figure 11.23. The wall is shown at a slight batter angle in this figure but, in reality, was installed near vertical for the most part.

Weep holes, which comprised slotted 50 mm diameter PVC pipes, were designed through the shotcrete wall, on a regular pattern. These drains were a back-up measure to ensure that no water pressure built up on the wall, were the dewatering pumps to fail for any reason.

11.8.1.4 Construction. Installation of dewatering wells around the peri-

Table 11.4 Details of the shoring zones for the excavation for tunnel portal at Edmonton, Canada.[72]

Zone No.	Description	Length (m)	Average height (m)	Vertical nail spacing (m)	Horizontal nail spacing (m)
1	E and W walls	47.0	15.0	2.0	1.8
2	E and W walls	16.0	14.5	2.0	2.0
3	E and W walls	21.0	11.5	2.0	2.1
4	E and W walls	26.0	7.5	2.1	2.4
5	E and W walls	23.0	6.5	2.1	2.4
6	E and W walls	12.0	5.5	2.1	2.1
7	Parking lot	31.0	6.5	2.1	2.0
8	South wall	28.4	19.0	2.0	1.8

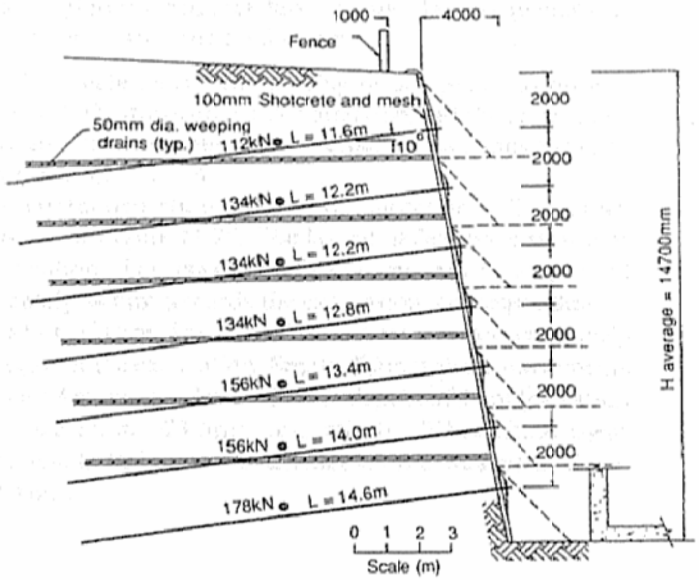

Figure 11.23 Excavation for tunnel portal at Edmonton, Canada—typical shoring design for Zone 2.[72]

phery preceded the excavation and the drilling of nails by one month. The construction procedure then included the following steps:

1. Small panels of soil, approximately 2 m high by 2 to 6 m long, were excavated with intact panels of ground in between. The excavated panels were cut with a sloping face, as illustrated by the dashed line in Figure 11.23.
2. Nail holes, 90 mm in diameter, were drilled with an air-track drill, and Dywidag bars were installed.
3. These bars were tremie-grouted with High Early strength cement at a water/cement ratio of 0.45. No pressure grouting was done.
4. After approximately 24 h of set time, the sloping face of the panel was excavated back to vertical. If located in sand, plywood was quickly installed around the nail to limit soil movement. Wire mesh and horizontal walers of rebar were placed; shotcrete was then sprayed over the entire surface of the panel. The shotcrete consisted of a low slump concrete, which was to develop a compressive strength of 30 MPa in 28 days.
5. After another 24 h, an anchor plate was installed on the end of the bar. A hydraulic jack was used to bring the nail to 133% of its working load for at least 1 min before being unloaded back to its working load. If the nail did not hold its required load after several loading attempts, another hole would be drilled adjacent to it.

Excavation, in alternating panels, proceeded around the site until a complete level was done. Before proceeding to the next lower excavation level, the piezometers inside and outside the site were read to ensure that the water was at least 2 m below the required elevation.

The construction of the shoring system was completed in less than five months.

11.8.1.5 Performance. Slope inclinometers and piezometers were installed around the portal excavation works prior to construction, at the locations shown in Figure 11.24. The purpose of the instrumentation was twofold:

(i) to monitor the performance of the temporary retaining system; and
(ii) to provide early warning of potential deep seated slide movements in the weak bedrock layers below the base of the excavation, as there was a potential concern that the construction works would reactivate the ancient landslide.

The slope inclinometers were installed in vertical boreholes and extended to well below the old bedrock failure planes. The standpipes and the pneumatic piezometers were also installed in boreholes, located within the sand deposit and the bedrock units, respectively, to monitor the effects of the groundwater dewatering program.

The instrumentation was monitored throughout the portal excavation and backfilling, and for several months after completion. The instruments were

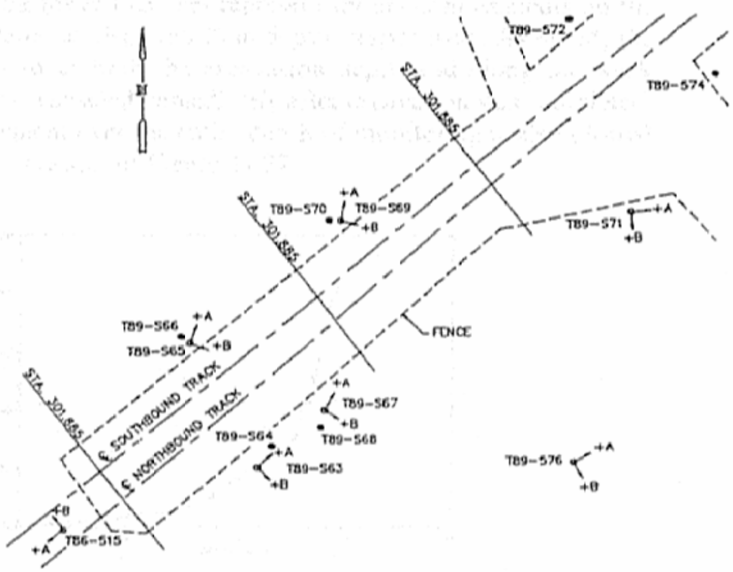

Figure 11.24 Excavation for tunnel portal at Edmonton, Canada—location of slope monitoring instruments.[72] ⊸ = slope inclinometer; ⊕ = piezometer.

initially read as the excavation reached each level of nails. This frequency was increased as the excavation neared the final depth.

Slope inclinometers. The inclinometer monitoring results provided profiles of lateral deflection in both the downslope ('A') direction and the cross-slope ('B') direction i.e. towards the excavation. The A and B directions for each instrument are shown in Figure 11.24.

A typical profile of lateral deflection towards the shotcrete wall is shown for inclinometer T89-S63 in Figure 11.25. The lateral deflection is shown at several stages of excavation. The maximum lateral movement at ground surface was approximately 84 mm towards the excavation, corresponding to a final depth of excavation of 15 m. The lateral movement increased relatively uniformly over the height of the excavation. Small, distinct shear movements of approximately 13 and 8 mm were also evident at depths of 19 m (elevation 632.0 m) and 27.4 m (elevation 623.5 m), respectively. When these shear movements were subtracted, the lateral movement over the excavation depth was approximately 64 mm.

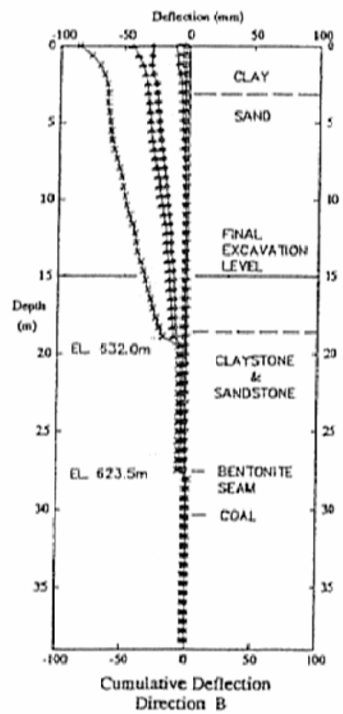

Figure 11.25 Excavation for tunnel portal at Edmonton, Canada—plot for slope inclinometer T89-S63 initiated on 6 March 1989 at an excavation depth of 0 m.[72] (□) 14 March 1989, depth 5 m; (▽) 5 April 1989, depth 7 m; (+) 17 April 1989, depth 9 m; (○) 16 May 1989, depth 12.8 m; (△) 7 June 1989, depth 13.3 m; (×) 8 November 1989, depth 15 m.

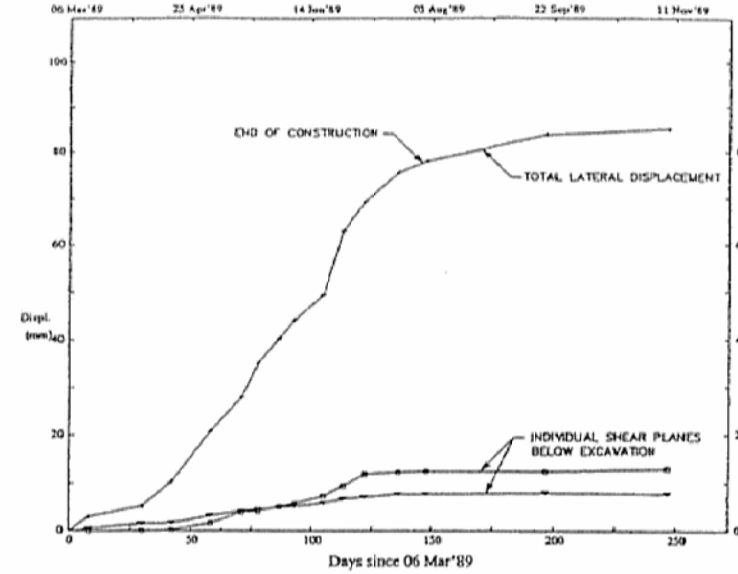

Figure 11.26 Excavation for tunnel portal at Edmonton, Canada—lateral displacement *v.* time for slope inclinometer T89-S63.[72] Displacements are shown in the negative (B) direction. (□) 18.9/19.5 m; (▼) 27.4/28 m; (+) 0/37.8 m.

The lateral deflections are also plotted against time in Figure 11.26. The upper line represents the lateral movement measured over the entire depth of monitoring, while the lower two lines represent the shear movements on the individual shear planes at 19 m and 27 m depth, respectively. As noted, the rate of movement—over both the excavation depth and along the weak bedrock slip planes—subsided immediately after excavation was completed. The lateral displacement over the entire depth of monitoring is also plotted against depth of excavation in Figure 11.27.

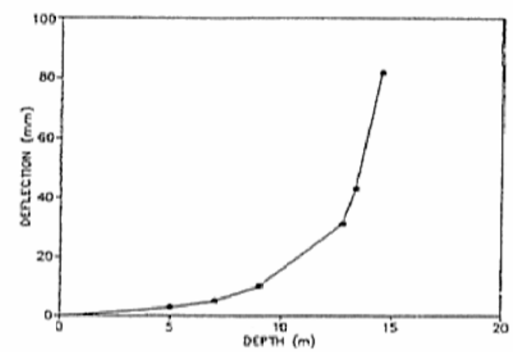

Figure 11.27 Excavation for tunnel portal Edmonton, Canada—lateral displacement *v.* depth of excavation for slope inclinometer T89-S63.[72]

Table 11.5 Summary of lateral displacements towards the excavation for tunnel portal at Edmonton, Canada.[72]

Slope inclinometer number	Maximum depth of excavation H (m)	Total lateral displacement* (mm)	Distinct shear displacement below excavation (mm)	Lateral displacement over excavation depth y (mm)	Angular distortion y/H (%)
T89-S15	20	60	5	55	0.28
T89-S63	15	84	24	60	0.40
T89-S67	13	60	15	45	0.35
T89-S69	8	82	45	37	0.46

* Measured between the top and the bottom of the slope inclinometer

Results of the slope inclinometer monitoring are summarized in Table 11.5. The lateral movement, y, recorded over the approximate excavation depth, was obtained by subtracting distinct shear movements below the base of the excavation from the total recorded movement. The resulting angular distortions (lateral displacement, y, divided by excavation depth, H) ranged from 0.28 to 0.46%. The resulting lateral movements were well within tolerable limits and the observed slope performance was considered to be acceptable, in terms both of angular distortion, and of overall slope stability.

Other monitoring methods. Two other methods were used to monitor the deformations of the shotcrete retaining walls.

(i) Electronic distance measurement monitoring prisms were attached to the south headwall and two monitoring stations were located at the north end of the site. Deformation of the headwall could thus be monitored from a remote location. Early readings indicated that the deformation of the wall could be measured with this system. Unfortunately, the construction sequence required the removal of the monitoring stations at an early stage, and therefore no further readings were recorded.

(ii) Graduated measuring tapes were attached, perpendicular to the wall, to several of the upper anchor heads. A theodolite was then used to sight between two points located parallel to the wall. In this way, it should have been possible to directly measure the movement of the anchor heads (and the shotcrete wall) in towards the excavation.

The limited readings fluctuated in terms of direction and could not be correlated with the slope inclinometer readings. Consequently, only the slope inclinometers were relied on for accurate deformation measurements.

Comparison with other case histories. Table 11.6 provides a comparison of the results from case histories for granular soils presented in Bruce and Jewell,[1] and for two locations on the project site. The length ratios were at the upper limit of the other case histories, perhaps reflecting the high factor of safety (6 to 9) used against shear and overturning. The bond ratio values were within the expected range. The most divergent result was the low strength ratio values: the published examples had four to eight times as much cross-sectional area of steel to soil as did the SLRT project.

Table 11.6 Comparison of other case histories with SLRT results.[72]

Derived parameters	Results from drilled and grouted nails in granular soils*	Results from SLRT project T89-S15	T89-S63
Length ratio	0.5–0.8	0.6	1.0
Bond ratio	0.3–0.6	0.3	0.4
Strength ratio (10^{-3})	0.4–0.8	0.1	0.1
Performance ratio (%)	0.1–0.3	0.3	0.4

*Bruce and Jewell[1]

The most important parameter is the performance ratio. In terms of this parameter, the SLRT shoring was at the upper values of other case histories, reflecting the influence of the variable stratigraphy and the greater height of the walls.

Problems encountered. Three serious problems were encountered with this shoring system over the course of the project. These included a relatively rapid 40 mm translation of the soil over the bedrock on a portion of the west wall, and a small localized shoring failure on the south wall.

The translational shearing observed on the west wall occurred in an area where the bedrock was at relatively shallow depth below the base of the excavation. This was indicative of one of the major concerns of the entire temporary shoring, namely mobilization of weak bedrock shear planes. The recommended remedial measure in such instances was to increase the number and length of soil nails in such areas to provide additional close reinforcement.

Late in construction, while attempting to read slope indicator T89-S65, the probe encountered a blockage in the casing at a depth of 18.0 m. The immediate reaction was that severe wall movement had sheared off the casing. Subsequent investigation of the blockage indicated that the shoring contractor had installed a nail through the casing.

Four weeks later, a 3 m high by 7 m wide panel of shotcrete failed at the bottom of the south wall. This particular section of the shotcrete wall had not been anchored, since it corresponded in location with the northbound tunnel, and the shoring contractor felt that no nails would be required here because the shoring was up against the bedrock. Unfortunately, the blocky nature of the bedrock, and resultant lateral pressure, caused this panel of the wall to be pushed out.

Remedial measures included building a soil buttress up against the failed mass, rebuilding the wall, and pressure grouting behind the wall.

11.8.1.6 Conclusions. The soil nailing shoring proposal proved to be a viable and relatively well-performing system, even in the variable and relatively poor ground conditions of landslide debris. It allowed the contractor to form and pour the portal structure without interference from bracing struts—measures that saved significant time and money.

The performance of this soil-nailed shoring system was similar to other published case histories with wall deflections 0.28 to 0.46% of the height.

The system was not, however, trouble-free. Constant monitoring had to be a prerequisite to evaluate the performance of the walls on a continual basis, provide early warning of potential hazards and enable timely implementation of slope remedial measures.

11.8.2 Design, construction and performance of a soil-nailed wall in Seattle, Washington[68]

11.8.2.1 Background. The soils in the Seattle area are generally heavily overconsolidated glacials, many of which are ideal for soil nailing. After a long history of the local use of prestressed ground anchorages, nails were first used in 1987, for temporary support of a building excavation. The present project was the first, however, to be designed and constructed by local firms. It was part of a temporary shoring system for a building excavation just east of the downtown area. The project consisted of two nailed walls, with heights of 10.7 m and 16.8 m adjacent to city streets, and two soldier pile and tieback walls adjacent to existing buildings. Soil nailing was not used to support the existing buildings because of the lack of experience with the system in local soil conditions, and an understandable initial concern with the potential for excessive movements associated with an unstressed shoring system.

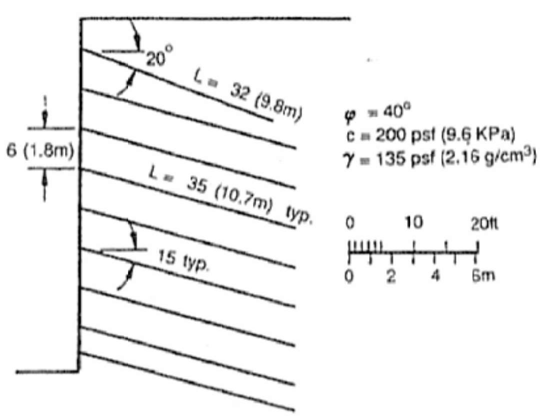

Figure 11.28 Design, construction and performance of a soil-nailed wall in Seattle, Washington—high-wall section.[68]

The soil conditions consisted of fill to a depth of 2.4 m, underlain by very dense glacial outwash sand and gravel, and very dense lacustrine fine sand and silt. The contact between the outwash and lacustrine deposits was encountered at the base of the excavation on the high wall, and at about mid-height on the low wall. Groundwater was below the base of the excavation. Soil properties used in the design analysis are shown in Figure 11.28.

Nails were generally installed on 1.8 m centers horizontally and vertically to a maximum length of 10.7 m. Holes were drilled with a 203 mm diameter, continuous flight, hollow stem auger. Nail bars consisted of Grade 150 Dywidag bars ranging from 25 mm to 32 mm in diameter. Nails were typically installed at an inclination of 15°, although the first row on the high wall was installed at 20° to avoid utilities. A typical section of the high wall is shown in Figure 11.28.

11.8.2.2 Design considerations. A force limit equilibrium analysis was chosen for the design of this wall. It was based on the 'Davis method'[30], but had been extensively modified to include more versatile modeling of a variety of wall and backslope geometries, variable nail lengths and spacings, and a treatment of nail capacities more consistent with local tieback design practices. The authors acknowledged that the limit equilibrium method has a number of limitations as a design tool. Perhaps the most significant was that limit equilibrium analyses can only provide an indication of the total nail force required to maintain a given factor of safety. The distribution of the total nail force among the individual nails cannot be solved for intrinsically, but must be assumed. The major shortcomings with existing limit equilibrium formulations for soil nailing design, including the Davis method, had been poor assumptions with regard to the nail force distribution.

In addition to difficulties with nail load distribution, limit equilibrium models do not address displacements of the reinforced soil mass. For applications such as an excavation shoring system in urban environments, such displacements are critical.

11.8.2.3 Instrumentation program. Instrumentation was installed to provide a better understanding of the aspects of soil nailing performance not well predicted by the limit equilibrium analysis, and to provide baseline information regarding the performance of soil nailing in specific local soils.

Vibrating wire strain gauges were installed on five nails, in a vertical section on the high wall (Figure 11.29). Four to six gauges were spot welded to the steel along the length of each nail. Economic constraint forced the use of a single gauge at each location. Thus, bending movements could not be measured and the effect of bending on the measured strains could not be explicitly determined. All gauges were located at the 3 o'clock position on the bar to minimize the potential for bending moment interference. All

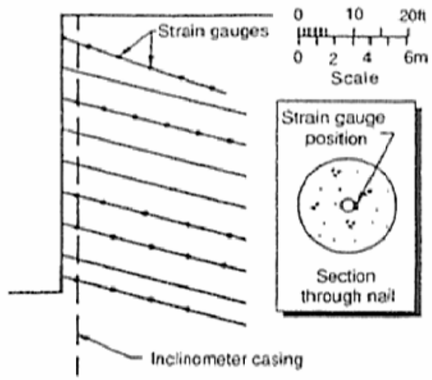

Figure 11.29 Design, construction and performance of a soil-nailed wall in Seattle, Washington—instrumented section.[68]

strain gauges were read daily during construction of the shoring wall, and monthly thereafter until the permanent basement wall was completed.

Two inclinometers were installed at a distance of 0.9 m behind the face of the wall. One was installed on the high wall, 3 m away from the strain-gauged section, and one was installed on the low wall. Inclinometers were read weekly during construction of the shoring wall, and monthly thereafter until the permanent basement wall was completed.

11.8.2.4 Data interpretation

Inclinometer data. Deflections measured on the high wall are shown in Figure 11.30. The total deflection at the top of the wall after the last lift of

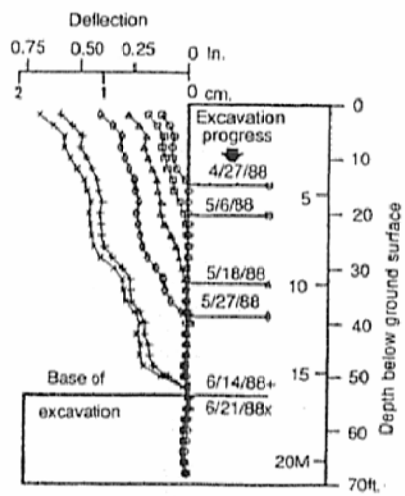

Figure 11.30 Design, construction and performance of a soil-nailed wall in Seattle, Washington—wall deflections.[68]

soil was excavated was 15 mm. One week later, after the nails and shotcrete facing had been installed on the last lift, the deflection at the top of the wall had increased to 18 mm. Monitoring of the inclinometer casings continued for another 10 weeks. No additional movements, within the accuracy of the instrument, were measured during that time.

The deflections were understandably slightly higher than might have been expected for a typical soldier pile and tieback wall in similar soil conditions. The maximum movement at the top of the wall was close to 0.1% of the height of the wall, or about what would be required to mobilize active earth pressure in dense granular soil.[73]

Strain gauge data. The strain gauge data provided considerable insight into the behavior of the nailed wall, including the short-term development of load on the nails, the change in nail loads with time, and indications of the combined influence of the steel and concrete portions of the nail section.

Figure 11.31 shows early strain measurements from nail level 6, plotted as a function of time. The excavation of each successive lift below nail level 6 can be seen as an initial rapid increase in strain related to the actual excavation of the next lift, followed by a slow increase in strain with time as the nails and shotcrete facing were installed. The construction of each of the three lifts below nail level 6 can be seen clearly in the strain data. This type of behaviour was seen on all nail levels. The influence of the excavation was most significant when the bottom of the excavation was within about 6.1 m of the nail. Following this the excavation effect decreased progressively but was still noticeable during every lift.

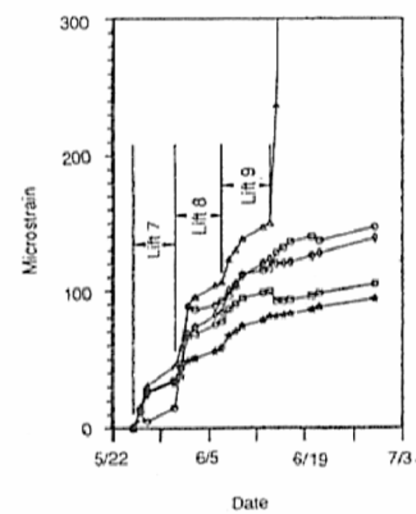

Figure 11.31 Design, construction and performance of a soil-nailed wall in Seattle, Washington—short-term strain from nail level 6.[68] (○) Gauge 1; (□) Gauge 2; (△) Gauge 3; (◇) Gauge 4; (∗) Gauge 5.

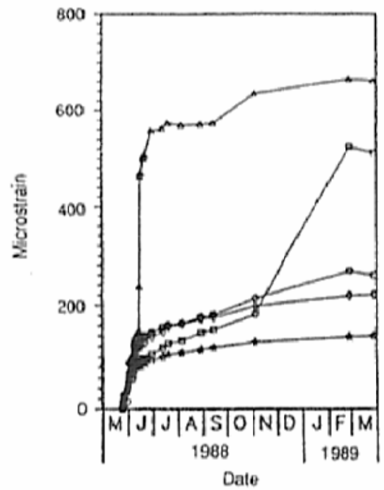

Figure 11.32 Design, construction and performance of a soil-nailed wall in Seattle, Washington—long-term strain from nail level 6.[68] (○) Gauge 1; (□) Gauge 2; (△) Gauge 3; (◇) Gauge 4; (•) Gauge 5.

Figure 11.32 shows the complete record of strain measurements from nail level 6. The strain increased rapidly at first, as active excavation took place. On completion of the excavation, the rate of strain increase slowed down in a pattern consistent with long-term creep of the nails. At the time monitoring was completed (nine months later), strains had increased by a factor of more than two over their values at the end of construction. This behavior is consistent with other cases where long-term measurements have been obtained.[59] It was ascribed to soil creep, and concrete creep and cracking (e.g. Figure 11.32 'jumps').

The determination of nail loads from strain data is not simple. It requires an assessment of the time-dependent effect of the grout on the overall stiffness of the nails. Concrete has a low modulus in comparison to steel, but the area of concrete in the composite nail section is greater than the area of steel by a factor of 50. Thus, the stiffness of the nail is strongly affected by the modulus of the concrete as long as the concrete remains effective in tension. When the concrete cracks, the local stiffness of the nail decreases substantially.

Two approaches were taken to calculate loads from the strain data. The first approach was to calculate a composite stiffness for the steel/concrete section based on reasonable assumptions regarding the section geometry and concrete modulus. A time-dependent concrete modulus was used, based on 3-, 7-, and 28-day strengths from 17 cylinders of the nail grout. The resulting maximum nail loads at each level are shown in Figure 11.33 as indicated by the curve titled 'Indirect method.'

The disadvantage of this approach is that the area and modulus of the concrete are based on assumptions that may not reflect actual conditions.

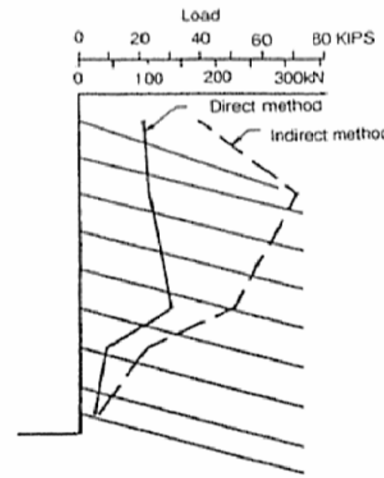

Figure 11.33 Design, construction and performance of a soil-nailed wall in Seattle, Washington—measured load *v.* depth.[68]

Furthermore, it is assumed that the load is shared in direct proportion to the stiffness ratio of the concrete and steel, with no allowance made for creep in the concrete. As indicated previously, creep may account for half or more of the deformation, particularly early on when the strength of the concrete is low and the nail loads are increasing most rapidly. This will result in less of the load being carried by the concrete than the stiffness ratio suggests and actual nail loads will be lower than predicted. Thus, loads determined by this method are considered to be limiting upper bound values.

The second approach was to assume that the jumps in strain caused by cracking of the concrete represented the full release of tensile stress in the concrete. Thus, the strain on the low side of the jump was related to the stiffness of the composite section, while the strain on the high side of the jump was related to the stiffness of the steel alone. By assuming that the total load remained constant over the strain jump, the stiffness of the uncracked composite section could be back-calculated. The resulting maximum nail loads are shown in Figure 11.33, as indicated by the curve titled 'Direct method'.

The advantages of this approach are that no assumptions need be made regarding the variation of concrete modulus with time, and any transfer of load to the steel which may have occurred by creep in the concrete is accounted for directly. The disadvantage is in the assumption that the strain jump results from the full release of the tensile stress previously carried by the concrete. This is only true if the gauge is located at the crack. If the strain gauge were located some distance away from the crack, the increase in strain would reflect only a portion of the tensile stress released by the concrete. The jumps in strain do provide a lower bound indication of the tensile stress in the concrete and thus a lower bound indication of the total nail load.

Loads determined by the indirect method were substantially in excess of those required under active conditions. However, the magnitude of the wall deflections suggests that active conditions had been established. Consequently, loads determined by the indirect method may not represent the best estimate of actual nail loads. Nail loads calculated by the direct method were consistent with active conditions, and thus may be more realistic.

The location of the maximum bar force is not affected by the assumptions made in converting from strain to force. The distribution of load along the length of each bar is shown in Figure 11.34, using the loads calculated by the direct method. Two maxima were seen on some bars. The peak at the front of the bar may be related to bending forces caused by the weight of the shotcrete facing hanging from the nails during excavation of underlying lifts. The peaks farther away from the face map the locus of maximum strain in the soil mass. This should, in theory, coincide with the critical failure surface predicted by the limit equilibrium analysis used to design the wall. The surface predicted by the analysis is shown in Figure 11.34. The maximum tensile force line typically used in the design of reinforced earth walls[74] is also shown and is actually in better agreement with the measurements than the limit equilibrium prediction.

11.8.2.5 Critique of design methodology. Limit equilibrium analyses are currently common tools for the design of soil-nailed walls in the USA,

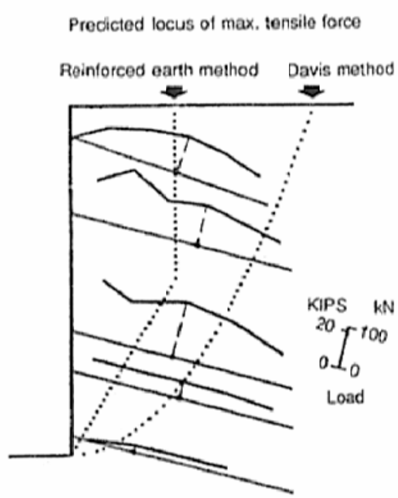

Figure 11.34 Design, construction and performance of a soil-nailed wall in Seattle, Washington—load distribution on nails.[68]

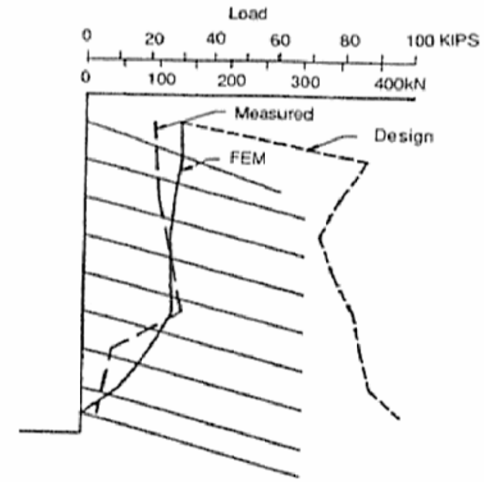

Figure 11.35 Design, construction and performance of a soil-nailed wall in Seattle, Washington—maximum nail load *v.* depth.[68] FEM = finite element method.

although it is well known that such analyses do not provide good estimates of the magnitude and location of the maximum nail forces.[6,7,75] However, working stress analyses can be cumbersome as design tools. Hence, limit equilibrium analyses are commonly used to estimate nail forces, and to design wall systems accordingly. Experience with the Davis method indicates that the predicted nail forces are conservative.

Figure 11.35 presents a compilation of the maximum nail force at each nail level as determined by the original design methodology (Davis method), the field measurements (only the direct method interpretation is shown), and finite element analysis. Clearly, the nail loads determined using the Davis method are excessive in comparison with loads obtained from the field measurements and the working stress analysis.

A number of factors contribute to the overestimation of nail loads. In the first place, the design loads calculated using the Davis method incorporate a factor of safety of 1.5 on soil shear, resulting in more load being carried by the nails. The other loads do not include factors of safety. However, the factor of safety alone does not explain the difference in nail loads.

The primary shortcoming of the Davis method is in the assumptions made regarding the distribution of the total nail forces among the individual nails. The method assumes that the force carried by each nail is proportional to the length of the nail behind the critical failure surface. This bears little resemblance to the distribution of nail forces predicted by working stress analyses or measured in case histories.

During the early stages of the excavation—when the nail length may be considerably greater than the height of the excavation—this assumption leads to excessively high nail forces, and high overall factors of safety. As the excavation is lowered, the failure surface extends deeper within the soil mass and the effective anchor length of the nails decreases. This often results in predicted nail loads that decrease with increasing depth of excavation. The load distribution also results in higher nail forces being carried by progressively lower nails.

This is not a good model of soil-nailed wall behavior. Field data[59] indicate that the soil strength is mobilized fairly quickly. In other words, the factor of safety with respect to soil shear is close to 1 during the entire excavation process. The forces mobilized in the nails are just sufficient to maintain the system at a factor of safety of 1. As conditions change, for example the excavation is lowered, additional forces are mobilized in the nails as needed. As long as sufficient excess capacity is available in the nails the wall will remain stable. The force mobilized in a nail is related predominantly to successive lowering of the excavation. Thus, loads in the lower nails are typically small.

A more rational approach to modeling the mobilization of nail forces would be to compute the total nail force required to maintain a factor of safety of 1 at each stage of the excavation. Limit equilibrium methods are capable of this analysis, since it is essentially an ultimate condition. For design purposes, suitable factors of safety could be applied reflecting the degree of uncertainty in the various strength parameters, which results in a somewhat higher total nail force than for the factor of safety of 1 condition.

This total nail force must then be divided between the individual nails. Limit equilibrium methods are not able to distribute nail loads explicitly, but assumptions can be made on the basis of empirical data—as has been done for years in the design of other types of braced excavations. In fact, field data[59] indicate that the pressure diagram for a conventional braced excavation[60] may provide a reasonable approximation of the nail load distribution. As shown in Figure 11.36, the project data also fit this model. By accepting the limitations of the limit equilibrium method and applying an empirical understanding of load distributions, more reasonable designs may be achieved.

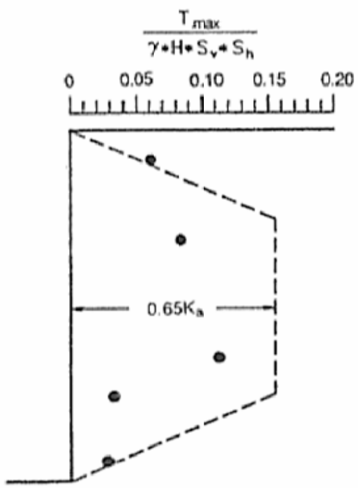

Figure 11.36 Design, construction and performance of a soil-nailed wall in Seattle, Washington—normalized nail force *v.* braced cut pressure distribution.[68]

11.8.2.6 Conclusions. Considerable insight has been gained into the mechanics of the performance of soil-nailed walls through the instrumentation program described in section 11.8.2.3. In particular, a better appreciation has been developed of the creep behavior of grout, and its effect on nail loads calculated from strain data.

Appropriately calibrated finite element analyses were able to provide a close approximation of the behavior of the wall system. Properly characterized joint elements between the soil and nail elements proved to be critical to the accurate calculation of nail loads.

Incorporating a more rational approach to the modeling of nail force mobilization and distribution, based on the body of knowledge that has been developed in recent years regarding the actual behavior of soil-nailed walls, may significantly improve the ability of limit equilibrium analyses to predict nail forces.

11.9 Final remarks

Soil nailing is a technique of great potential, proven cost-effectiveness and excellent technical performance. It has been used routinely—in appropriate ground conditions—in North America and Western Europe for over 20 years, while each year there are reports of first applications in other countries. Soil nailing is used on both new and remedial projects, and for both temporary and permanent purposes.

Soil-nailed structures are relatively easy and fast to construct, and lend themselves well to informative monitoring and test programs. The ongoing debate over design methods has ensured that the technique has continued to attract the attention of experienced and innovative geotechnical engineers. The main drawback of current design methods is that they do not provide an estimate of either the structural displacement, or the nail forces mobilized at the expected working loads. Indeed, the development of the technology has been essentially empirical, and field experience has markedly preceded theory and fundamental research. The ongoing programs of full-scale experiments, and laboratory and theoretical studies are, however, leading to significant improvements in design methods and predictive capabilities.

Consequently, there is every reason to believe that the number of applications will continue to grow, to the benefit of owner and engineer alike, on a world scale.

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