Soil Nailing: Application and Practice — part 2

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

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

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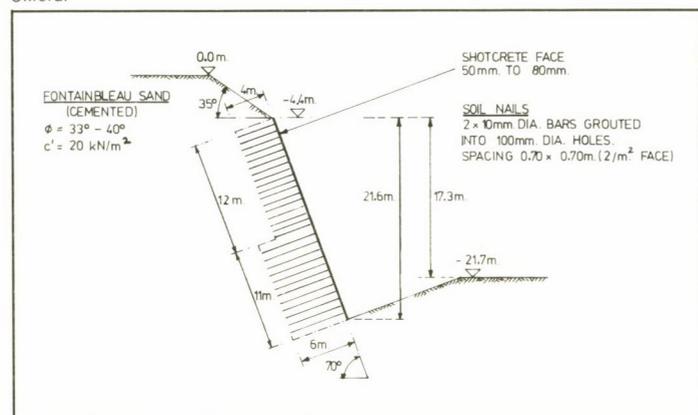
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of current experience, each aspect of soil nailing construction is examined to establish guidelines for good practice.

4. Review of illustrative case histories

The seven case histories have been selected for review on the basis of scale and historical significance. Of the five summaries of nail installations in new construction, three are from France, with two from the United States. This emphasises both the pioneering role of engineers in these two countries, and, regrettably, the shortage of published data on case histories from West Germany. The important work in West Germany on field trials is described in the companion Paper.*

One of the two case histories of applications in remedial work is from France, and the British example has importance

given the domestic trends for structural repair and maintenance engineering as opposed to new construction.

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

NEW CONSTRUCTION

4.1 Retaining wall for the French railways (SNCF) at Versailles-Chantiers, France (1972) (Rabejac and Toudic 1974, Toudic 1975, Hovart and Rami 1975, Medio *et al* 1983.)

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

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

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

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

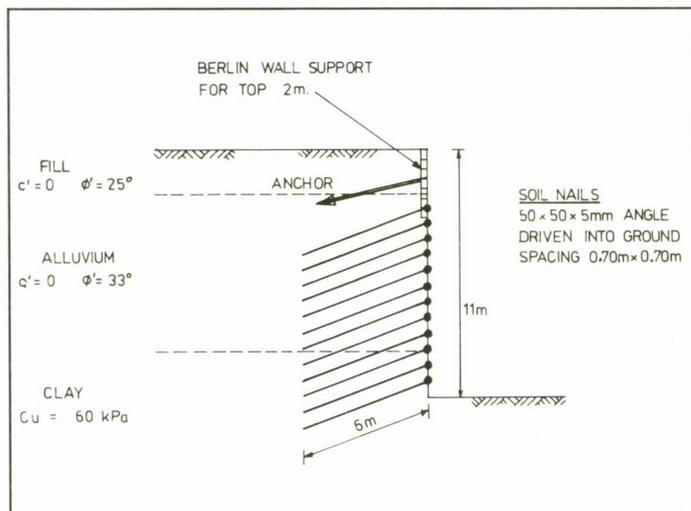*Fig. 20. Cross-section of the Nogent-sur-Marne A86 excavation (After Goulesco and Medio, 1981)*

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

and the geometry of these cuts was dictated by a highway at the top of the slope (Fig. 18). A slope height of up to 22m was required at a 70° slope angle. The decision of the Engineer, described as "la plus originale et la plus audacieuse", was the first recorded use of soil nails for temporary slope stabilisation, in this case until a permanent gravity retaining wall was constructed (photograph 6). The soil consisted of cemented Fountainbleu Sand, with 80% of particles 0.1 to 0.4mm in diameter and an angle of friction ranging from 33° to 40°. A "cohesion" of $20kN/m^2$ was measured, caused by fines, capillary water and weak cementing.

Cuts were made generally 1.4m high and in 100m lengths but were reduced to 0.7m high where pockets of Marl were found. A 50mm x 50mm steel grillage was fastened to the face before 100mm rotary drilled holes were formed, at a density of two per m^2 (i.e. a vertical and horizontal spacing of 0.7m). The holes were drilled perpendicular to the face, at 20° below the horizontal. Two 10mm bars were grouted into each hole with a neat cement grout (w = 0.8). The face was then formed with 50-80mm of sprayed concrete as a protection against ravelling and to bond the various elements of the system together.

During the works, pullout tests were conducted on 4m long nails and gave the following results:

after 12 hours, resistance greater than 15kN
after 36 hours, resistance greater than 30kN
after 7 days, resistance greater than 70kN
after 11 days, resistance greater than 90kN

In total, 18 100 No. 4m long nails (for the upper 12m), and 7 100 No. 6m long nails were installed in a surface area of 12 600m². The excavation took 250 working days, and involved the removal of 80 000m³ of soil. The reinforced concrete wall (0.6-1.6m thick) was then cast for the long term support of the cutting.

4.2 Excavation for the underground car park, Boulevard Victor, Paris (1978) (Medio et al, Goulesco 1984)

Following the success of the soil nailing at Versailles-Chantier, the contractor Bouygues in 1974 executed a small contract involving 1 000m² of excavation support up to 12m high, at Les Invalides Metro Station in Paris. This particular contract was noteworthy since it was the first application with driven nails (49mm tubes) as opposed to the drilled and grouted type. Bouygues then applied the principles of this technique in their next application, the stabilisation of the 12 000m², 11m deep excavation for a car park for la Ville de Paris in the Boulevard Victor (Fig. 19). This contract was executed in 1978 and boasted several "firsts"

— first "mixed wall", Berlin wall at the top over soil nailing (due to services in the upper zone, and poor made ground)
— first applications of angle steel reinforcement (50 x 50 x 5mm) driven directly into the soil
— first application of soil nailing in an urban area within close proximity of high-rise buildings and traffic routes.

The "mixed wall" solution has been used since in locations including La Defense (Paris) and in Biarritz (Goulesco, 1984).

4.3 Cut and cover excavation for A86 Autoroute, Nogent-Sur-Marne, France (1980) (Goulesco and Medio, 1981, Cartier and Gigan 1983).

In its path across the city of Nogent sur Marne, the A86 autoroute is constructed in a covered trench 2.3km long. For much of the

TABLE 1: Soil nailing case histories with construction details; Granular soils

PROJECT NAME Description and Scale	DATE	MAIN REFERENCES	GROUND CONDITIONS Cohesion/Friction/- Unit Weight (kN/m2/degrees/kN/m3)	SLOPE ANGLE degrees
DRILLED AND GROUTED				
VERSAILLES-CHANTIER railway cutting 70 degree cutting stabilisation over 12,600 m2 facing, up to 21.6 m deep.	1972	Rabejac & Toudic, 1974 Toudic, 1975 Hovart & Rami, 1975	Cemented Fontainbleu Sand 20 32 20	70
GOOD SAMARITAN HOSPITAL, Portland, Oregon Vertical foundation excavation over 2140 m2 facing, up to 13.7m deep.	1976	ENR, 1976 Shen et al., 1981a Shen et al., 1981b	Silty Fine Sand 0 34 16.2	90
MUR DE FERRIERES-SUR-ARIEGE, France 80 degree cutting up to 19m deep in weathered Schists	1982	Blondeau & Ledeuil, 1984	Weathered Schists 0 45 20	80
PPG BUILDING, Pittsburgh, Pennsylvania Vertical foundation excavation up to 9.1m deep, adjacent to sensitive buildings.	1982	Nicholson, 1986b	Alluvial Silt, Sand & Gravel 0 25-35 15.4	90
CUMBERLAND GAP, Kentucky 75 degree cutting stabilisation over 900 m2 facing up to 12m deep.	1985	Nicholson, 1986a	Weathered Shales & Siltstones 0 38 16.4	75
DRIVEN NAILS				
LES INVALIDES METRO, Paris Vertical excavation over 1000 m2, up to 12m deep.	1974	Medio et al., 1983 Goulesco, 1984 Guilloux, 1982	Fill 0 25 * Alluvium 10 20 * Sand 0 35 *	90
PARKING BOULEVARD VICTOR, Paris Vertical excavation over 12000 m2, up to 11m deep, with top 4m Berlin wall.	1978	Medio et al., 1983 Goulesco, 1984	Fill 0 25 * Sand 0 33 * Clay 60 0 *	90
AUTOROUTE A86, Nogent-sur-Marne Vertical excavation over 900 m2, up to 11.6m deep, for cut and cover road.	1980	Goulesco & Medio, 1981 Cartier & Gigan, 1983	Fine sand 0 33 20	90
PARKING DE MAISONS LAFITTE, Paris Vertical excavation over 2500 m2, up to 12m deep, facing used in permanent works.	1981	Medio et al., 1983	Sand * * * Marls * * *	90

NOTES:
1. (as.) = assumed
2. (est.) = estimated
3. (eq.) = equivalent
Conversion factors for Hurpinoise Equivalent Equivalent

TABLE 2: Soil nailing case histories with construction details; Overconsolidated Soil

PROJECT NAME Description and Scale	DATE	MAIN REFERENCES	GROUND CONDITIONS Cohesion/Friction/- Unit Weight (kN/m2/-/kN/m3)	SLOPE ANGLE degrees
DRILLED AND GROUTED				
LA CLUSAZ, France Steep excavation up to 14m deep over 1000 m2 facing in very compact Moraine	1980	Guilloux, 1982 Guilloux et al., 1983	Compact Moraine with high friction 0 55 - 35 22	80
DRAGUIGNAN, France Steep excavation up to 11m deep over 750 m2 facing in Marl/Limestone deposits.	1981	Guilloux, 1982	Interbedded Marl and Limestone 20 35 *	80
GRAND PARC DE CIMIEZ, NICE, France Steep excavation over 400 m2 facing up to 11 m deep, in mixed Marls.	1981	Guilloux, 1982	Sandy Marls 10 35 *	80
Marl slope stabilisation, West Germany Steep excavation up to 16m deep in a red Keuper Marl.	*	Gudehus, 1982	Keuper Marl	80 (est.)
ELYSIAN PALACE, Nashville Permanent vertical excavation over 170 m2 facing up to 8.2 m deep, in firm clay.	1985	Unpublished	Firm Silty Clay * * *	90
DRIVEN NAILS				
PARAVALANCHE DE CHAMONIX, France Permanent vertical excavation 8m deep over 1500 m2 in compact Moraine	1983	Goulesco, 1984 Gigan, 1986	Compact Moraine with boulders 10 40 *	80 overall

NOTES:
1. (as.) = assumed
2. (est.) = estimated
3. (eq.) = equivalent
Conversion factors for Hurpinoise Equivalent Equivalent

TABLE 4: Examples of soil nailing used for repairs

PROJECT NAME Description and Scale	DATE	MAIN REFERENCES	GROUND CONDITIONS Cohesion/Friction/- Unit Weight (kN/m2/degree/kN/m3)	
DRILLED AND GROUTED				
FREJUS WALL, France Repair to 60 m2 of facing of a reinforced earth wall caused by frost action	1981	Long et al., 1984	Compact granular fill	
COURS D'HERBOUVILLE, Lyon, France Slope 35m high collapsed July 1977. Nailing provided temporary stability for the repair works in 10m cuts	1982	Gausset, 1985	Silty Boulder Clay 0 25 18 Cemented basal beds 50 35 20	
DENHOLME CLOUGH, Bradford, UK Repair to 120m collapsing drystone wall supporting 3m cut in sloping terrain	1985	Unpublished	Head material over completely weathered sandstone	

HEIGHT (m)	NAIL LENGTH (m)	NAIL Diameter (mm)	NAIL HOLE Diameter (mm)	NAIL SPACING Horizontal/Vertical/Area (m/m/1 per m2)	NAIL ANGLE (-Horiz) degs	CUT STEPS (m)	FACING (mm)	MEASURED MOVEMENT (at crest) (mm)	NOTES	OVERALL L/H	DERIVED PARAMETERS BOND (d)hole.L spacing	STRENGTH (d)bar^2 spacing	MEASURED MOVEMENT movement height
11.3	4.0	14.1	100	0.70 0.70 0.49	20	1.4	80		First recorded application of nailing for temporary slope stabilisation	0.35	0.82	4.06E-04	
21.6	6.0	14.1 (eq.)	100	0.70 0.70 0.49	20	1.4	80			0.28	1.22	4.06E-04	
11.3	8.5	38.1	100	1.83 1.53 2.80	15	1.5	100		First published USA project	0.75	0.30	5.18E-04	
11.3	7.0	38.1	100	1.22 1.53 1.87	15	1.5	100	33	One wall section adjacent to a four storey building	0.62	0.38	7.78E-04	0.00292
16.5	9.0	32.0	56	2.00 1.00 2.00	10	2.0	100	24 (max)		0.55	0.25	5.12E-04	0.00145
9.1	7.0	30.0 (est.)	127	1.22 1.22 1.49	15	1.2	200	3.2 (max)	First combined nailing with face stabilisation by preinjected grout columns	0.77	0.60	6.05E-04	0.00035
12.3	9.0	30.0 (est.)	114	1.52 1.52 2.31	15	1.5	150	10 (max)	Use of fabric drains placed immediately behind the shotcrete facing	0.73	0.44	3.90E-04	0.00081
12.0	6.0	28.0 (eq.)	49	0.70 0.70 0.49	20	1.4	*		First application with nails driven directly into the ground	0.50	0.60	1.60E-03	
11.0	6.0	25.2 (eq.)	64 (eq.)	0.70 0.70 0.49 (assumed)	*	*	*		First "Hurpinoise" wall with driven angle iron reinforcement	0.55	0.78	1.30E-03	
5.6	5.5	25.2	64	0.70 0.70 0.49	20	1.4	50-100		First "Hurpinoise" wall to be fully monitored and with experimental sections	0.98	0.72	1.30E-03	
11.6	7.0	30.3 (eq.)	76 (eq.)	0.70 0.70 0.49	20	1.4	50-100			0.60	1.09	1.87E-03	
12.0	6.0	25.2	64	0.70 0.70 0.49 (assumed)	*	*	250		Shotcrete facing used as part of the permanent works finish	0.50	0.78	1.30E-03	

steel angle: Angle (B x B x t)
bar diameter: (d)bar = (8Bt/pi)^0.5
hole diameter:(d)hole = (4B/pi)

ls — Moraines and Marls

HEIGHT (m)	NAIL LENGTH (m)	NAIL Diameter (mm)	NAIL HOLE Diameter (mm)	NAIL SPACING Horizontal/Vertical/Area (m/m/1 per m2)	NAIL ANGLE (-Horiz) degs	CUT STEPS (m)	FACING (mm)	MEASURED MOVEMENT (at crest) (mm)	NOTES	OVERALL L/H	DERIVED PARAMETERS BOND (d)hole.L spacing	STRENGTH (d)bar^2 spacing	MEASURED MOVEMENT movement height
14.0	11.0	25.0	100 (as.)	3.00 2.00 6.00	10	2 (est.)	150	15 (10 to 20)	Nailing replaced a section of Berlin wall. Movements and forces recorded in winter.	0.79	0.18	1.04E-04	0.00107
11.0	11.0	32.0	100 (as.)	3.00 2.00 6.00	15	2 (est.)	Metal facing			1.00	0.18	1.71E-04	
11.0	8.0	28.0	100 (as.)	2.50 2.00 5.00	10	2 (est.)	100-200			0.73	0.16	1.57E-04	
16.0	8.0	*	*	* * *	10 (est.)	*	*	24 (max.)	Almost no creep detected. Three levels of nails instrumented.	0.50			0.0015
8.2	4.1 average	25.4	102	1.50 1.60 2.40	15	1.5	150		Permanent excavation with good performance reported.	0.50	0.17	2.69E-04	
8.0	8.0	30.3 (Angle 60x60x6)	76 (Angle 60x60x6)	1.10 0.60 0.66	20	1.2 (as.)	*		First permanent Hurpinoise structure (note some nails were drilled & grouted).	1.00	0.92	1.39E-03	

steel angle: Angle (B x B x t)
bar diameter: (d)bar = (8Bt/pi)^0.5
hole diameter:(d)hole = (4B/pi)

SLOPE ANGLE (degree)	HEIGHT (m)	NAIL LENGTH (m)	NAIL Diameter (mm)	NAIL HOLE Diameter (mm)	NAIL SPACING Horizontal/Vertical/Area (m/m/1 per m2)	NAIL ANGLE (-Horiz) degs	CUT STEPS (m)	FACING (mm)	NOTES	OVERALL L/H	DERIVED PARAMETERS BOND (d)hole.L spacing	STRENGTH (d)bar^2 spacing
90	6.0 repaired	5.0	28.0	100 (ass.)	1.50 1.50 2.25	10	-		Collapsed facing replaced and existing panels were nailed in damaged area.	0.83	0.22	3.48E-04
75 overall	10.0 repair steps	10.0	30.0	100 (ass.)	3.00 2.50 7.50 (est.)	10	2.5	150	Nailing used to stabilise 10m vertical cuts. Permanent support by anchorages and abutments.	1.00	0.13	1.20E-04
80	3.0	5.0	16.0	115	1.50 1.50 2.25	15	-	50 on dry-stone wall	Probably first soil nailing repair in the UK. Existing wall stabilised by nails.	1.67	0.26	1.14E-04

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

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

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

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

The soil was mainly sandy alluvial deposits, (ø' = 33°, c' = 10kN/m², γ = 20kN/m³) but with frequent pockets or lenses of clay, silt and fine sand. In general the nailed wall was constructed in ground regarded as more difficult than that encountered by the Berlin Wall.

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

The main wall consisted of driven 50 x 50 x 5mm angles in the upper 9 rows (5.5m long), and 60 x 60 x 6mm angles in the lower eight rows (7m long). Cuts were generally 1.2m high with the nails spaced at two per m². The angle steel reinforcement was bonded to the face mesh and then 50-80mm of sprayed concrete placed (Photograph 7). The main wall was instrumented and showed excellent performance, including:
— maximum vertical displacement 14mm at the crest, completely stabilised within 4 months
— pullout bond of 15 to 20kN/m length of reinforcement, largely independent of depth
— mobilisation of the maximum pullout force at displacements of a few millimetres.

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

4.4 Foundation excavation for the extension to the Good Samaritan Hospital, Portland, Oregon (1976)
(Shen et al 1981 a, b).

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

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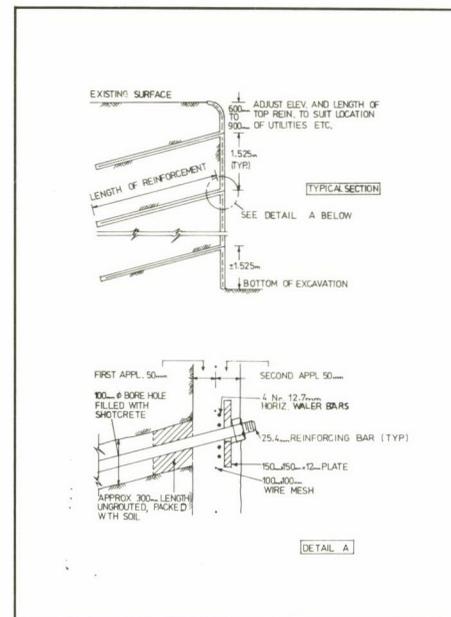

Fig. 21. Details of the soil nailing at the Good Samaritan Hospital site (Shen et al, 1981a)
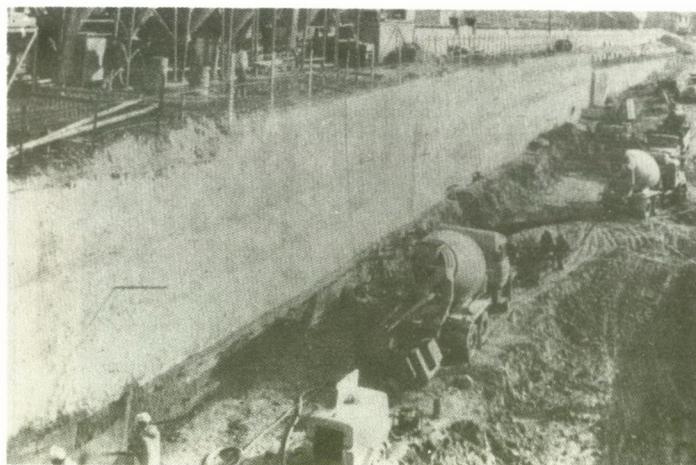

Photo. 7. General view of the Nogent-sur-Marne A86 excavation (Medio et al, 1983)

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

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

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

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

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

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

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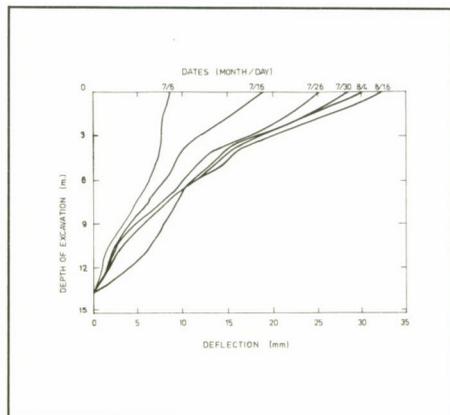

Fig. 22. Horizontal deflections measured on the western boundary wall at the Good Samaritan Hospital excavation (Shene et al, 1981a)
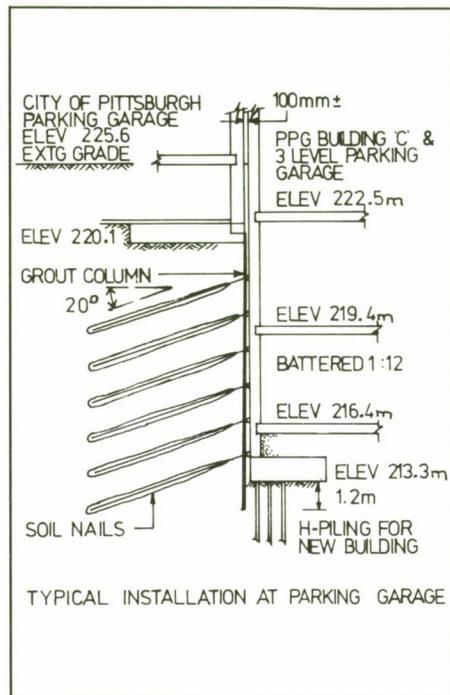

Fig. 23. Cross-section at the PPG Building Parking Garage excavation, showing the face pretreated by grouting (Nicholson and Wycliffe-Jones, 1984)

caused particular concern where the excavation level had to reach 3.3-10m below their foundations:
(i) The building of the Catholic Diocese of Pittsburgh main office; built in the 1950's, of masonry and brick construction, on spread footings on alluvial sand, silt and gravel.
(ii) Third Avenue Parking Garage: five stories high on spread footings.
(iii) Landmark Buildings: built around 1900, brick bearing wall, three stories high, with strip masonry and rubble footings.

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

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

The particularly sensitive nature of the Diocese Building foundations required another level of protection against

movement. Groups of vertical 450kN mini-piles were installed to support about half of the total vertical load of the foundation directly, before the excavation proceeded (Fig. 24).

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

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

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

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

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

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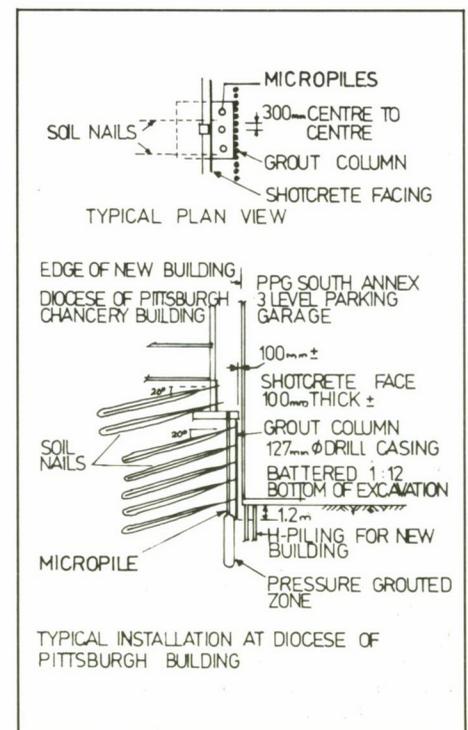

Fig. 24. Excavation adjacent to the Diocese Building showing direct underpinning through the nailed zone to minimise settlement (Nicholson and Wycliffe-Jones, 1984)

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

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

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

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

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

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

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

The following sequence of operations was conducted (Fig. 15):
Drainage. A series of 50mm diameter temporary drain holes were drilled 300mm above the road level at 2m horizontal spacings. After shotcreting and nail installation, two rows of permanent 115mm dia. drain holes were drilled 300mm and 1 500mm above road level extending to the back of the drystone wall at 3m horizontal spacings.
Repairing wall facing. Sections of wall that had collapsed were rebuilt in a single skin brickwork with strengthening piers. The void behind was backfilled with a sand cement grout with Conbex 653 additive to restrict flow. A layer of light steel mesh was fixed to the drystone wall and 50mm thickness of concrete was sprayed to hold the drystones in place during the subsequent nailing.

Vertical construction joints were formed at 15m centres.
Nailing. A track-mounted drilling rig was used to drill the 115mm diameter nail holes, with one nail for 2.5m² face. The reinforcement was 16mm diameter high yield bar with a threaded end, and the grout was a neat cement mix (w = 0.45). A corrugated PVC sleeve was placed through the upper section of each hole to restrict lateral travel of grout in the masonry. After setting of the grout, a headplate was fitted and a nominal stress applied to each nail. Pull-out tests were conducted on three test nails, and confirmed that the bond was well in excess of the required working conditions.
New masonry face. A new stone facing was then erected as a finish to the stabilised wall.

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

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

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

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

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

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

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

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:

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

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

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

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

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

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

5.2 Observations on case histories
A few general observations may be made based on the tabulated data:
5.2.1 Steep granular slopes
For steep slope (80° or more) projects in granular soils there is a reasonable correlation of the derived parameters as shown in Table 5.

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

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

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

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

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

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

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

experimental projects in the companion Paper).

5.2.2 Comparison between projects in granular soils and stiff clays

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

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

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

5.2.3 Comments on reported failures

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

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

6. Construction

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

6.1 Excavation and facing

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

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

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

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

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

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

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

6.2 Drainage

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

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

6.3 Installation of nails

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

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

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

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

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

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

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

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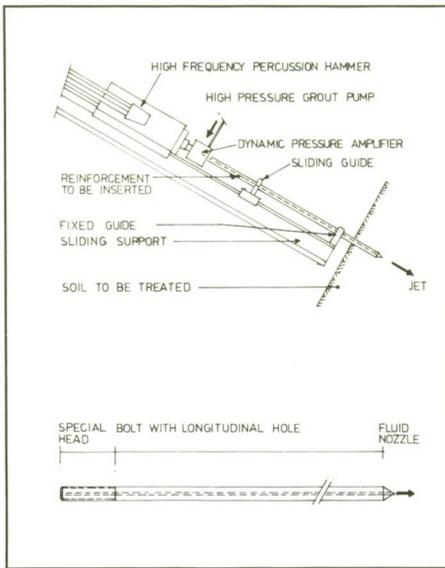

Fig. 25. The "Jet Bolting" technique for soil nail installation which combines vibro-percussion and high grout pressure at the nail tip (Louis, 1984)

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

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

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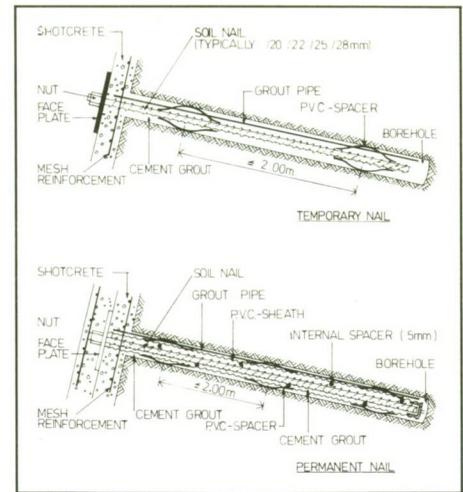*These numbers most commonly used in soil nailing

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

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

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

6.4 Reinforcement and corrosion protection

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

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

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

TABLE 7: Typical nail dimensions and properties in European practice

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

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

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

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

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

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

6.5 Slope claddings

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

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

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

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

6.6 Instrumentation and monitoring

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

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

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

distance measurements to be made.

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

7. Conclusions

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

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

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

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

Acknowledgements

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