

Developments in Geotechnical Construction Processes for Urban Engineering

In situ reinforcement, small-diameter cast-in-place minipiles and ground treatment are three newer technologies that can address infrastructure problems in urban environments.

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THE CONTRACTING skills of an engineering community develop in response to the times and the environment. Many of the largest civil engineering firms mastered the practicalities and controls of massive earth-moving schemes during the heydays of dam and highway construction. More recently, structural engineers have responded to the challenges of oil exploitation: the colossal platforms, both on- and off-shore, in the world's less hospitable regions are fitting testimony. At different times, the innovative talents of bridge engineers, railroad builders and

hydraulic specialists, for example, have all been in particular demand.

One of the major themes in world construction today is infrastructure development, redevelopment and upgrading. There has been especially intense activity in subway and sewage projects in both developed and developing countries: Cairo's current re sewerage scheme, for example, involving many miles of bored tunnels, cut and cover excavations and huge pump stations, is one of the major engineering projects in the world. In addition, many old and dilapidated "inner city" areas are being aggressively redeveloped for commercial, residential or recreational purposes. Examples in this category extend from London's Docklands to Pittsburgh's Golden Triangle, and from Boston's waterfront to St. Louis' railway station.

Irrespective of purpose or geography, there is a common set of factors facing engineers engaged in such projects:

- Construction must be carried out in heavily urbanized areas with existing above and below ground structures and

services.

- Construction is confined to specific sites within these areas, and so must accommodate the particular geological conditions. Few major cities are founded directly on solid rock: most have extensive artificial fills overlying glacial, alluvial or marine deposits in which the groundwater level is within the depths touched by construction.

- Construction is witnessed by, and directly impacts, the inhabitants of these areas.

As a consequence, urban construction must resolve the problems of restricted access, unfavorable ground conditions and environmental compatibility. There is no room — literally and metaphorically — for the “walk away” solution: in situ solutions must be found since relocation to an easier area is not usually a viable option. In addition, there is increasing pressure, nationally, that such solutions must be achieved at minimum cost. As Nicholson wrote: “For a long time, America used to have surplus money to ‘throw’ at problems . . . this has not been true for a number of years.”¹ Furthermore, competition between the growing number of highly competent specialist contractors is intense, and so cost competitiveness is crucial from their viewpoint also.

These problems have already been addressed for many years in several countries of the Old World and the Far East for many reasons ranging from the need to repair war damages to the necessity to provide proper facilities for growing populations in geographically restricted areas. These countries have fostered the development of many original geotechnical construction techniques, popularly referred to in America as the “European Technologies.” And, of course, such techniques have been allowed to mature in contractual and legal environments that encourage calculated risk while not punitively penalizing imperfections.

Engineers in the United States frequently debate the reasons that they appear to be importers of new technologies, and seem to have little innovative impact on foreign

practice. O’Rourke and Nicholson both emphasize the restrictive impact of the unimaginative contractual framework and heavily litigious atmosphere in America, factors that are unlikely to change either favorably or quickly.^{2,1} There is also the fundamental question of necessity and, in this respect, the exploitation of new technologies is being forced on American engineers as they face contemporary challenges in several fields, as well as in urban engineering. Waste containment, dam rehabilitation and liquefaction control are equally demanding attention at the present time.

The debate on the reasons for the status of American engineering as net importer of new technologies need not be restated. Rather, the situation is accepted, and American engineers should, therefore, adopt the viewpoint that they are in a very privileged position. They can afford to be very selective and highly critical of these techniques with respect to satisfying current particular requirements — without having to endure the risk and expense of the development process.

Examples of the new technologies of particular relevance to urban construction are described under the following categories:

- *In Situ Reinforcement*: the use of grouted steel inserts, commonly called soil nails, to stabilize slopes and excavations.
- *Structural Support*: the use of small diameter cast-in-place “minipiles” to sustain axial loads.
- *Ground Treatment*: the use of new techniques and materials to grout soils to improve strength and/or decrease permeability.

However, advances in other techniques are also occurring: the American Society of Civil Engineers (ASCE) Committee on Placement and Improvement of Soils presented in April 1987 a ten-year update on an entire range of processes entitled “In Situ Reinforcement, Densification, Reinforcement of Constructed Earth, and Chemical Admixtures and Miscellaneous Methods.”³ Furthermore, the following new trends should have

widespread familiarity:

- in diaphragm walling (e.g., the development of powerful hydromill type excavating machines that cut easily and quickly through soil and rock, and minimize panel joint problems); and,
- in ground anchoring, where the use of post-grouting techniques permits substantial upgrading of safe anchor capacities in soils, and where refinements in corrosion protection are being assimilated for permanent anchors.⁴

In Situ Soil Reinforcement

The benefits of soil reinforcement in enhancing soil stability were appreciated by the Babylonians 5,000 years ago.⁵ Its use has been exploited since then by engineers employed by a variety of organizations from the Roman Army to the U.S. Government. In each case, construction has been from "bottom up" and has employed carefully selected materials for reinforcement, facing and fill.

In situ reinforcement, however, has a much shorter and less exotic history. First, it is essential to differentiate between *piles* (inclusions placed in the ground to support external, directly applied loads) and *in situ reinforcement* (inclusions placed to maintain equilibrium under the soil and surcharge loading). In situ reinforcement is thus usually placed from "top down," and into natural ground, the properties of which cannot be preset. The technique is proving increasingly popular in a wide range of applications for slope and excavation stability associated with deep foundations, tunneling or highway construction.

There are three basic classes of in situ reinforcement (see Figure 1):

- *Soil nailing* refers to reinforcing elements installed horizontally or subhorizontally into the cut face, as top down staged excavation proceeds. The inserts improve the shearing resistance of the soil by being forced to act in tension.
- *Reticulated micropiles* are similarly cast in

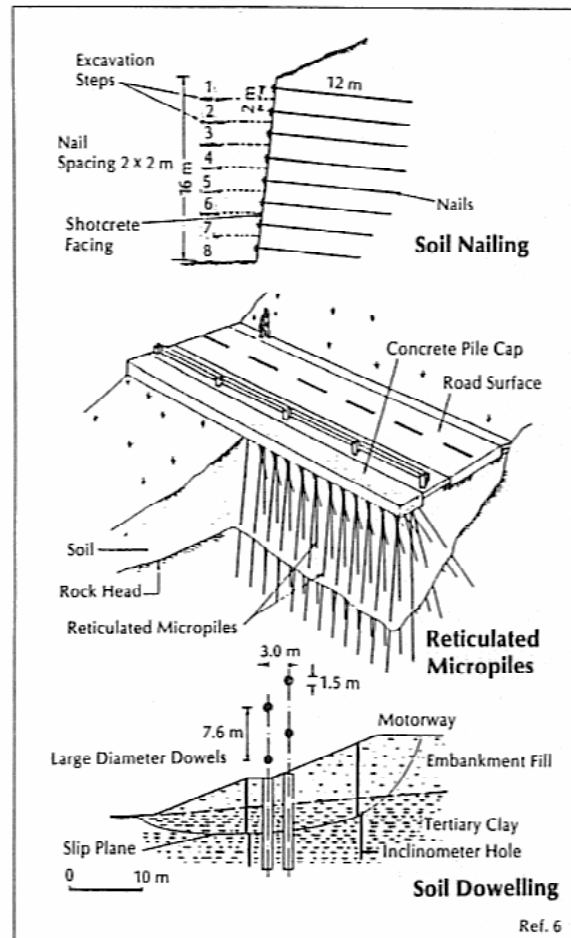


FIGURE 1. The family of in-situ soil reinforcement techniques.

place, small-diameter piles, steeply inclined in the soil at various angles, both perpendicular and parallel to the wall face. The overall aim is to provide a stable block of reinforced soil to act as a gravity retaining structure.

- *Soil dowelling* is applied to reduce or halt downslope movements on well defined shear surfaces. The principle exploits the large lateral surface bearing area and high bending stiffness of the dowels that are of far larger diameter than nails or micropiles that are seldom greater than 15 cm. The use of soil dowelling is rare in urban environments, although it can prove attractive when combined with linked deep drainage in arresting massive land movements (e.g., in eastern Italy and southern California).⁷

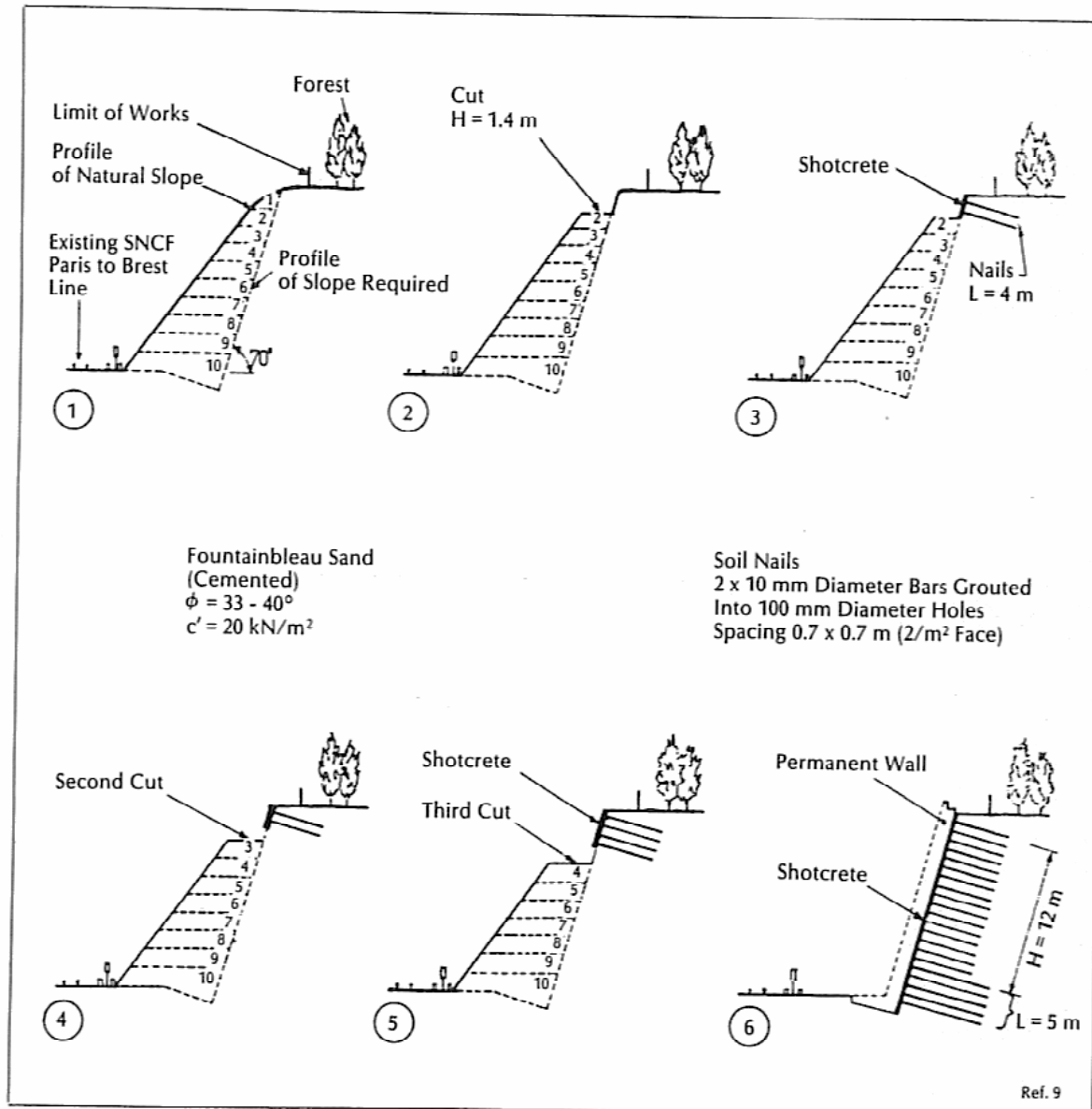


FIGURE 2. Soil nailing construction sequence as used at Versailles, France.

Soil nailing is currently the most popular option in urban construction, being one of the fastest growing geotechnical techniques in the country. Reticulated micropiling as in situ reinforcement is so far less common, but its success in rural applications is becoming more widely appreciated, and its potential for application in urban environments is considerable.

Soil Nailing

Evolution and Applications. Bruce and Jewell trace the growth in soil nailing in France

and Germany from 1972 onwards.⁶ With origins owed to *ancrages passifs* for mines and rock slopes on the one hand, and the techniques of the embryonic "New Austrian Tunnelling Method" on the other hand, the first published example of soil nailing was of a railway widening project near Versailles in 1972 (see Figure 2).⁸ About 13,000 square meters of cut-in cemented Fountainbleau sands were successfully stabilized, and the case history was well publicized (e.g., Hovart and Rami⁹). Within a few years, the technique was being applied routinely in sub-

way, highway and railway construction in France and West Germany especially, although it was not until an international conference in Paris in 1979 that the practitioners in each country seemed to have begun to actively exchange data, particularly on design methods.

The growth of the technique in France is reflected in the execution of several model mathematical and full-scale trials under the national research program, "Clouterre." In addition, there is a very active postgraduate research element at several universities such as L'Ecole Nationale des Travaux Publics de l'Etat in Lyon.

In West Germany, developments have been led by a specialist contractor in association with the Institut für Bodenmechanik und Felsmechanik (IBF) of the University of Karlsruhe, with financial support from the Federal Ministry of Research and Technology.¹⁰ In a four-year program that began in 1975, eight carefully instrumented large-scale field trials were conducted and analyzed. Model testing and theoretical research into soil nailing is also conducted at IBF.

Elsewhere in the world, and outside America, Banyai reported ten applications in Hungary up to 1984, while nailing has also been used in Mexico, Spain and Belgium.¹¹ More recently, first examples in South Africa and New Zealand have just been conducted, perhaps encouraged by the publication of review papers in the international press.^{12,13}

The evolution in North America is a fascinating story, retarded by a dearth of published data and a lack of communication for its first ten years. The concept of soil nailing was applied in Western Canada in the late 1960s, while the first American application appears to have been to support a deep excavation for the Interstate 95 Inner Loop Highway in Washington, D.C., in the early 1970s. The first published account, however, remains that of an article in the *Engineering News-Record* in 1976 that described the excavation support system for a 14 m deep basement in Portland, Oregon.¹⁴

A research team at the University of California at Davis monitored this project and thereafter conducted centrifuge model

testing, an instrumented full-scale trial and finite element analyses.¹⁵ In a report on this project in 1981, Shen *et al.* made no reference to any experience or methods outside of North America.

Although nailing was conducted on the West Coast from the late 1970s, it was not until the successful execution of the foundations for the PPG Building in Pittsburgh in 1982, and its subsequent widespread publicizing that interest became nationwide.¹⁶ This increase in interest was reflected in the publishing of the National Cooperative Highway Research Program (NCHRP) Report 290 in 1987 on all types of in situ reinforcement and in the attention paid to the topic at the ASCE Atlantic City Ten-Year Update Symposium.^{5,3} On the experimental side, FHWA research contracts were let in 1985 for additional fundamental laboratory and field tests, leading to the issue of a formal design manual next year.

Soil nailing has had almost 20 years of successful application in various parts of the world in temporary and permanent structures, in rural and urban settings and in a wide variety of soils. Bruce and Jewell summarized construction details of published case histories.⁶ Juran and Elias provide a closer examination of some of these examples.¹⁷ These syntheses record the various construction parameters used (e.g., nail spacing, nail length, diameter and inclination, facing characteristics, etc.).

Aspects of Construction. The standard sequence of construction is illustrated in Figure 2. Details are provided in comprehensive case histories by Louis, Nicholson and Boley, and in the description of a recent Nashville contract that follows.^{18,16}

With regard to excavation and temporary facing, soil must be prevented from running into the excavation during the face cutting process, since the effects may cause potentially dangerous settlements to adjacent structures. Therefore, if the in situ soil has been judged to possess inadequate "stand-up" capacity or time (such as provided by natural cementation or cohesion), then positive steps must be taken prior to excavation. For example, the French have used compo-

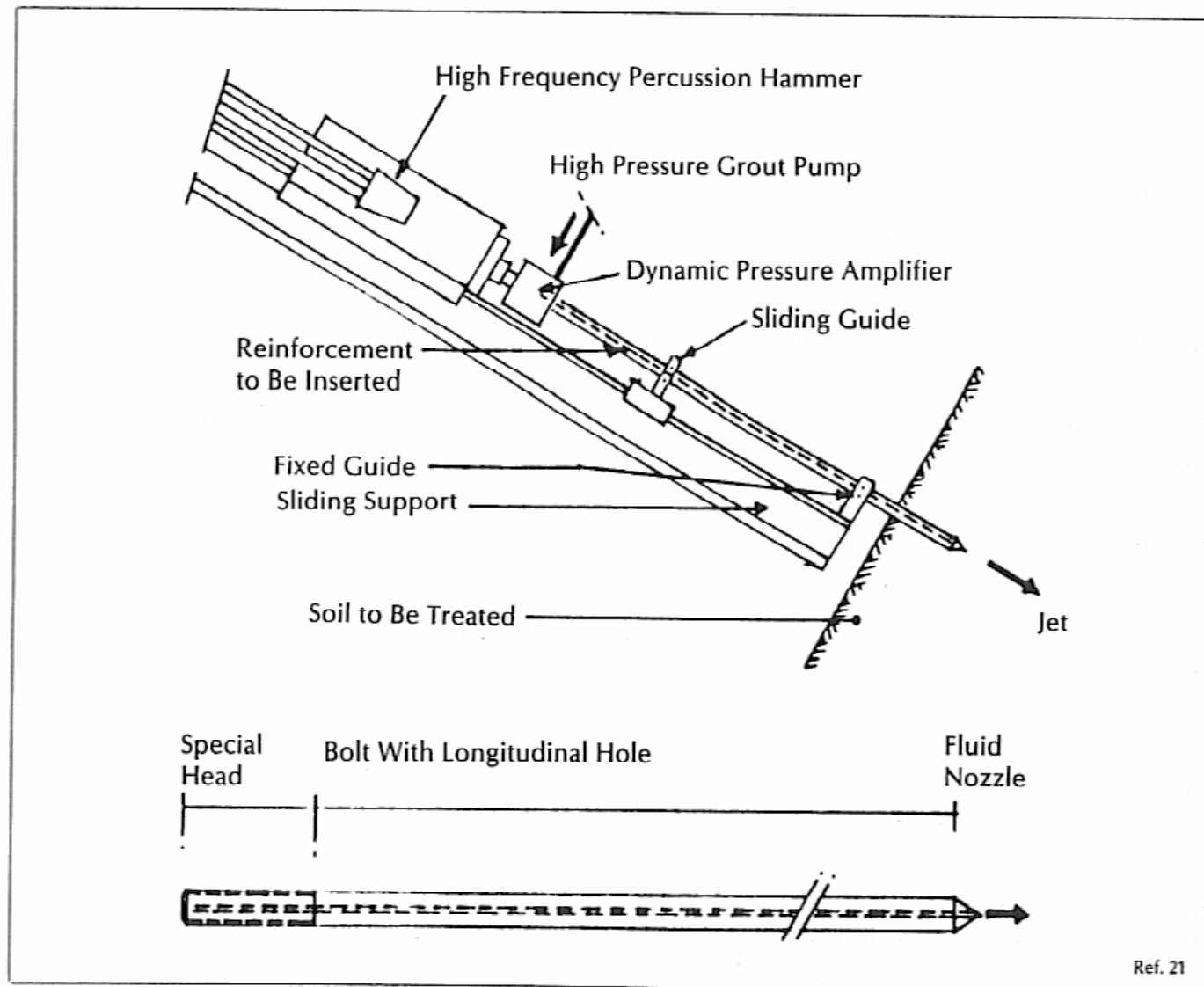


FIGURE 3. The "jet nailing" technique for soil nail installation that combines vibropercussion and high pressure grout jetting at the nail tip.

site walls wherein the upper, looser sediments have been retained by a shallow low-strength diaphragm wall, simply and cheaply constructed by backhoe. In the U.S., the major projects for the PPG Building in Pittsburgh and for the St. Thomas Hospital in Nashville both featured pretreatment of the excavation face by vertical grout holes that were closely spaced and lightly reinforced.

In conventional applications where early face support by shotcrete is adequate, the use of silica fume and fiber reinforced concretes may be considered, since they offer the advantages of higher, earlier strength and superior performance, often with a reduction in the volume of shotcrete required and associated shorter spraying times. For permanent structures where an exposed shotcrete surface is acceptable, the

appearance can be made more attractive or more sympathetic to the natural environment by the addition of special pigments or aggregates to the final layer.

The use of shallow face drains (e.g., 10-cm diameter PVC tubes about 30 cm long) or deeper soil drains into and beyond the retained mass is standard practice. An interesting alternative was used recently at Cumberland Gap, Kentucky, where vertical geotextile band drains were fixed to the face at regular horizontal distances prior to shotcreting.¹⁹ As each successive cut was made, each drain was connected downwards, so that by the final cut, the slope was being drained by full height continuous band drains, feeding ultimately into a base collector. As in all such face drainage systems, the intensity of such drainage must be chosen

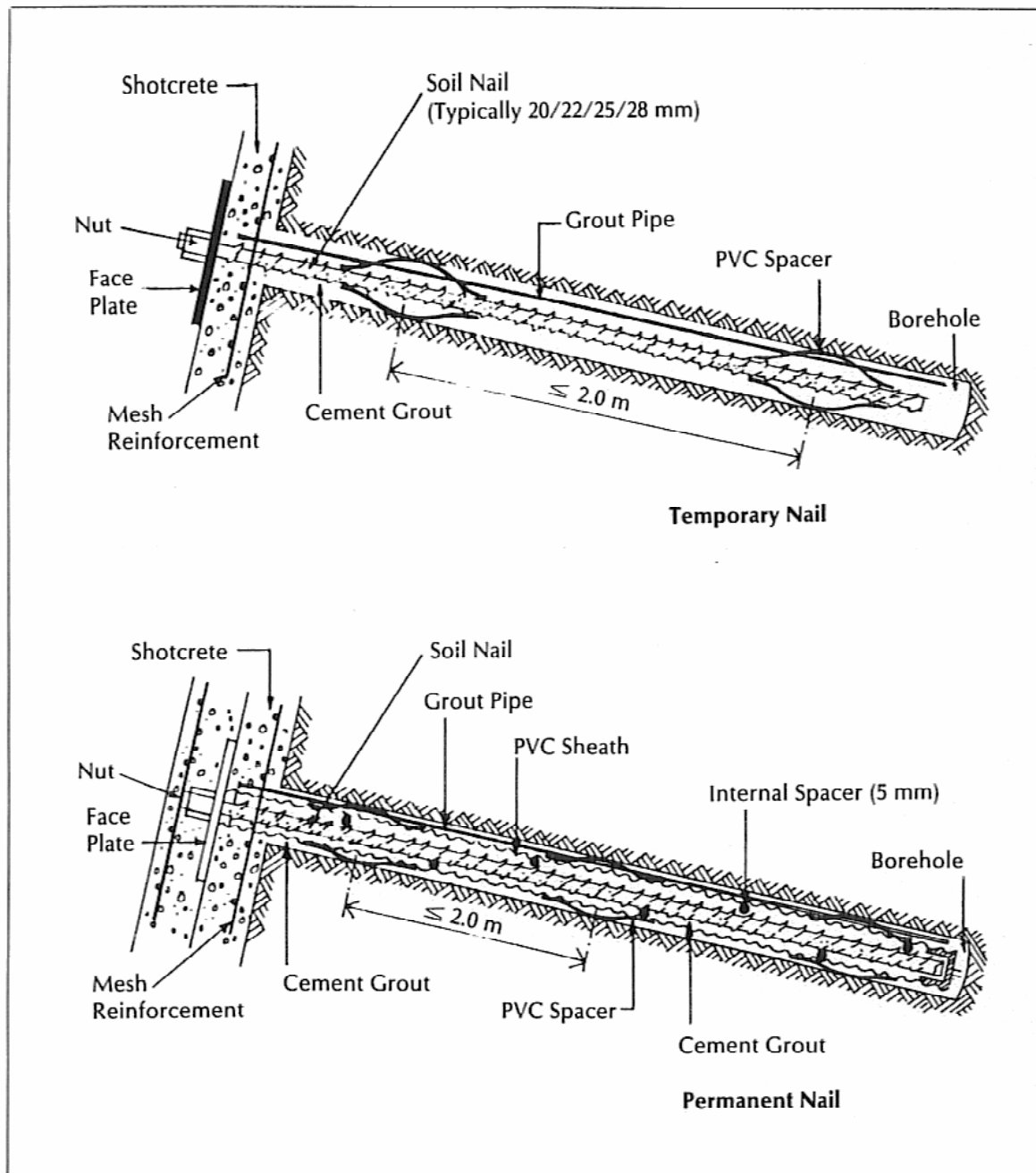


FIGURE 4. Examples of drilled and grouted nail installation for temporary and permanent applications (based on West German experience).

to reflect the threat of damage by frost or ice action as well as the usual hydraulic considerations.

In many cases, the soils are sufficiently competent to allow bore holes to remain stable enough after drilling to permit homing of the steel element and grouting. Otherwise, temporary drill casing or hollow stem augers

are used. Generally, good practice should follow the Post Tensioning Institute's (PTI) *Recommendations for Prestressed Rock and Soil Anchors*.²⁰ New European techniques of "jet bolting" (see Figure 3) claim excellent potential both economically and technically, but have yet to prove their reliability and applicability in typical field conditions.

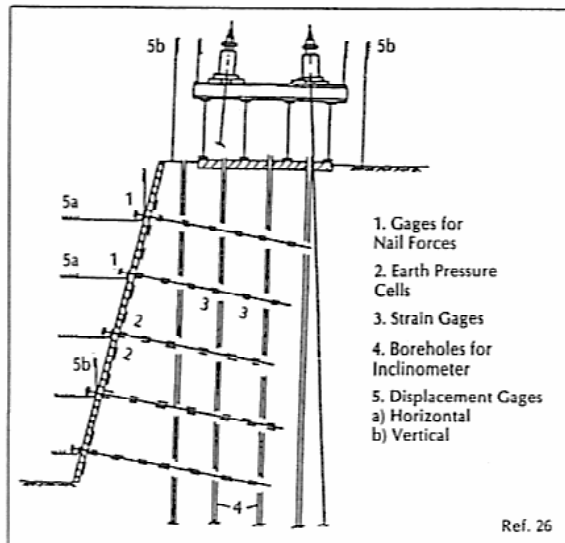


FIGURE 5. Range of instrumentation used for monitoring the performance of soil nails.

Most nailing applications have been temporary, and in any event have been geographically far removed from particularly aggressive conditions such as marine environments. Thus, corrosion protection systems have been basic — relying on the bonding grout as a protection often supplemented, as in the U.S., by epoxy coating to the bar surface. However, there is an increasing number of permanent applications and there is concern regarding the corrosion-related failures of certain reinforced earth applications. In light of the findings of recent reports (e.g., from the FHWA and also the Fédération Internationale de la Précontrainte (FIP) state-of-the-art review, "Corrosion and Corrosion Protection of Prestressed Ground Anchorages"^{22,4}), increased attention will be paid to this question regarding soil nails. Thus, the general rule is to have at least one layer of sheathing provided to the bar, as is the case for permanent nails in Germany (see Figure 4). Typically, grouted nails consist of high-strength steel bars 1.5 to 2.5 cm in diameter and with a yield stress of 1,050 N/mm² (150 ksi).

The use of soil nailing is increasing in permanent applications where a shotcrete finish — no matter how carefully integrated or disguised — is not visually acceptable. Slope stabilization in scenic areas of the

French Riviera or western California are typical examples. In such circumstances, precast element facings or reinforced concrete masonry blockwork are proving excellent choices, permitting the landscaper or architect free reign. Elements are being used that incorporate vegetation troughs, or that provide particularly good durability or noise absorption properties.

Aspects of Design and Performance. The design of soil nailed structures has been reviewed most recently by an ASCE Committee and by Juran and Elias (both at the 1987 Atlantic City Symposium), and in NCHRP Report 290.^{3,17,5} Their design is complex, and with respect to certain aspects such as the overall slope stability analysis, there seems to be no "best" method, although the "kinematical" limit analysis proposed by Juran and Beech appears consistent with observed structural performance.²³ Other common limit equilibrium analyses are the Davis method and the French modified slices methods of Schlosser, wherein the shear capacity and the bending stiffness of the nails are also considered in addition to their tensile capacity.^{15,24} The observed behavior of nailed cut slopes is fundamentally similar

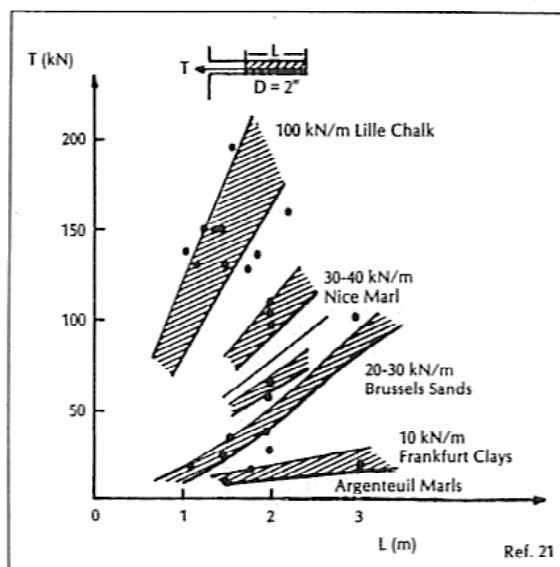


FIGURE 6. Variation of pull-out resistance of reinforcing elements with the length of embedment (cement or resin grout) for different soils.

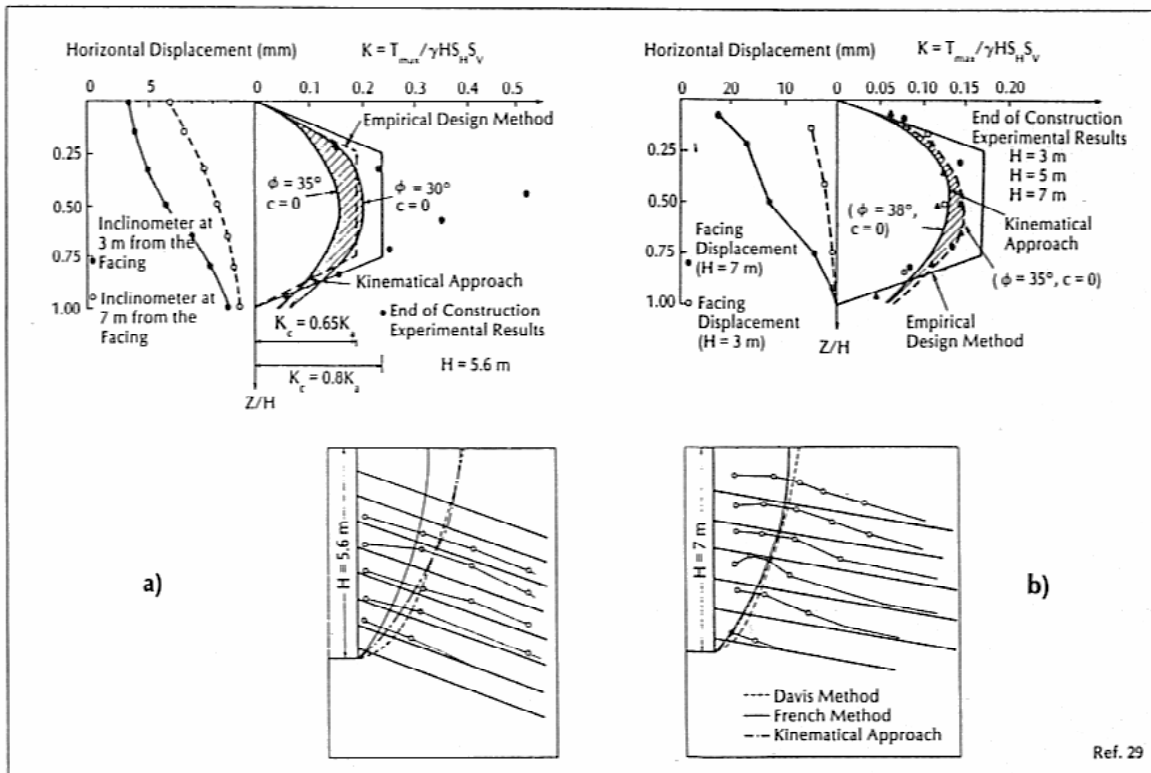


FIGURE 7. Data from: a) Parisian wall — experimental data and theoretical predictions of tension forces (upper), and predicted and observed locus of maximum tension forces (below); and, b) CEBTP experimental wall.

to that of braced open cuts, thus providing a preliminary basis for the development of semi-empirical design diagrams for predicting tensile forces in the reinforcements.

Four failure criteria must be considered in the general stability analysis approach:

- shear resistance of the soil;
- soil to reinforcement friction — the mobilized tensile force must be balanced by the effective friction mobilized at the soil reinforcement interface;
- normal interaction between the soil and the inclusion resulting in a progressive mobilization of passive earth pressure along the reinforcement; and,
- strength of the reinforcement — their tensile and shearing resistances must not be exceeded.

Much of the earlier experimental data have now been published, especially those from West Germany (e.g., Stocker *et al.*, and

Gassler and Gudehus).^{25,26} In addition, much valuable field information has also been made available from closely instrumented commercial projects (e.g., Goulesco & Medio, and Cartier and Gigan).^{27,28} Data have been recorded both during construction and in service, and are generated as shown in Figure 5 by:

- load cells placed under nail heads to monitor nail loads;
- earth pressure cells placed under the facing to monitor earth pressures transmitted to the facing;
- strain gauges to measure the magnitude, distribution and development of bar stresses;
- borehole inclinometers to illustrate the displacements in the supported ground at various distances back from the face; and
- displacement gauges to permit survey of the face.

In addition, pull-out tests on individual bars are an essential prerequisite to the start of full-scale production works on every site, and much data of this type have been published (see Figure 6).

These instrumented structures were constructed in different types of non-plastic soils using a variety of installation techniques. Despite this variance, the experimental results and field observations yielded consistent data regarding system behavior, structural displacements and variations of maximum tensile forces with the depth of the excavation (see Figure 7). Juran and Elias concluded that post-construction observations in non-plastic soils have shown that after the end of the construction, ground movement and facing displacements do not usually exceed 0.3 percent of the total excavation depth and are rapidly stabilized.¹⁷ The post-construction creep displacements, local dislocations in residual soils and volumetric strains induced by environmental loads (climatic changes and freezing effects) can cause a significant increase of tensile forces in the inclusions.

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

- Ground anchorages are stressed after installation so that in service they ideally prevent any structural movement from occurring. In contrast, soil nails are not prestressed (except for a small seating load to provide proper mating of the nail/shotcrete/soil interface) and require a finite (albeit very small) soil deformation to cause them to work.
- Nails are in contact with the ground over most of their length (typically 3 to 10 m), whereas ground anchorages transfer loads only along the distal, fixed

anchorage length. A direct consequence of this load-bearing feature is that the distribution of stresses in the retained mass is different for each type.

- Since nails are installed at a higher density (typically 1 per 1 to 4 sq. meters of face), the consequences of a single unit failure are not necessarily so severe. In addition, the construction tolerances during installation need not be so high given their overall interactive mode of operation.

- Since high loads have to be applied to anchorages, appropriate bearing facilities must be provided at the head to eliminate the possibility of "punching" through the face of the retained structure. Substantial head bearing arrangements are not necessary with nails whose low individual head loadings are easily accommodated on small steel bearing plates placed on the shotcreted surface.

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

- In general, if the overall stability calculations show the problem to be deep seated, then ground anchorages will most probably be required, either as the only retaining element or compositely with the soil nailed structure.

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

- Economic advantages: where nailing is possible technically, it is typical to find that the cost savings for excavations on the order of 10 m deep are 10 to 30 percent, relative to the use of an anchored diaphragm or Berlin wall. These projections of savings are supported by the reported savings of 30 percent on a soil nailed excavation in

Portland, Oregon, in 1976.¹⁴

- Construction equipment: drilling rigs for reinforcement installation and guns for shotcrete application are mobile, quiet and relatively small in size. Their use is highly advantageous in urban environments, where noise, vibration or access may pose limits on the type of equipment that can be used.

- Construction flexibility: soil nailing can proceed rapidly and the excavation can be shaped easily. It is a flexible technique, readily accommodating variations in soil conditions and work programs as excavation progresses.

- Performance: field measurements indicate that the overall movements required to mobilize the reinforcement forces are surprisingly small. Furthermore, nailing is applied at the earliest possible time after excavation, and in intimate contact with the cut soil surface, thus minimizing any disturbance to the ground and the possibility of damage being caused to adjacent structures. This "early support" also allows the natural undisturbed properties of the soil to be exploited to advantage. As demonstrated by Gassler and Gudehus, nailed structures can withstand both static and dynamic surcharge loading without excessive settlements if properly designed.²⁶

Naturally, the technique has certain practical limitations to its application, and these are primarily as follows:

- Soil nail construction requires the formation of cuts generally 1 to 2 m high in the soil. These cuts must then be capable of standing up unsupported for at least a few hours, prior to shotcreting and nailing. The soil must, therefore, have some natural degree of "cohesion" or cementing. Otherwise, a pretreatment such as grouting may be necessary to stabilize the ground immediately behind the face.

- A dewatered face in the excavation is desirable for soil nailing. If the groundwater tries to percolate through the face,

the unreinforced soil will slump locally on initial excavation making it impossible to establish a satisfactory shotcrete skin.

- Excavations in soft clay are also unsuited to stabilization by soil nailing. The low bond resistance possible in soft clay would require a very high density of in situ reinforcement of considerable length to ensure adequate levels of stability, while creep will also affect performance. Bored or jet grouted piles or diaphragm walls with anchorages are more suited to these conditions.

Case History. The construction of the new Ambulatory Services Building, St. Thomas Hospital, in downtown Nashville, Tennessee, involved an excavation as deep as 15 m. About 1,400 m² of vertical face stabilization was required in two phases — the first being 160 m long and 15 m high, the second being a 39 m length adjacent to an existing structure and having a maximum supported height of 4.5 m (North Wall).

The bedrock is described as Ordovician limestone dense-medium, crystalline, but variably shaley and heavily fractured. It contains zones of yellow brown soft to hard silty clayey soil and decomposed rock. In addition, there is a major occurrence of dense grey silt and reconstituted clayey fault gouge with occasional large, fresh limestone boulders in a wide zone striking obliquely across the site. Ground level varied from 497 to 500 feet with the piezometric level at 450 to 460 feet (*c.f.*, the excavation level was generally 451 feet). The bedrock surface varied greatly, as is common in such terrains. The soil to be nailed was very variable, but typically had the following range of properties:

- SPT — Very variable, large zones 10 to 15 blows, some as low as 3 to 5, others up to 30 to 40 (bearing on rock fragments)
- γ_w — 126 to 130 pcf
- γ_d — 103 to 108 pcf
- LL — 41 to 64 percent
- PI — 20 to 42 percent
- Moisture Content — about 20 percent
- ϕ — 21 to 29°

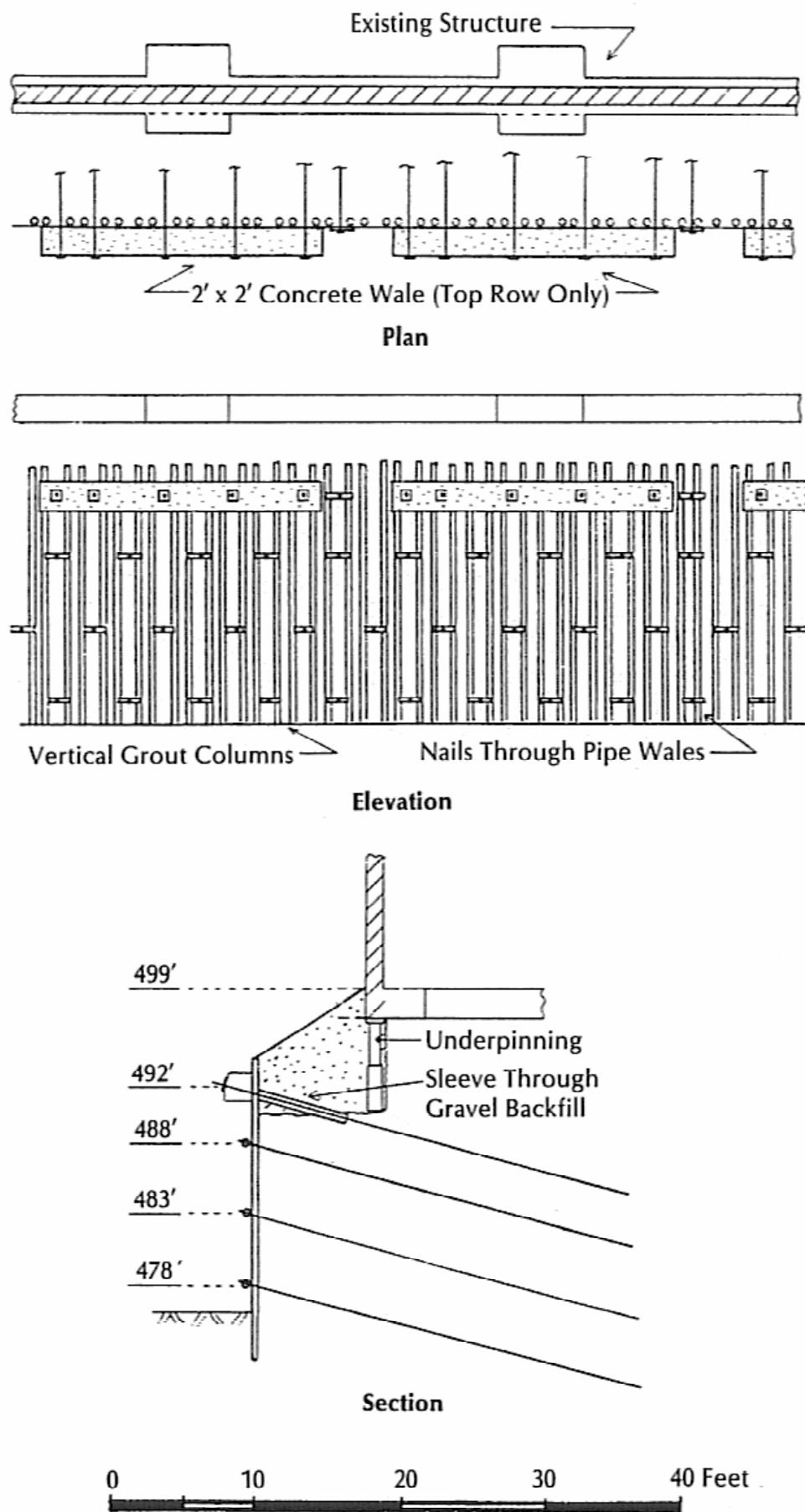


FIGURE 8. Details of soil nailing at St. Thomas Hospital, Nashville, Tennessee (North Wall).

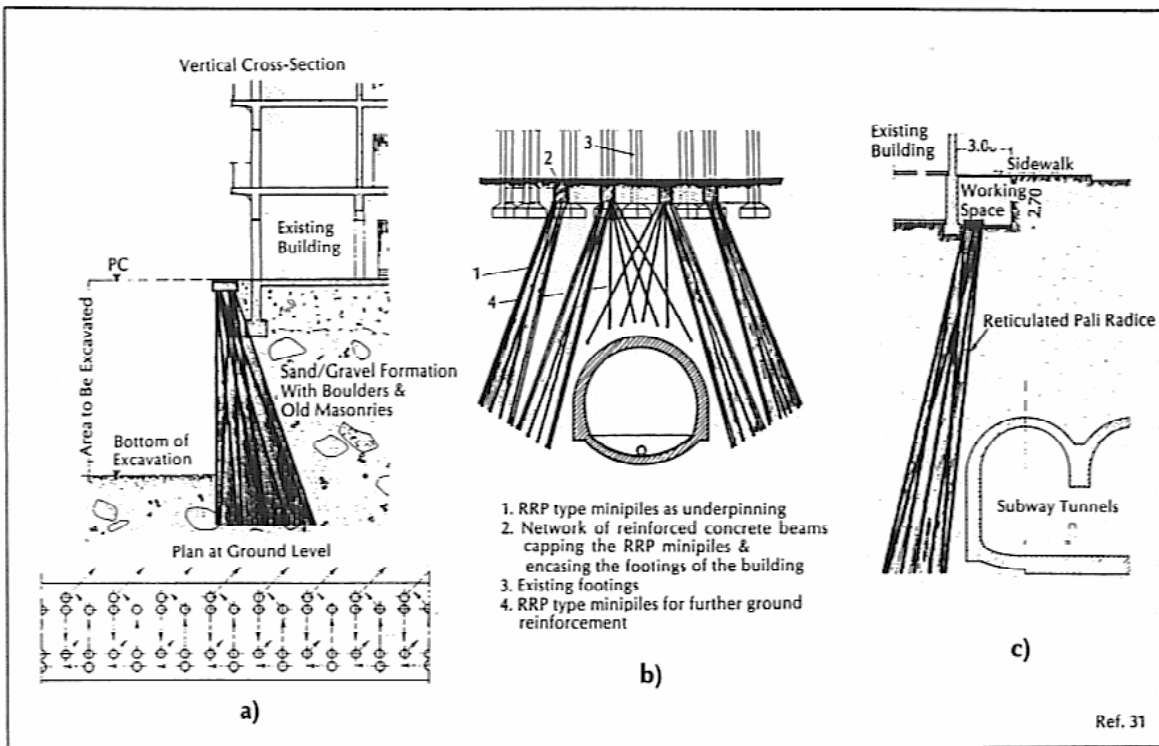


FIGURE 9. Applications of reticulated micropiles as used for in situ reinforcement: a) for cut and cover excavation; and, b) & c) around bored tunnels.

- C — 180 to 220 psf
- V. Ratio — 0.55 to 0.62
- Saturation — 94 to 100 percent
- S.g. — 2.69

This very variable nature of the ground virtually ruled out the construction of conventional retaining walls involving bored or driven piles or slurry trenches. On the other hand, this variability posed no major construction problems to the drilling and grouting activities of soil nailing. In addition, it was foreseen that some kind of face pretreatment would be prudent prior to excavation, especially since the effectiveness of shotcrete under these conditions could be questionable. Thus, the face support was provided by closely spaced vertical, lightly reinforced grout columns, installed as a precursor to excavation and nailing (see Figure 8). These holes, installed to a level 1 m below the base of the excavation also provided an accurate assessment of rock head profile that was extremely useful in planning the later excavation works.

The design was based on the modified limit equilibrium analysis, adapted from Shen *et al.* and appropriately expanded.¹⁵ The geology was acknowledged as being structurally complex, but the design assumed a generalized profile of homogeneous soil (over competent rock), the properties of which were taken as:

- Unit weight — 125 pcf
- Frictional strength — 21°
- Cohesive strength — 550 psf
- Ultimate pull-out capacity — 1,200 psf clay; and, 2,000 psf rock

The analysis method factored all input by 1.5 to provide a sufficient degree of conservatism. For the nail spacing adopted, maximum nail loads of 5 to 9 tons were predicted, depending on the wall height.

Diesel hydraulic track rigs were used to drill the 16.5 cm diameter vertical grout holes through the soil and rock. A No. 6 bar was grouted into each with a 1:1 sand-cement grout (20 N/mm² at 28 days), and

Table 1

Summary of Reticulated Micropile Wall Case Histories (U.S. Applications)

Project Name & Location	Construction Date	Ground Conditions	Upslope/ Downslope Grade (U/D)	Depth to Slip- Surface at Wall (ft)	Cap Beam Geometry LxWxH (ft)	Pile Data	
						Diameter (in.)	Reinforcement Type & Grade
Forest Highway 7, Mendocino National Forest California	1977	Micaceous phillite $\phi = 13^\circ$, $c = 500$ psf & schistose bedrock	2H:1V/2H	55	310x6x2.5	5	#9 Rebar Grade 60
Route 23A Catskill State Pk., New York	1977	Moist very dense glacial till with boulders $\phi = 30^\circ$, $c = 0$ & green & red shale	2H:1V/ 1H:1.3V	10-26	250x11x1.75	4 (min.)	#10 Rebar Grade 60
PA-306 Monessen, Pennsylvania	1979	Random fill colluvium & weak red shales $\phi = 17^\circ$, $c = 100$ psf $\gamma = 134$ pcf	2.5H:1V/ 4H:1V	20	200x6x2.5	5	#9 Rebar Grade 60
L.R. 69 Armstrong County, Pennsylvania	1985	Random fill underlain by wet sandy clay & a hard clay residual of bedrock $\phi = 30^\circ$, $c = 0$ $\gamma = 120$ pcf	Level/ 1.2H:1V	34	310x6x2.5	6	#11 & #14 Rebar Grade 60

the upper 1 to 2 m of each hole was protected against mechanical damage by 10-cm (outside diameter) plastic pipe. Twin primary holes (0.5 m center to center) were located at 1.5 m intervals, followed by intermediate pairs of secondary holes, thus giving an intensity of 4 holes per 1.5 m of wall perimeter.

The 11-cm diameter nail holes were up to 12 m long, but were typically less than 11 m, spaced on a 1.2 x 1.5 m grid in the soils, but 1.5 x 1.5 m in the rock. No. 8 bars (Rows 1 to 6) or No. 11 bars (Row 7 and higher) were grouted with a Type 1 neat cement grout of $w = 0.45$. Each 8-cm square steel headplate bore on a short cross wale, fixed across pairs of vertical holes.

Regarding testing and monitoring, several pull-out tests were conducted to verify design assumptions on nail bond with the ground. Inclinoimeters were installed at four locations around the site and their data,

together with face surveys, were recorded throughout and after excavation. A total of 14 nails were strain gauged. All the data confirmed the expected performance of the nails and structure, and certainly conformed with the trends as described above. A detailed report on this site is currently under preparation for presentation late this year.³⁰

Reticulated Micropiling

Evolution and Applications. The technique of micropiling was first conceived, and patented, in Italy in 1952. The early applications were as direct underpinning, and often involved closely interlocking arrays of piles in which a positive "group effect" was achieved. Later, the system was applied as in situ reinforcing (see Figure 9) to solve slope stability problems, typically in rural areas. Such examples in the U.S. are listed in Table 1, while Lizzi provides numerous

Average Number of Piles per Foot Length of Wall	Orientation (Degrees from Vertical) Upslope	Maximum Length (ft)	Depth Below Slip-Surface (ft)	Effective Wall Width along Slip-Surface (ft)	Measured Lateral Movement (in.)	Comments
2.25	19 0 -19	69 63 69	8	41.5	Not measured	Strain gage data indicate mostly compressive strains except near the pile cap
2.8	15 -15	40 50	30 24	33	Before cap 4 After cap 0.3	Designed as a space frame, the cap beam was constructed after the piles were finished
2.25	19 -19	45	9	20	After completion 0.1 - 0.75	Slope downhill of wall failed after completion, no significant movement of piles was noticed
1.33	40 -8	50	15	20	After completion 0.125	Piles oriented at 40° back from vertical were designed as tiebacks to satisfy rapid drawdown condition, section south of insert wall failed in 1987 but without effect on stabilized section

case histories of successful applications throughout Western Europe.³¹

With particular reference to urban engineering, the major applications are illustrated in Figure 10 for cut and cover as well as bored tunnel construction. In each case, the concept is to create protective structures in the ground in order to separate the foundation soil of the building from the zones that are potentially subject to disturbance. Attwood describes the application of this system in the widening of a major roadway near London, where the very restricted access at the top of the embankment (1.75 m from cutting to existing structure) ruled out "conventional" techniques (see Figure 11a).³² Where the structure had to act as a retaining wall for a bridge abutment (see Figure 11b), higher capacity tubular minipiles (40 tons WL) were incorporated into the vertical front face. Inclinator readings during and after construction showed a

maximum outward deflection of 1.5 cm for the 8 m high cut, with a profile resembling a typical anticipated cantilever performance.

Construction. The drilling and grouting equipment and systems are generally the same as those used for soil nail and ground anchor construction. Unlike nails, however, pressure grouting with neat cement or sand-cement grouts is usually conducted to pressures of up to 1.2 N/mm². The concrete capping beam merits particular attention during design and construction, although it can be completely buried after construction. There are indications that if the beam is cast *before* drilling is commenced, there is less movement of the reinforced mass during and after construction.

Design and Performance. The basic design concept is that the structure acts as a gravity wall, consisting of a monolithic mass. This mass relies for its continuity on soil-pile interaction and not on intergranular cemen-

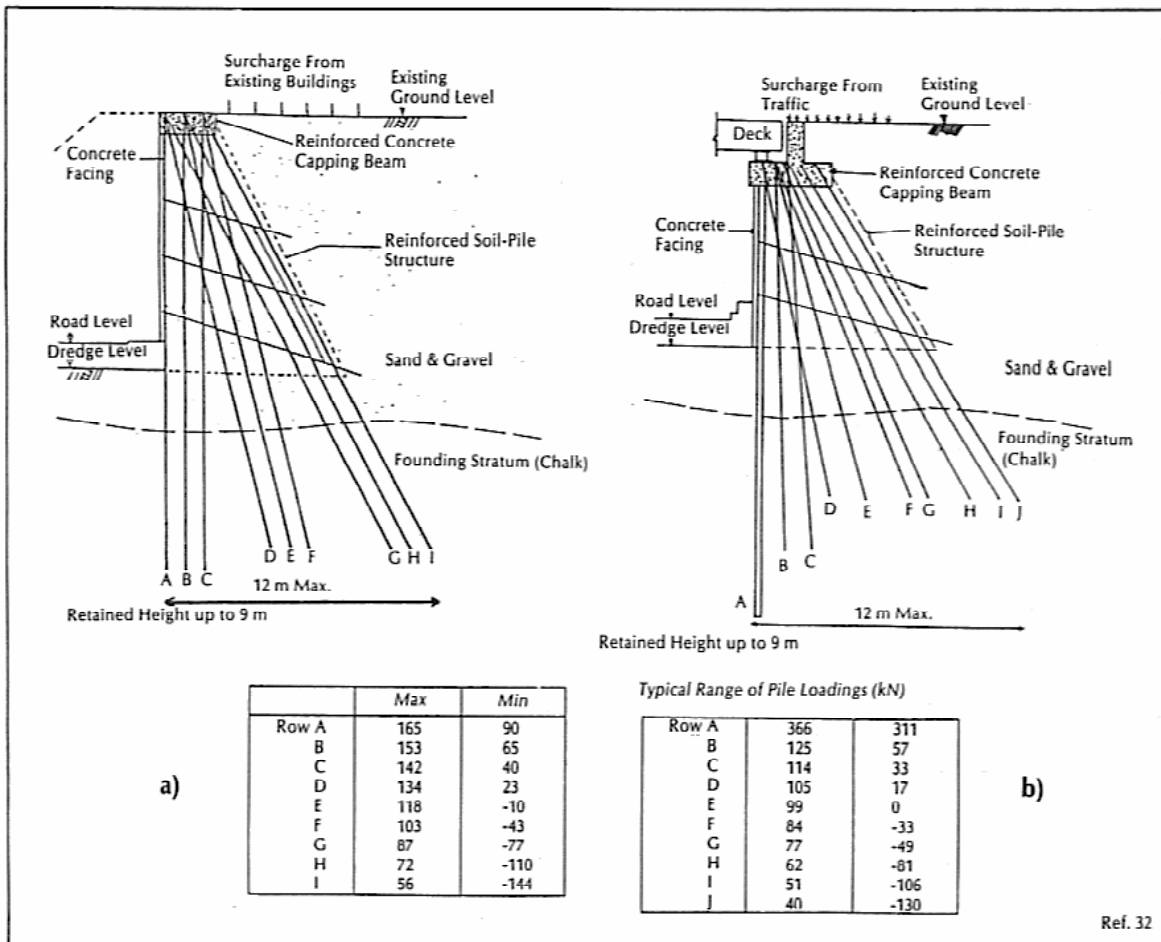


FIGURE 10. In situ reinforcement for road widening project, Dartford, London: a) normal retaining wall; b) retaining wall serving as bridge abutment.

tation. The soil itself supplies the gravity, while the piles supply the "lines of force" that allow the whole mass to support compression, tension and shear.³¹ The completed wall should thus physically prevent loss of soil from behind, and prevent sliding along potential failure planes.

Instrumented field programs have confirmed that reinforcement stresses and overall wall movements are minimal, and that most probably the designs to date have been highly conservative (see Table 1). This conservatism clearly reflects the current lack of a rigorous design approach. Lizzi confirmed "it is not yet possible to have at our disposal an exhaustive means of calculation ready to be applied with safety and completeness."³¹ Lizzi noted that his "paper does not suggest formulas or elaborate mathematical calcula-

tions because it cannot offer theoretical approaches in the usual way geotechnical problems are dealt with."³² In addition, the ASCE Committee also alluded to the great reliance placed in designs on the soil/pile interaction "which is still subject to experience and intuition."³

In brief, the design approach is to calculate the contribution offered by the piles to the resistance of the natural soil. The factor of safety, F' , is calculated as:

$$F' = (R + R')/A$$

where:

F' = Factor of safety

R = Soil shear resisting forces on the critical surface

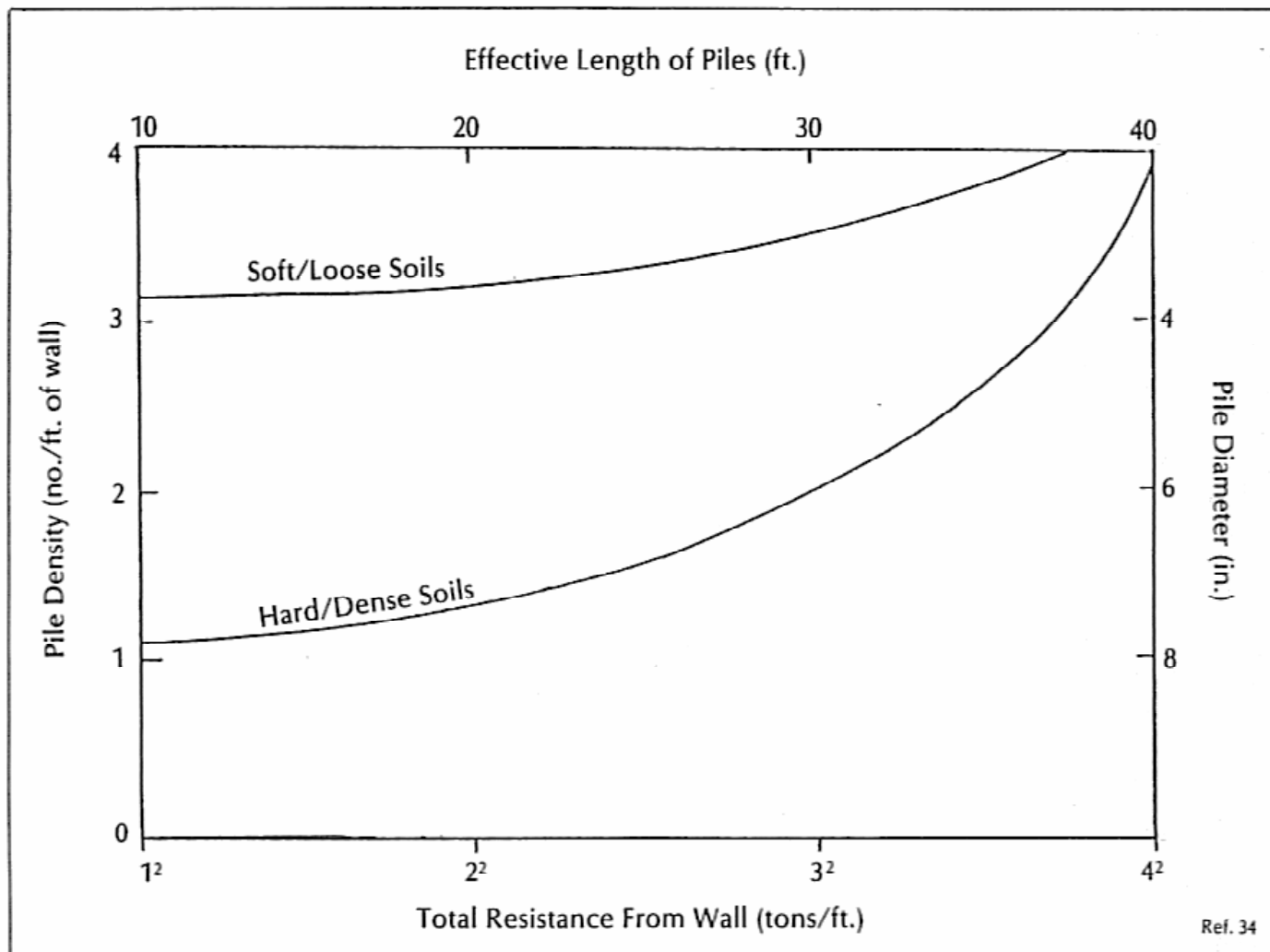


FIGURE 11. Chart for the preliminary design of reticulated micropiles.

R' = Additional shear resistance supplied by the piles

A = Total driving forces along the critical surface

R' is the allowable stress in the grout and reinforcement in the pile cross-section, or the shear resistance of the soil along a section of the pile mobilized by the sliding mass (whichever is less), multiplied by the number of piles on the critical surface. This approach is considered conservative, since it does not take into account the interaction developed between the soil and the pile.

As an example, the general design for the Mendocino wall involved:

- Estimating loads (active and passive) on the wall.
- Conducting a stability analysis to determine the shear force needed to maintain a required factor of safety.

- Determining the number of piles needed to provide the required shear resistance.

- Calculations (similar to those for a conventional gravity wall) to check stability against overturning, sliding and bearing failure at the base of the wall.

The recorded design procedure included extending the piles into rock where possible (and always below the failure plane). However, a valid comment on design was made by Dash, who reaffirmed that no checking is done to *ensure* that the reinforced mass really behaves like a composite material.³⁴ He conceded that it was "very difficult" and "controversial," reflecting on the density of 1.33 piles per foot (L.R. 69 on Table 1) with 2.25 at Monessen, Pennsylvania, used by different contractors. Based on "our present experience," he proposed a chart to aid in the selection of pile spacing and

diameter (see Figure 11).

In situ retaining structures, as shown in Figure 10, can be constructed in close proximity to existing buildings and in relatively tight access locations without the need to excavate and without causing any decompression of the foundation soil. Given their mode of construction, they can be installed in any type of ground, including boulders, old foundations or any other obstructions with no constructional limit placed on hole inclination or orientation.

There are two major restrictions that have limited the popularity of in situ retaining structures in the U.S. so far. First, as described above, there is no rigorous design methodology; and second, the intensity of the inserts installed has often impaired their cost effectiveness. However, fundamental research is underway to try to understand better the soil-structure phenomena. This research should lead logically to a more precise design approach, and to more economic solutions resulting from a reduction in the intensity of the reinforcement deemed necessary. These two consequences should result in a greater acceptability of the technique in the U.S.

Minipiling

Minipiling is now well into its fourth decade of application. Comprehensive reviews of design, construction and performance are readily available (e.g., Koreck, Weltman, and Bruce *et al.*^{35,36,37}), while a recent synthesis was provided by ASCE.³

Evolution and Applications. Small-diameter cast in place bored piles saw their first application in Naples in 1952. With the expiration of the original patents in 1972, their application became much more widespread, fostered by the upsurge in remedial and subterranean works in urban areas, and the mass of supportive test data available from (cheap) load test programs. Acting singly or with the benefit of the group effect, load-bearing minipiles have been used in many applications wherein the transfer of structural loads to more competent lower horizons has had to have been achieved with minimal environmental disturbance or

structural settlement.

The applications basically revolve around the prevention or arrest of structural settlements generated by adjacent underground excavations, changes in the groundwater level, changes to foundation loadings (by additions) or the imposition of machine vibrations to structures and foundations.

In Boston, a typical example was the recent support of the addition to the Dana Farber Cancer Institute where a total of 165 piles with a 75-ton working load were installed in a sub-basement having 3 m of headroom through the clay and into bedrock. Also in Boston, the Hynes Auditorium renovation and expansion required the installation of over 330 piles with up to 250-ton working loads.³⁸ Over half of the piles were installed with headroom as low as 10 to 14 ft. This project remains the largest single contract, in dollar terms, yet executed in the U.S. In Brooklyn, over 4,000 piles of 15- to 30-ton working load are currently being installed in the Coney Island Main Repair Facility of the New York Transit Authority with minimum disruption to shop operation.³⁹ In this case, the piles penetrate loose fill and swampy, peaty deposits before being founded at depths of 10 to 13 m in dense fine-medium sands. They support the floor slab and its upgraded live and dead loads.

Construction. The most common basic method of installing minipiles is shown in Figure 12. Variants using compressed air to pressurize the grout, or a vibrated mandrel (displacement pile), are described by ASCE, but are largely obsolescent.³ In addition, the "expanded base" pile and the Menard inflatable cylinder pile are likewise seldom seen nowadays.^{31,40}

Although certain authors try to differentiate minipiles or micropiles in terms of diameter,³⁶ it is perhaps more appropriate to regard all such piles that can be installed with conventional drilling and grouting equipment as minipiles. This definition normally limits diameter to about 25 cm for piles within the normal depths involved (less than 25 m).

The details of construction vary, but

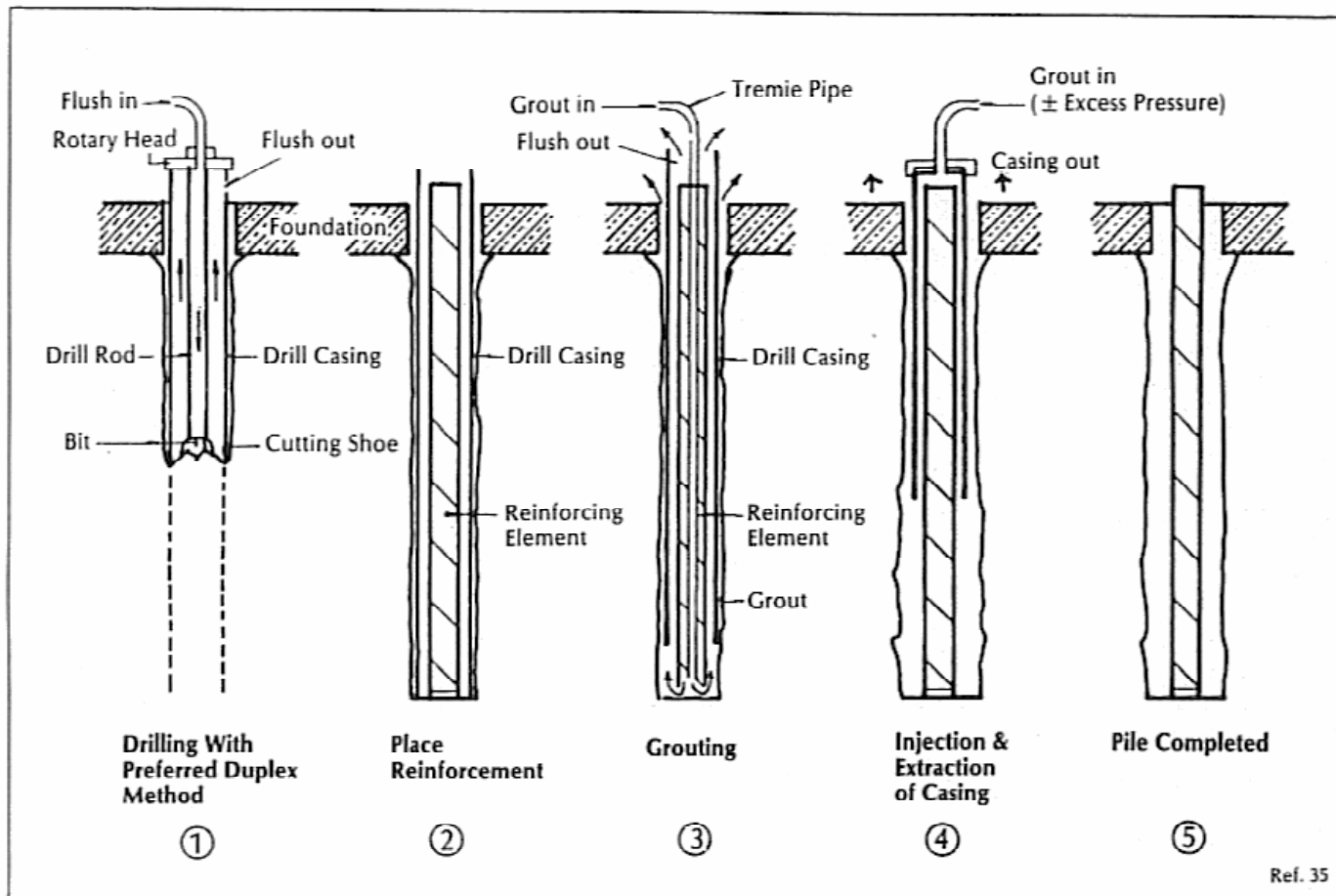


FIGURE 12. Stages in the construction of a standard minipile.

certain factors should remain common:

- **Drilling:** the method must cause minimal disturbance or upheaval to the structure or soil.
- **Grouting:** relatively high strength neat cement or sand-cement grouts are used at injection pressures rarely above 0.8 to 1.0 N/mm².
- **Reinforcement:** may be reinforcing cages (compressive loads only), high strength bars (compression or tension) or pipes (to resist bending stresses).
- **Connection to structure:** adequate bond or connection must be provided in order to properly transfer loads (see Figure 13). In the case where the load must be transferred solely by bond within the existing structure, the (normally smooth) structural interface can be roughened to give additional mechanical interlock in order to ensure adequate load transfer. Such a system, termed *Ankerbonder*, has recently been used on a major underpinning project for cooling towers in England.⁴¹

Employing vibrating air driven pistons with tungsten carbide tips, the head is lowered into the (diamond drilled) hole and rotated slowly. The typical groove configuration is shown in Figure 14. The resultant roughened interface gives ultimate structure/pile bond values up to ten times higher than conventional systems.

- **Corrosion protection:** when piles are required to act in tension, or when they are installed in particularly aggressive conditions, then particular attention must be paid to the corrosion protection of the steel element. Similar to ground anchorages, protection in the form of an outer corrugated sheath can be used, while it is prudent to centralize the steel in the hole to ensure that a minimum grout cover of 2 cm is provided to the steel.

Design and Performance. Minipiles are designed to operate by side friction as opposed to end bearing due to their geometry: they are very slender elements in

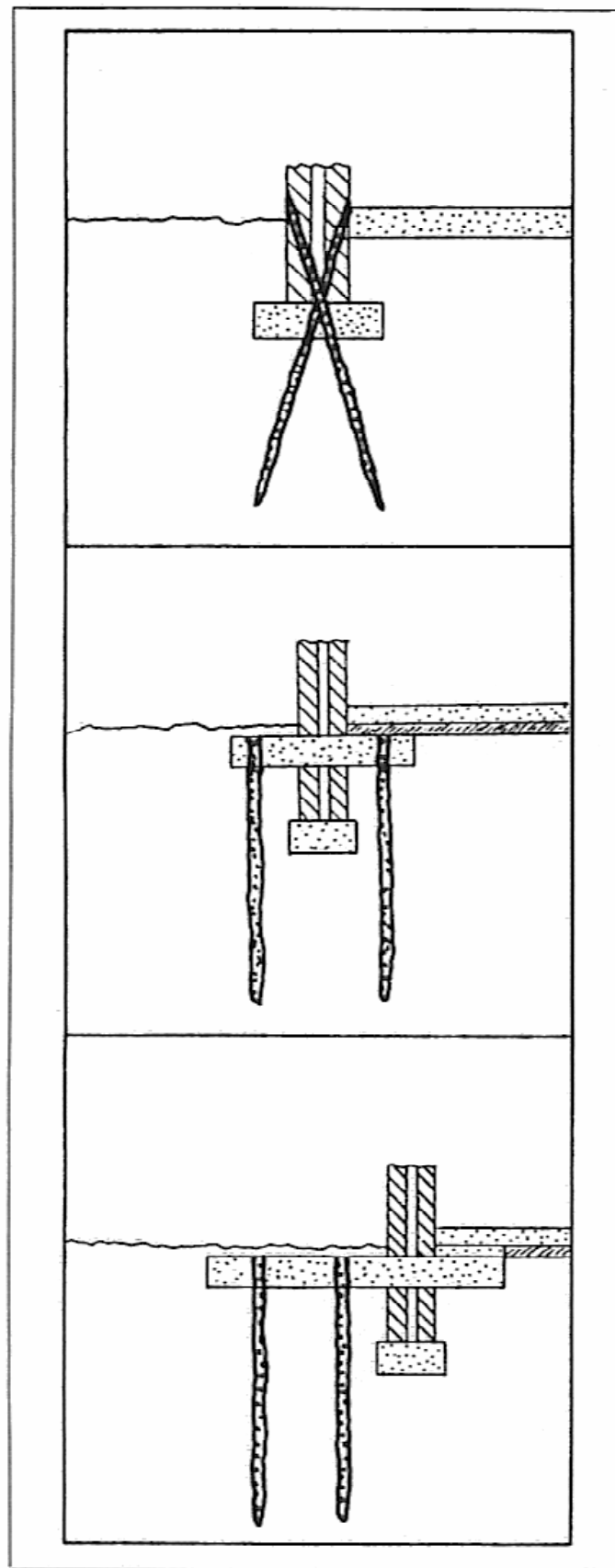
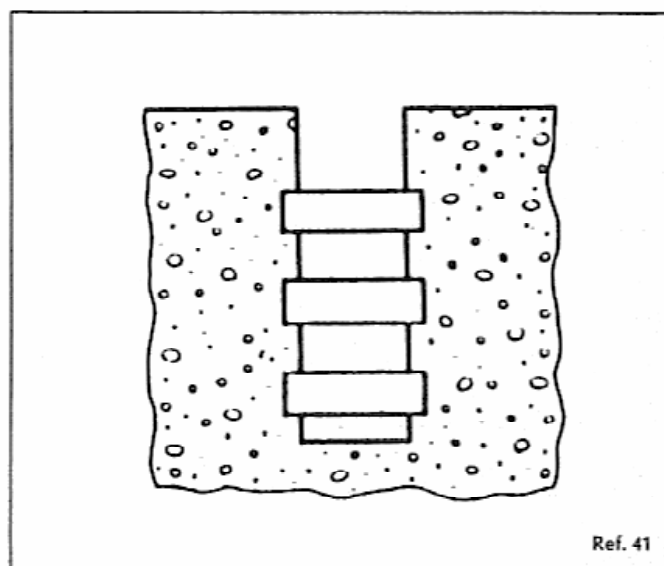


FIGURE 13. Basic methods of load transfer from minipiles to structure.

which the lateral area is typically hundreds of times larger than the base. Their geometry partly explains their excellent load/settlement characteristics: the relative displace-



Ref. 41

FIGURE 14. Typical triple-groove bonding profile produced by the *Ankerbonder* in a diamond cored hole. At the West Burton project, in England, two sets of triple grooves were provided per hole.

ment needed to mobilize frictional resistance are much smaller than those needed to develop end bearing.

In addition, the method of their construction, and in particular the use of high-strength grouts injected at significant pressures, acts to promote excellent bond characteristics with the soil. Analogies can be drawn with soil anchor practice (see Figure 15), albeit for interfaces in the opposite sense of shear. As a general guide to the design of the transfer length, the recommendations of PTI, pages 27-30, may be followed.²⁰

With these major points in mind, the basic design philosophy differs little from that for any other type of pile: the system must be capable of sustaining the anticipated loading requirements within acceptable settlement limits, and in such a fashion that the elements of that system are operating at safe stress levels. In detail, attention must be paid analytically to settlement, bursting, buckling, cracking and interface considerations; whereas, from a practical viewpoint, corrosion resistance, and compatibility with the existing ground and structure (during construction) must be regarded. The system must also be economically viable.

Reference must therefore always be made

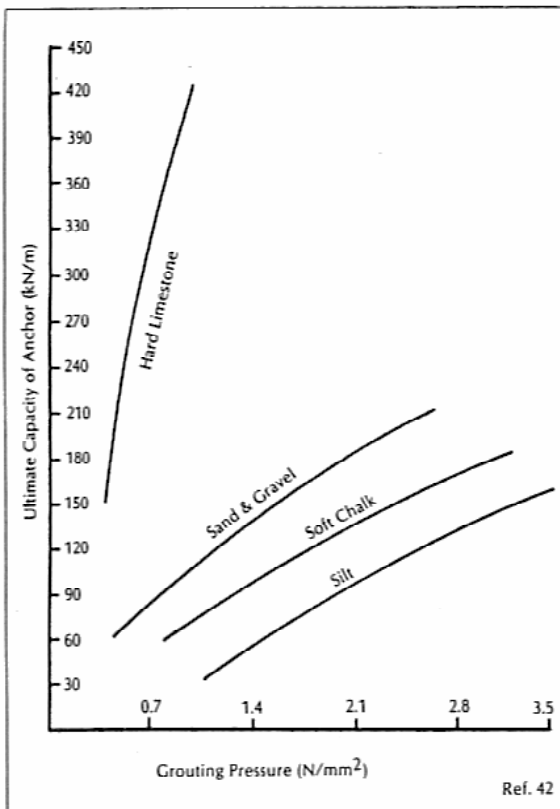


FIGURE 15. Influence of grouting pressure on ultimate load holding capacity.

to local construction regulations for guidance, although the special aspects of minimipiling may not be adequately or specifically addressed. In that event, sensible interpretation is necessary.

Generally, it is found that whereas the design of a conventional system is normally controlled by the external (i.e., ground related) carrying capacity, their small cross sectional area dictates that minimipile design is most often limited by the internal carrying capacity. Emphasis is therefore placed on the steel and grout parameters as well as the grout/steel bond.

Regarding the internal stability of minimipiles, mathematical models can be called upon to investigate the stability of minimipiles with respect to buckling and bursting resistances. Regarding the former, early work by Bjerrum is supported by the detailed analyses of Mascardi and Gouvenot.^{43,44,40,45} All authors conclude that only in soils of the very poorest mechanical properties, such

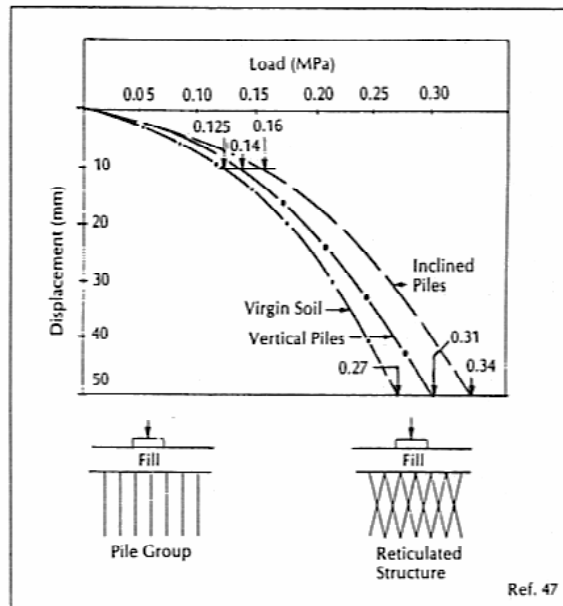


FIGURE 16. Field test data for different minimipile arrangements.

as loose silts, peat and non-consolidated clays, where E is less than 0.5 N/mm^2 there is even a possibility of failure through insufficient lateral restraint.

Similarly, bursting can typically be discounted, but where the possibility does exist, additional lateral restraint can be provided by increasing the thickness of the grout annulus, modifying the grouting design and methods, increasing the spiral reinforcement or by maintaining a permanent casing through dubious horizons.

A final point may be made in relation to the "group effect" — a vital consideration in minimipile design. The contrast with conventional piling is fundamental. For example, the *British Code of Practice 2004* states that for "friction piles, the spacing centre-to-centre should be not less than the perimeter of the pile; with piles deriving their resistance mainly from end bearing, the spacing centre-to-centre should be not less than twice the least width of the pile."⁴⁶ This spacing is to avoid the "negative" group effect.

On the other hand, Lizzi, and others, including ASCE, refer to the "knot effect" whereby a "positive" group effect is achieved in the loading of the soil-pile

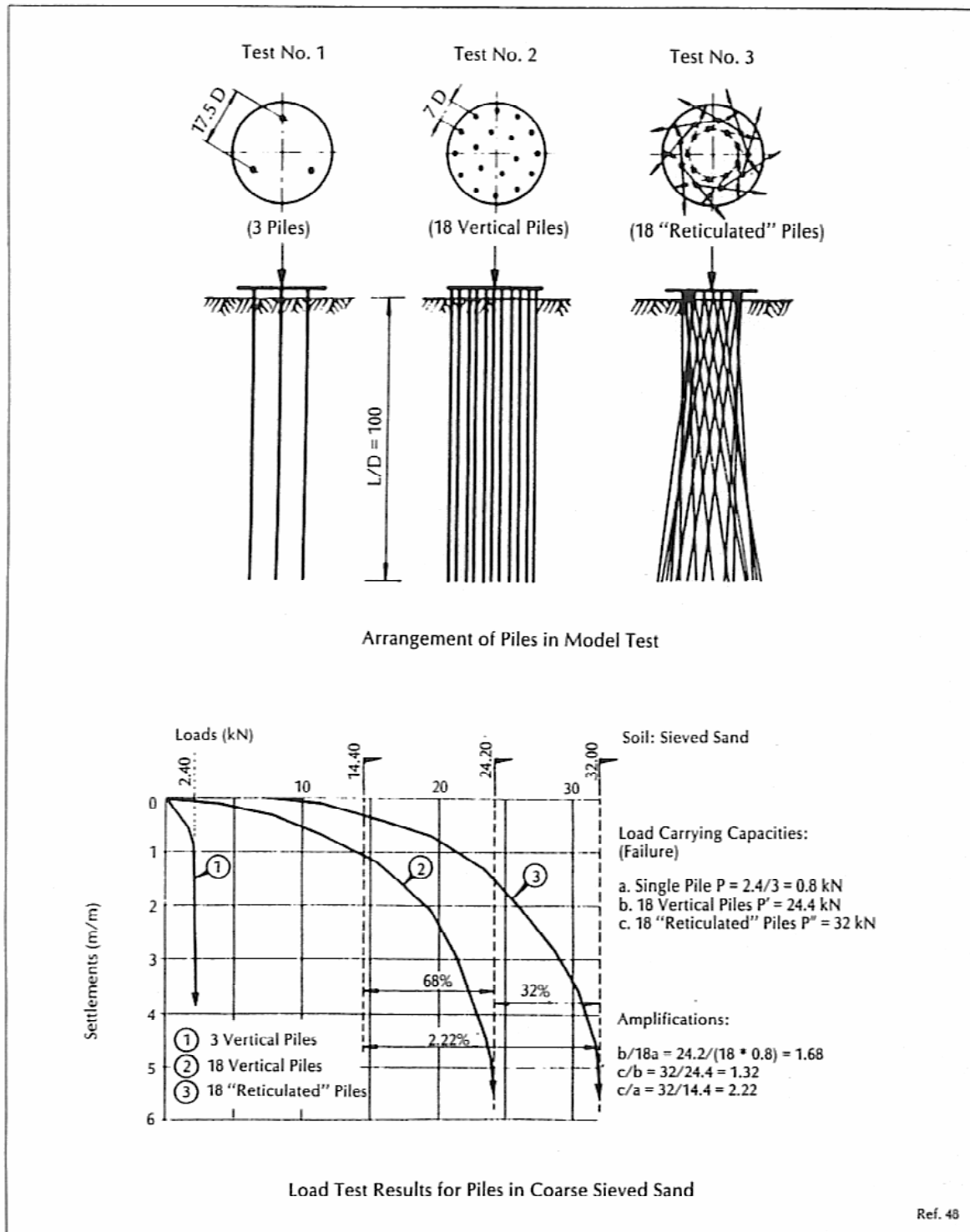


FIGURE 17. Model test data for different minipile arrangements in coarse sieved sand.

system.^{31,3} For example, Plumelle's full-scale testing yielded the results shown in Figure 16 that confirmed Lizzi's earlier model tests (see Figure 17).⁴⁷ The latter noted that the

increase was proportionally greater in the sand than the cohesive pozzolanic material that allowed interaction in even the Group A arrangement. This "knot effect" cannot

be relied on when preloaded piles are used.

Development Trends. In the last few years, as the market has increased and the number of geotechnical specialist contractors qualified to do the work has grown, several innovations have been made. These have been directed towards providing piles of superior performance more cheaply and in more challenging structural or environmental settings. Three groups merit special attention:

- Post-grouting of bond zone
- Reinforcement of free zone
- Preloading

By injecting cement grouts into the bond zone after the first stage grout has set, a significantly improved load bearing performance can be provided. The injection of cement grouts can be accomplished via a separate grouting tube (i.e., a sleeved pipe) or by using the steel reinforcement itself as the grout pipe. This method is used in the TUBFix and ROPRESS type piles, wherein the packer is introduced into the steel core pipe and grout is ejected through the rubber sleeved ports at regular intervals (see Figure 18). Post-grouting greatly improves the grout/soil bond, but in addition it may increase the nominal cross section, particularly in the weaker soil layers or near ground level where natural in-situ horizontal stresses are small. Pressures of up to 2 N/mm² are commonly used.

Mascardi noted that in cases of repeated post-grouting, an effective pile diameter in the range 30 to 80 cm may be expected, considering that standard minipile construction normally provides bond zone diameters significantly larger than the nominal drill diameter. In general, pressure grouting is most effective in improving pile capacity in ground where deformations can be imparted relatively quickly: sands and gravels, residual soils, shales, and some weaker sedimentary and low grade metamorphic formations. Jones and Turner also noted that there was a very favorable response to post-grouting in stiff clay.⁴⁹ No experience of good behavior in very soft non-consolidated

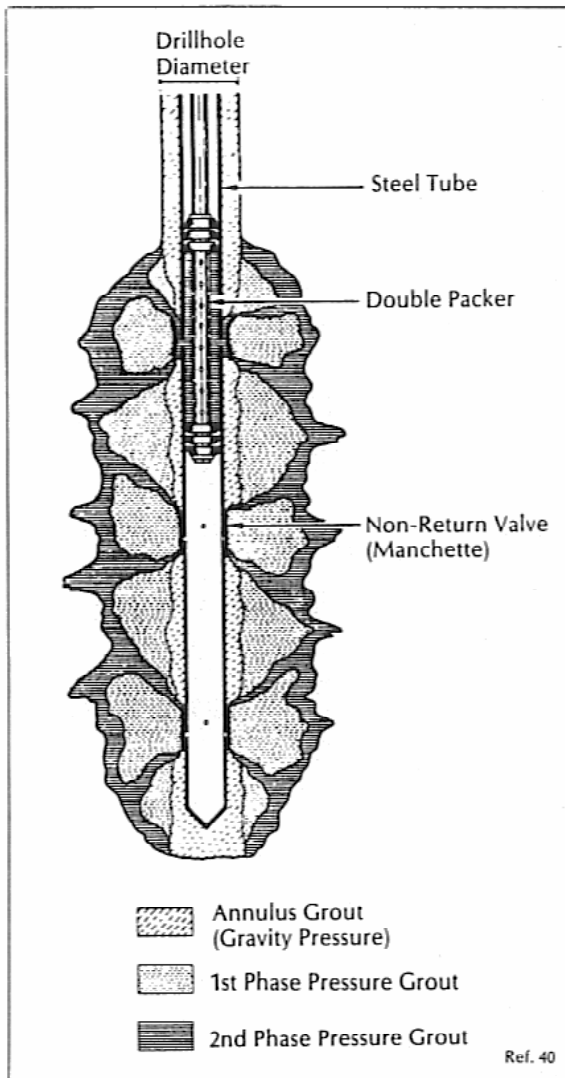


FIGURE 18. Concept of repeated post-grouting in increasing effective pile diameter.

clay or soft peat has been recorded.

Reinforcement of at least the upper part of the minipile to guard against buckling or bursting is common. However, this concept has been developed further in the "pinpile" design. In this case, the drill casing is pushed back down into the bond zone (3 to 6 m) after the completion of pressure grouting, and is then left in place fully to the surface. This method provides excellent corrosion protection, eliminates the possibility of upper layer pile structural failure, and prevents the wasteful travel of grout into often very permeable upper horizons. The load/settlement performance is also excep-

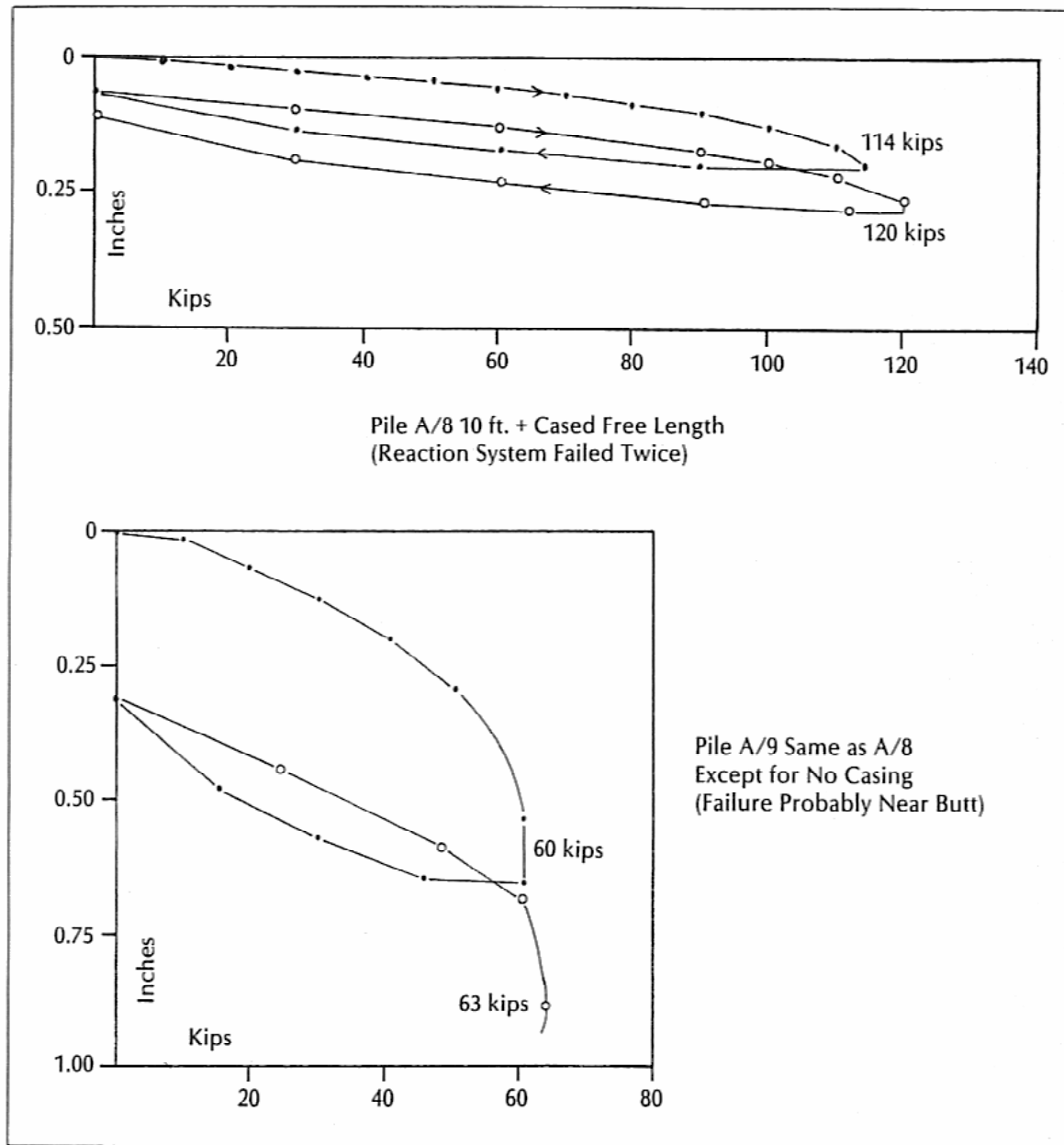


FIGURE 19. Minipile load test data from a site in Coney Island, New York, showing the benefits of a steel liner through upper strata above the bond zone.

tional, as illustrated in Figure 19, while additional resistance is automatically provided against lateral loading.

Minipiles furnish excellent load/settlement performance, with total deflections in normal cases being less than 1.2 cm at working load. However, there are cases where even this magnitude of movement is unacceptable to a particularly delicate structure. Preloading can then be used, wherein the pile has

the working load preapplied via prestressing methods that induce settlement of the pile.

Preloading can be accomplished in many ways. In the ROPRESS system, the pile is not bonded directly to the structure, but via the anchor pipe and screw. It is preloaded by a hydraulic jack, acting on the anchor pipe and locked off against the pipe when the desired load/deformation is achieved. An alternative system to preload pinpiles has

been used, achieving preloading by a strand anchorage founded below the pile tip. An additional benefit of preloading is that, as in ground anchors, each pile is routinely tested to at least its working load.

Benefits and Limitations. The technique of minipiling is especially valuable in conditions where the ground is very variable and "difficult," where access is restrictive and where environmental considerations are highly significant, especially relating to vibration. Minipiles can be installed in almost any direction and through any structure or soil, and in close proximity to existing buildings. They can sustain extremely high loads relative to their diameter at exceptionally low deformations and can be installed so as to underpin structures with no settlement via preloading. Compressive, tensile and axial loadings can be accepted.

In most soil conditions, working loads of up to 50 to 60 tons can be generated safely, with far higher individual capacities recently recorded when founded in rock.⁵⁰ For very heavily loaded structures (e.g., bridge piers), groups of minipiles are therefore required. Even here, however, it would seem that the resulting group effect is positive, as opposed to demanding reduced individual loadings.

Testing is relatively cheap, and by using adjacent piles as reaction, need not involve massive test frames and deadweights. There are a great deal of data available on minipile performance in all ground types.

The main limitation is cost — lineal costs are far greater than driven piles, for example. Nevertheless, circumstances often conspire, especially in urban construction, to make minipiling the only viable method of positive underpinned support.

Case History. The existing derelict six- and three-story commercial buildings at 739 and 749 Boylston Street, respectively, in the Back Bay area of Boston were scheduled for redevelopment and refurbishing. The structures dated from 1902 and were founded on pile caps bearing on driven wooden piles. To accommodate the increased loadings resulting from the new construction, additional piles were required under enlarged pile caps (Figure 20). The owner accepted

an alternate bid involving the installation of 260 minipiles with a working load 40 tons in compression and 12 tons in tension. The access, headroom and environmental restraints ruled out the use of driven wooden piles, while the overall reconstruction program demanded a short piling program.

The piles served to transfer load through saturated fill (loose fine sand, trace silt), and organic deposits (soft silt with traces of shells, fine sand and fine gravel) into the bearing horizon — a medium dense to dense fine-coarse sand with traces of gravel and silt. The upper soft deposits were typically 3 to 4 m thick, while the sand, overlying the Boston Blue Clay varied from 6 to 8 m in thickness. In order not to penetrate into the clay, the maximum length of the piles was 8.4 m.

The piles were designed to have an ultimate load of 2.3 times the working load (i.e., 92 tons in compression and 27 tons in tension). Two test piles were installed and tested to verify the design assumptions, especially the adequacy of the 4.6-m bond length of the anticipated grouted diameter of 18 to 20 cm. Each pile was equipped with a steel casing from the surface to within 2.3 m of its tip to ensure proper performance even in the very soft upper materials, and a full-length centralized steel rebar. Pressure grouting up to 0.7 N/mm² with neat cement grout was conducted during the withdrawal of the temporary drill casing in the sand horizon. A telltale anchored near the tip of the pile was also installed in each test pile, which was also debonded from the upper horizons by an outer steel liner.

Reaction for each test was provided by adjacent ground anchors, and the tests were executed in accordance with the recently proposed modifications to the Massachusetts State Building Code and ASTM D1143.^{51,52} The data are summarized in Table 2, while the performance of test pile no. 2 (in compression) is shown in Figure 21 together with that of a timber pile for comparison.

The elastic (recoverable) settlement at 80 tons was about half the total deflection, while no indication of pile or soil failure

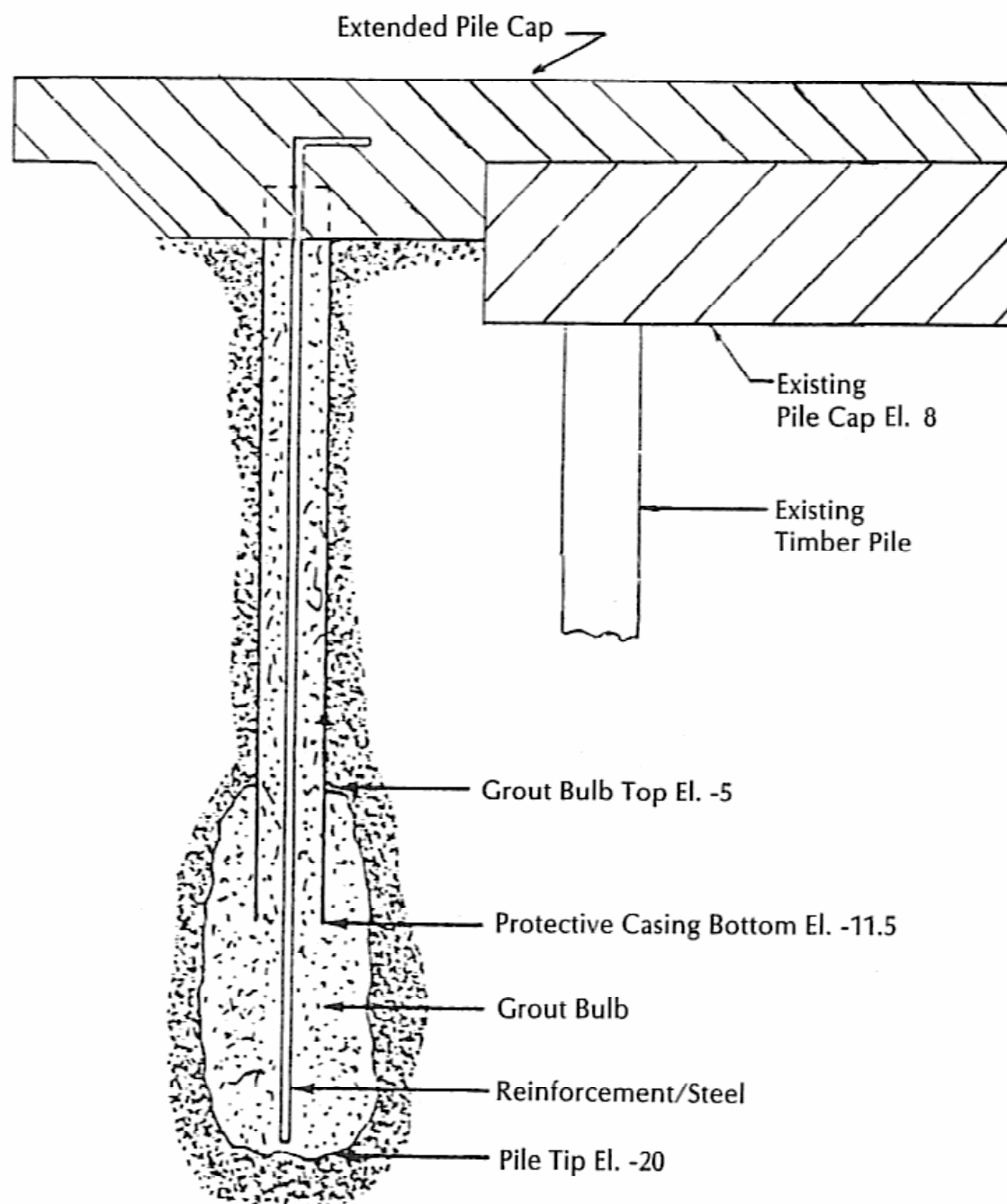


FIGURE 20. General arrangement of minimipiles, Boylston Street, Boston.

was evident from the butt or tip displacement curves. Furthermore, the net butt settlements were well below the recommended state building code criteria for maximum net settlements. The performance in tension was equally satisfactory.

Production piles were therefore installed in the same way. Particular care was taken to ensure that the pressure grouted zone was correctly located in the sand. Timber and other obstructions were recorded in a limited number of holes, and were accom-

modated by the drilling method or (in three cases) by relocation. Other potential problems arising from access, and difficult site conditions were also easily dealt with. In total, 262 piles were installed in about 5 working weeks, substantially ahead of the anticipated program.

Ground Treatment

A distinction should be made between ground treatment and ground improvement. Ground treatment relates only to injec-

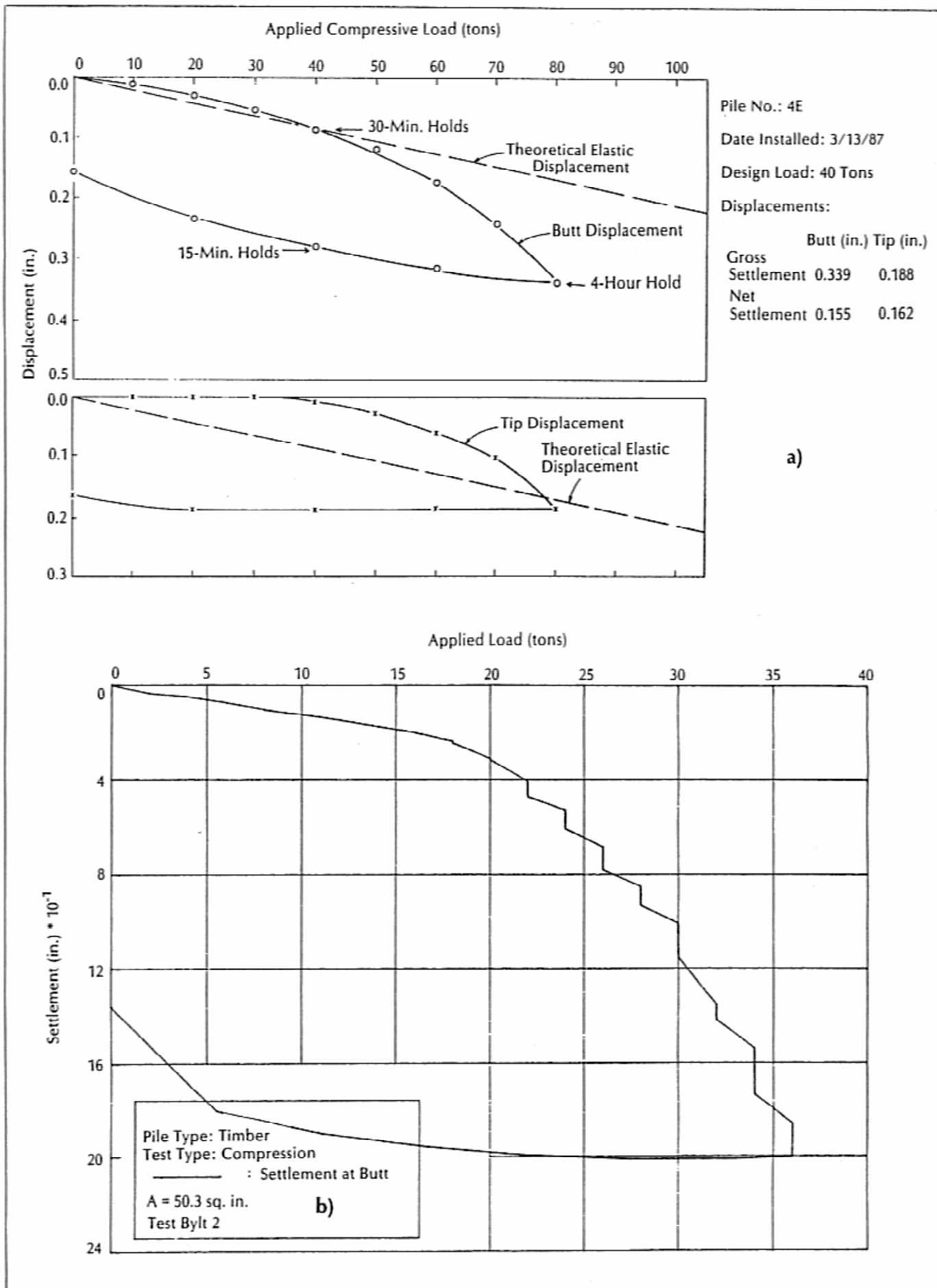


FIGURE 21. Load-settlement performance of: a) a drilled and grouted minipile; b) a driven timber pile; Boylston Street, Boston.

Table 2

Test Pile Performance Data

	Butt (cm)		Tip (cm)	
	Test Pile 1	Test Pile 2	Test Pile 1	Test Pile 2
<i>Compression Test</i>				
Gross Settlement	1.120	0.861	0.800	0.478
Net Settlement (Permanent)	0.640	0.394	0.638	0.411
<i>Tension Test</i>				
Gross Heave	0.605	0.358	0.442	0.165
Net Heave (Permanent)	0.419	0.221	0.391	0.165

tion related processes (grouting) that are designed to make the ground less permeable and stronger. Ground improvement encompasses a variety of techniques including vibroflotation and compaction, dynamic consolidation and wick drains, and its prime purpose is to cause densification and accelerate consolidation.

The methods of rock grouting are well known and comprehensively documented (e.g., Ewart⁵³). Newer developments are generally associated with refining materials and techniques, and improving on-site monitoring procedures. Thus, microfine cements are gaining popularity in cases where very fine fissures have to be grouted, while the Multiple Packet Sleeved Pipe (MPSP) system has proved an excellent tool when treating unstable rock masses with poor mechanical properties.⁵⁴ This system combats the problems of collapsing ground by permitting injections through a sleeved port pipe secured in the borehole by cement-filled fabric collars.

Soil grouting, however, is in a far more dynamic situation, benefiting from the technological advances made by chemists, physicists and geotechnical engineers on the one hand, and prompted by the increasingly severe demands made by structural engineers, environmentalists and property devel-

opers on the other.

Many of the new developments in soil grouting have been associated with tunnelling in urban areas, principally for subway or sewer projects. In most cases, such developments have been designed as an *aid* to tunnelling contractors in order to speed progress, improve safety and minimize associated settlements. Such has been the progress that grouting is now established not only as a final remedial option when "conventional" techniques have failed, but as the "design tool, as it should be from the onset."⁵⁵

The following new developments have been selected to highlight:

- major improvements in the properties of grouting materials;
- special techniques and systems to combat the problems of soft ground tunnelling;
- the advantages of integrating such techniques as routine construction methods; and
- the high degree of operational control that can now be applied by an experienced and efficient grouting contractor.

Categories of Ground Treatment. Overviews of ground treatment conventionally identify

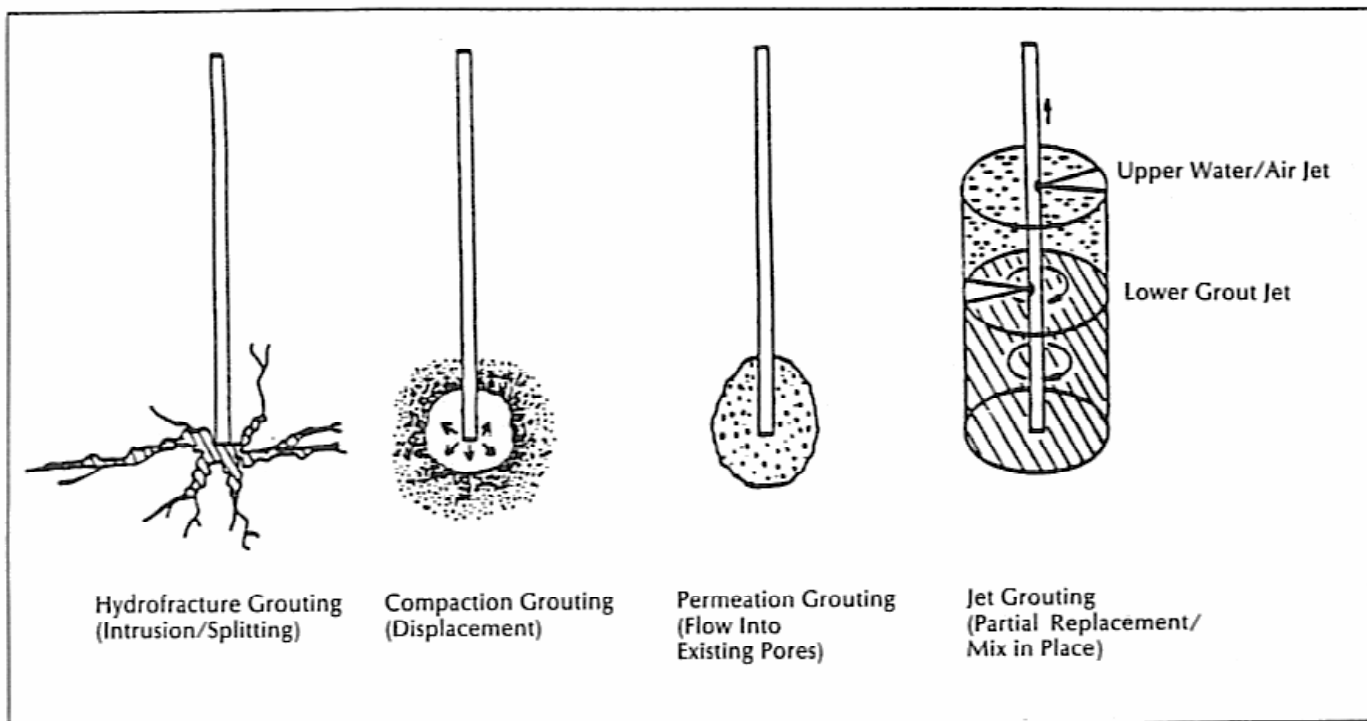


FIGURE 22. Basic categories of soil grouting.

four basic categories of soil grouting (see Figure 22):

- hydrofracture
- compaction
- permeation
- replacement

In *hydrofracture grouting*, the ground is deliberately split by injecting stable but fluid cement-based grouts at high pressures (for example, up to 4 N/mm^2). The lenses and sheets of grout so formed increase total stress, fill unconnected voids, possibly consolidating the soil under injection pressure, and conceptually constitute mainly horizontal impermeable barriers. However, it is typically very difficult to control, and the potential danger of damaging adjacent structures by the use of high pressures often proves prohibitive. It is not common to find this technique alone deliberately exploited outside the French grouting industry, although some hydrofracture phenomena accompany most permeation grouting contracts either accidentally or in conjunction. Tornaghi *et al.* note that hydrofracture naturally occurs with conventional cement-based grouts in soils with a permeability of less than 10^{-1} cm/sec .⁵⁶

Compaction grouting is a specialized "uniquely American" process that has been used since the early 1950s.⁵⁷ Very stiff soil-cement mortar is injected at high pressures (up to 35 N/mm^2) at discrete locations to compress and increase the density of soft, loose or disturbed soil. Unlike the case of hydrofracture grouting, the grout forms a very dense and coherent bulb that does not extend far from the point of injection. Near-surface injections result in the lifting of the ground surface (the technique of slab jacking as described, for example, by Bruce and Joyce⁵⁸) and, indeed, the earlier applications were used exclusively for levelling slabs and light buildings on shallow foundations.^{59,60} Prior to the Bolton Hill Tunnel project, compaction grouting had been used in the Baltimore subway project to correct settlement problems caused by subway tunnel construction — but only *after* the tunnel had been completed and settlement of overlying buildings had occurred.⁵⁷ However, the Bolton Hill project marked a fundamental change, in that compaction grouting was conducted *during* the excavation of the tunnel at locations just above the crown. In this way, major surface settlements were prevented from developing at the source. Although compaction grouting has practical

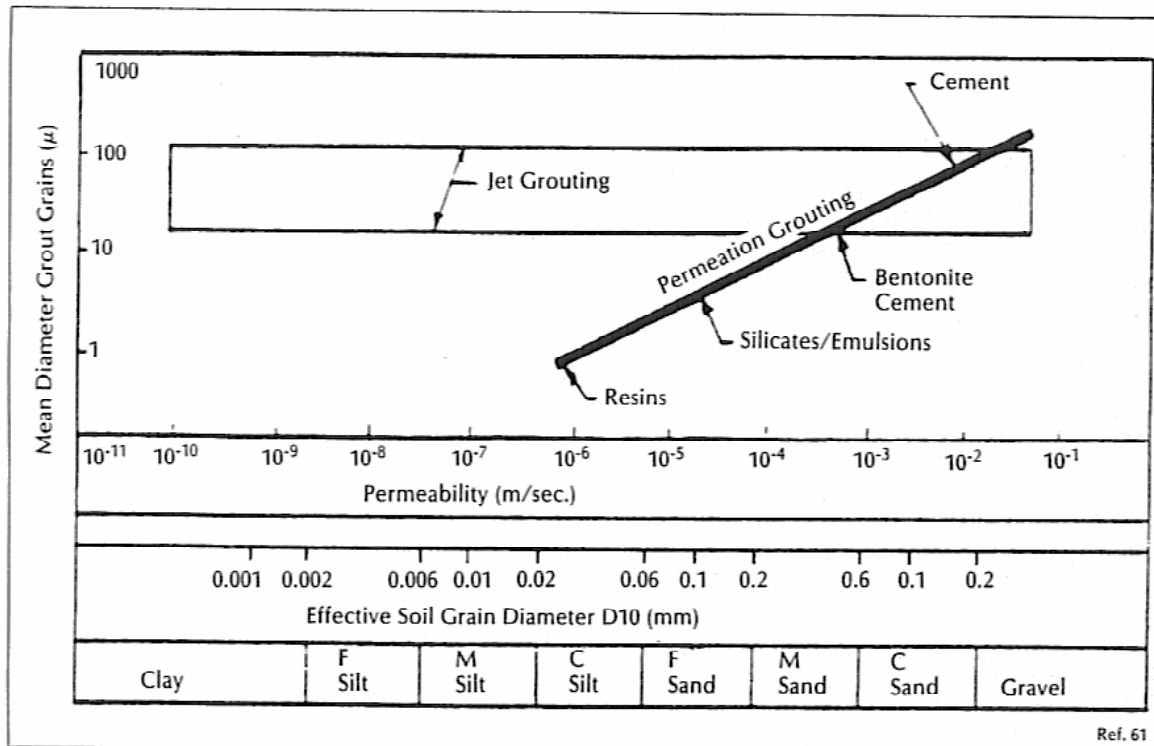


FIGURE 23. Groutability of soils in relation to grout and soil properties.

and technical limitations, its popularity is growing, mainly as the result of the well-researched (and publicized) Bolton Hill project. However, its application should be most carefully reviewed when dealing with tall structures or buildings that can tolerate only the smallest differential movements. Under such conditions, it is imperative to attack the cause of the settlements at the source, and prevent them from migrating away from the excavation. Permeation or replacement grouting are then necessary.

In certain ways, the techniques involved in *permeation grouting* are the oldest and best researched. Its history may be traced back to 1802 and the efforts of Charles Berigny to repair the Dieppe harbor sea lock that had been damaged by slump and washout. It would appear that grouting was first attempted in the U.S. during the construction of the New Croton Dam, in New York, although the first major application followed around 1910, during shaft sinking and tunnelling for the Catskill Aqueduct, also in New York.

The aim of the method is to introduce

grout into soil pores without any essential change in the original soil volume and structure. The properties of the soil, and principally the geometry of the pores, are clearly the major determinants of the method of grouting and the materials that may be used (see Figure 23). Excellent reviews of the subject are provided by the FHWA, Cambe-fort, Karol, and Littlejohn.^{62,63,64,65} New developments in procedures, monitoring and materials as related to injection exclusively by the tube à manchette (sleeved pipe) system have arisen. Other permeation grouting methods, principally from Japan, are described in an earlier study.⁶⁶

Replacement grouting is the youngest major category of ground treatment. According to Miki and Nakanishi, and Miki, the basic concept was propounded in Japan in 1965, but it is generally agreed that it is only within the last 10 years that the various derivatives of jet grouting have approached their full economic and operational potential (see Figure 24 and Table 3). Its development was fostered by the need to thoroughly treat soils from gravels to clays to random fills in

Table 3

Major Categories of Jet Grouting Variants

Original Japanese Name	Principal of Operation	Jetting Pressure (N/mm ²)	Jetting Nozzle Dia. (mm)	Revolving Rate (rpm)	Anticipated Column Dia. (cm)	Notes
Jet Grout (JG)	Upper water & lower grout jet	20	?	None	—	Panels only, soon obsolete
Chemical Churning Pile (CCP)	Single grout jet	20 - 40	1.2 - 3.0	20	30 - 60	1. Chemicals now replaced by cement 2. Similar to Rodinjet 1
Jumbo Special Grout (JSG)	Single jet of grout enveloped in air	20	3 - 3.2	6	80 - 200	1. Originally called Jumbo Special Pile (JSP) but name changed for patent reasons 2. Similar to Rodinjet 2
Column Jet Grout (CJG)	Upper water & air jet & lower grout jet	40 - 50	1.8 - 3.0 (upper) 3.0 - 5.0 (lower) (8 - 9 mm in Kajima system)	5	150 - 300	1. Referred to as "half replacement" 2. Similar to Rodinjet 3 or Kajima/GKN Keller system
Mini Max (MM)	Like CCP but uses special "chemi-colime" cement	20	1.2	20	80 - 160	Specially for very weak soil & organics (e.g., soft peaty clays under water)
Jumbo MiniMax (JIM)	As for MM except for addition of 20 - 40 cm wing jet	20	1.2	20	20	Specially for very weak soil & organics (e.g., soft peaty clays under water)
Super Soil Stabilization Management (SSS-Man)	Air water jet used to excavate volume completely underwater. This is then surveyed ultrasonically. If OK, then tremied full of desired material	20 - 60	2 - 2.8	3 - 7	200 - 400	1. To provide absolute control over shape & composition of column 2. Effective to over 70 m depth 3. "Complete replacement" 4. Most expensive technique, but ensures desired performance

areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements. As indicated in Figure 23, jet grouting can be executed in soils with a wide range of permeabilities. Indeed, any limitations with regard to its applicability are imposed by other soil parameters

(e.g., the shear strength of cohesive soils or the density of granular deposits).

The ASCE Geotechnical Engineering Division Committee on Grouting defined jet grouting as a "technique utilizing a special drill bit with horizontal and vertical high speed water jets to excavate alluvial soils and produce hard impervious columns by

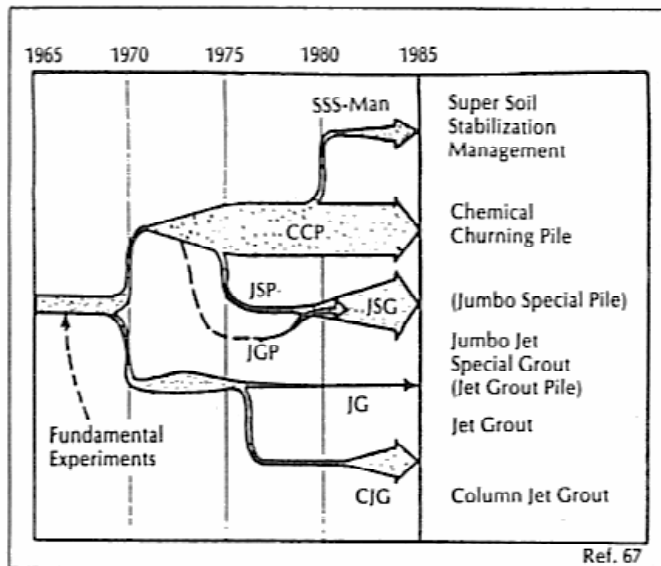


FIGURE 24. Development of jet grouting methods in Japan.

pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn.”⁶⁹ Figure 25 depicts one particular type in which the soil is jetted by an upper nozzle ejecting water at up to 60 N/mm² inside an envelope of compressed air at up to 1.2 N/mm². The debris is displaced out of the oversized hole by the simultaneous injection

of cement-based grout through a lower nozzle (up to 7 to 8 N/mm²). Other simpler variants utilize grout jetting only to simultaneously erode and inject, giving much more of a “mix in place” action. At the other extreme, the newest Japanese Super Soil Stabilization Management (SSS-Man) system provides total (and verifiable) excavation of the soil prior to grouting or concreting (see Figure 26). Clearly, each system has its own cost implications.

Most jet grouting is conducted to provide circular columns, but panels or membranes can be cut in the ground by omitting rotation during the withdrawal of the tool: the nozzles then act monodirectionally.

Applications of jet grouting have been reported throughout Western Europe, the Far East, Soviet Union and South America. Currently, there is a small but growing market in North America, largely under the promotion of certain government agencies and specialist contractors, following a slow and uncertain start.⁷⁰ Major applications are:

- Underpinning of existing structures
- Formation of water cutoff walls or

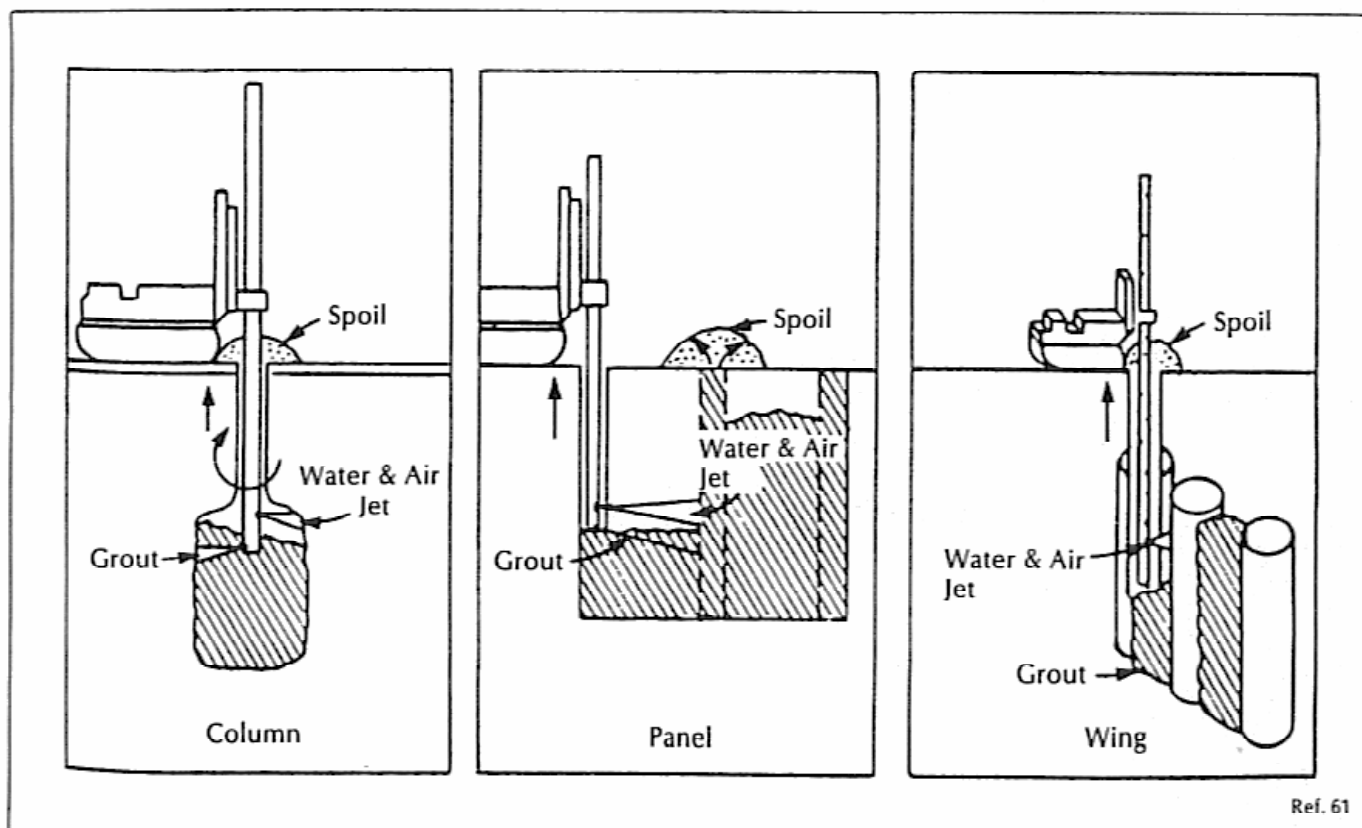


FIGURE 25. Jet grouting options using the three-fluid system.

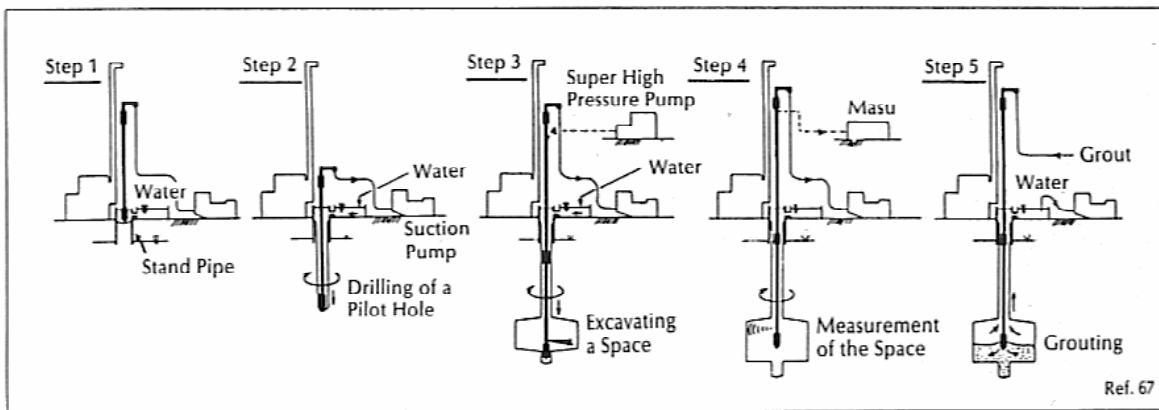


FIGURE 26. Working procedure of the SSS-Man method of jet grouting.

diaphragms (to 10^{-6} to 10^{-9} cm/sec)

- Foundation soil consolidation for new structures, embankments and retaining walls
- Soil consolidation for the excavation of shallow tunnels
- Excavation support for open cuts and shafts
- Land slide stabilization

A useful review of jet grouting was provided by ASCE, while a more comprehensive review is to be published soon by Kauschinger.⁷¹

New Development Trends in Permeation Grouting. Individual companies continue to develop their own systems for installing the grouting tubes and executing the grouting. However, certain trends are evident on a worldwide basis:

- use of long mast "one stroke" drilling rigs, permitting fast installation programs and usually operating with bentonite flush or a self-hardening drilling mud to further reduce operating costs.
- use of more flexible grouting tubes delivered in one piece to the site in order to ease installation, especially from within restricted tunnel access conditions, and in order to reduce the risk of malfunction due to leakage at joints. Alternatively, steel grouting tubes are being used in certain cases, doubling as in situ reinforcement or underpinning for very delicate structures.

- use of hydraulically or pneumatically inflated double packers for grouting, thus reducing labor effort, ensuring efficient sealing and permitting any deviated or damaged hole to be "rescued" and still be used for grouting.

In order to appreciate more fully the significance of the new developments, it is necessary to first reconsider the broad categories of grouts used in permeation. Mongilardi and Tornaghi developed the following classification on the basis of rheological performance.⁷² The classification is presented in the order of increasing penetrability (and cost):

- particulate suspensions (Binghamian fluids) — cement-based grouts
- colloidal solutions (evolutive Newtonian fluids) — chemical grouts
- pure solutions (non-evolutive Newtonian fluids) — chemical grouts

Suspensions of solids in water are termed *unstable* when water loss by bleeding is significant. This instability occurs when pure cement grouts are used at high water/cement ratios in fissured rocks, since the water acts largely as a vehicle for cement grains. A suspension is termed *stable* when bleeding is negligible, as required in general for the treatment of granular soils (with cement-clay and cement-bentonite mixtures). Stabilized thixotropic grouts have both cohesion (yield value) and plastic viscosity

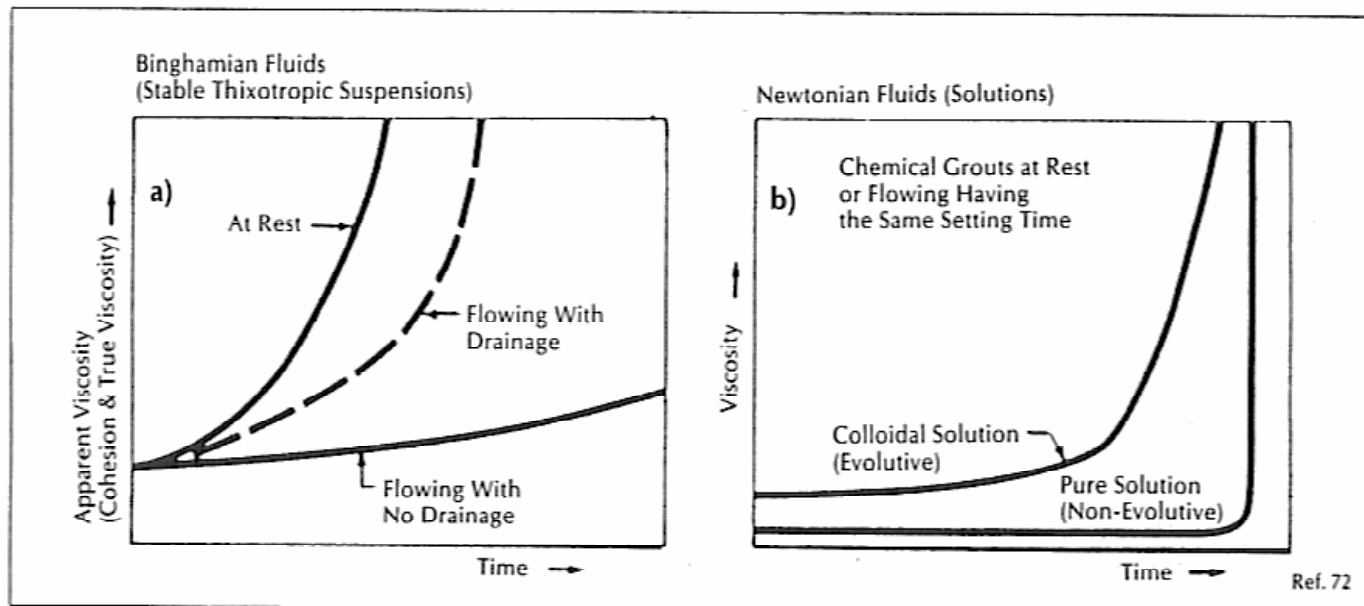


FIGURE 27. Rheological behavior of typical grouts.

increasing with time at a rate that may be considerably accelerated by drainage under pressure — i.e., “pressure filtration” (see Figure 27a). Though the addition of colloidal products can minimize bleeding, filtration must be always considered as an important design factor with respect to penetrability

and the final effects of water loss on mechanical properties and volumetric yield. The poor permeation of suspensions in granular soil with a permeability lower than 10^{-1} cm/sec involves the additional or alternative use of chemical solutions in order to minimize hydrofracturing effects.

Table 4

**Composition & Characteristics of Mistra Grout,
Lot 1PB, Passante Ferroviario, Milan, Italy**

Composition	Cement/Water Ratio	0.35
	Additives/Water Ratio	0.04 - 0.05
Bleed Capacity (%)		0 - 2
Marsh Viscosity (sec.)		33 - 37
Rheometer Parameters	Apparent Viscosity (cP)	8 - 12
	Plastic Viscosity (cP)	5 - 8
	Yield Strength (Pa)	1.5 - 5
Filter Press Test at 0.7 N/mm ²	Filtrate (cm ³) after 30'	36 - 72
	Filtration Rate (mm ^{-1/2})	0.016 - 0.032
U.C. Strength (N/mm ²) of Grouted Sand after 28 Days		1.2 - 1.8

Ref. 56

By far the best known *colloidal solutions* consist of diluted sodium silicate with inorganic or organic reagents that produce relatively soft to hard silica gels. The term *evolutive* means that viscosity increases before setting at a rate that mainly depends on the concentration (see Figure 27b, upper curve).

The more expensive *pure solutions* based on acrylic, phenolic or amino resins, are *non-evolutive* Newtonian fluids since viscosity may be kept constant until setting within an adjustable period of time (see Figure 27b, lower curve). This outstanding property, associated with a very low viscosity, allows the impregnation of the finest granular soils within the practical and economical limits imposed by the rate of flow and pressure (silty fine sands with a virgin permeability not lower than 10^{-4} cm/sec).

Regarding the *particulate suspensions*, main obstacles to penetrability are related to:

- the maximum particle size of the solid components in the grout relative to the pore sizes in the soil; and,
- the rate of pressure filtration that may induce rapid clogging even under low pressures.

The first problem is being tackled by broadly introducing fine grained cements (for example, MC500) in which the average particle size is claimed to be 4 microns, and by minimizing grain agglomeration or flocculation by the improvement of mixing plants and the addition of dispersing agents.⁶⁴ In contrast, the filtration problem has represented the main obstacle in the past, since in conventional "stable" grouts (Type C in Figure 28) a reduction of water loss rate can be obtained only at the cost of increasing viscosity, by an additional content of active colloidal particles such as bentonite. Usual chemical additives (such as fluidifiers, retards or dispersing agents) reduce viscosity, but the filtration rate remains significant (grout Type C in Figure 28).

Recent research, however, led to the development of an entirely new class of cement-bentonite grouts named Mistra (Type B in Figure 28) that has the following

major properties:

- very low filtration rates: very close to pure bentonite muds even at very low viscosity
- no bleeding
- low values of yield point and plastic cohesion over an adjustable period of time (up to several hours)
- higher long-term strength and lower permeability in comparison with conventional stable grouts having the same cement content

With respect to composition, the water/cement ratio can vary from 1 to 5, with the overall content of additives ranging from 3 to 6 percent by weight of water, thus providing a wide range of rheological and long-term properties. For example, the Mistra grout being used in Lot 1 PB of the Milan subway has the composition and properties as listed in Table 4.⁵⁶

The practical advantages of this new class of grouts to the tunnelling engineer can be summarized as follows:

- improved penetrability under a lower pressure in sandy-gravelly soils
- lower water loss, and therefore, a greater volume of voids filled with the same volume of grout
- the possibility to fill all the voids consistent with the size of individual cement particles and therefore to permeate medium-coarse sands with refined products, minimizing any hydrofracturing effects.

Chemical grouts based on sodium silicate solutions and inorganic reagents (e.g., sodium aluminate or sodium bicarbonate) can produce only soft gels (i.e., an unconfined compressive strength [UCS] of < 0.3 N/mm²) for waterproofing sands, since high dilution is necessary to achieve low viscosity and the appropriate setting time. Most recently, the Japanese have developed specialized drill and grout systems, such as Limited Area Grouting (LAG) and Double Tube Drilling (DDS), that can handle the problems of

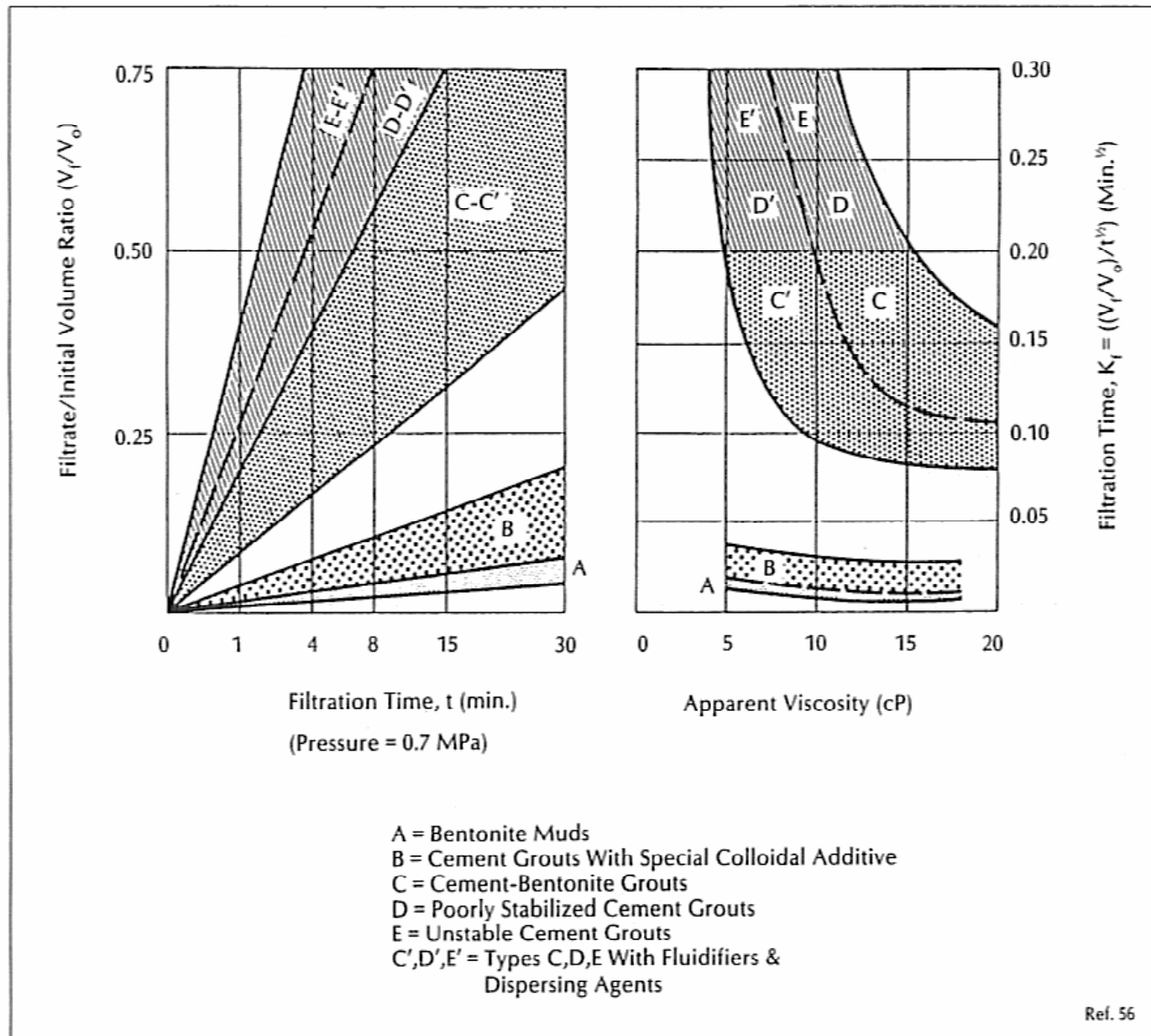


FIGURE 28. Ranges of significant rheological properties for various types of particulate suspensions.

injecting the stronger flash setting grouts of this type, although these systems are only practical in the softer uniform deposits.^{73,66} Each method uses a small diameter double rod drilling system through which the sodium silicate and reagent phases are pumped to be mixed and ejected at the bottom during the withdrawal of the drill string.

The introduction of organic reagents in the early 1960s allowed for the adjustment of setting time independent of silicate concentration, thus providing "hard to soft" gels depending on the silicate to water ratio. However, in certain areas including Japan and Germany, such organically based

reagents are not environmentally acceptable. Creep effects may be a significant problem for silicate gel stabilized soils if the design involves a high and permanent loading.⁷⁴ In addition, questions of permanence and durability under certain conditions may be valid. Syneresis is the major factor in this context, and it is not widely understood that while it is significant in pure gel samples, it tends to become negligible in fine-medium sands (see Figure 29).

In light of these problems, potential and real, the newest developments have led to the evolution of a new type of chemical grout (Silacsol) composed of an activated silica liquor and an inorganic reagent based

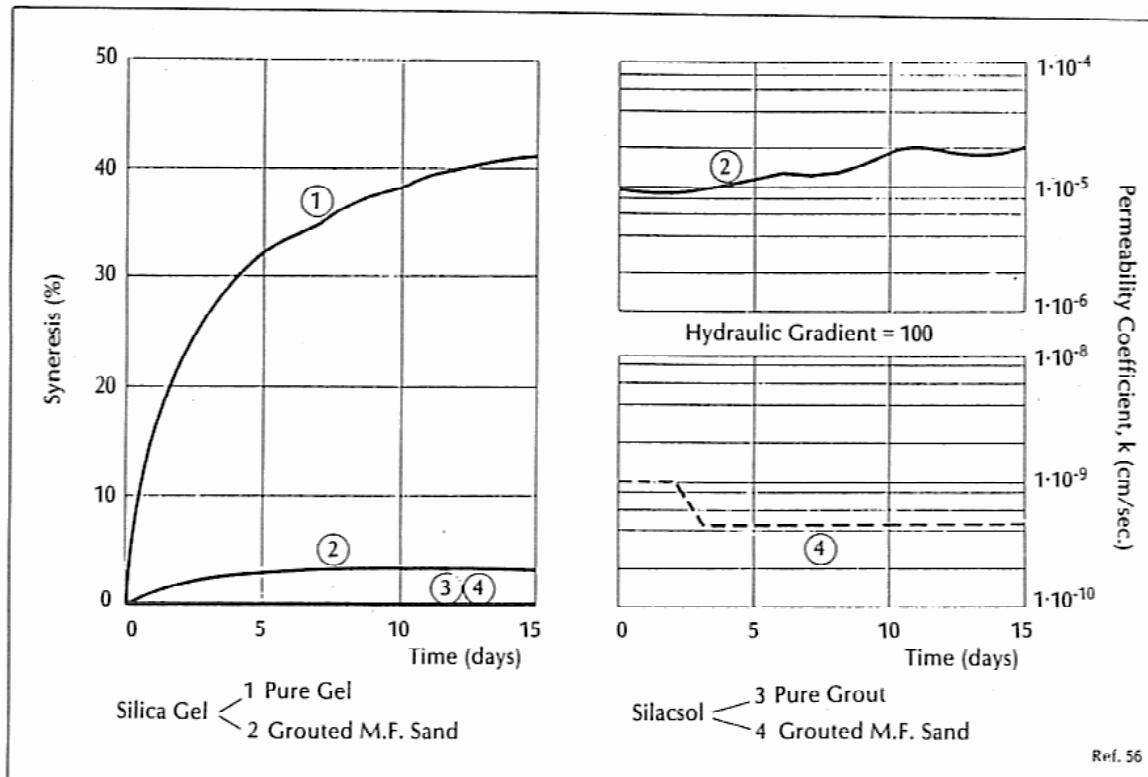


FIGURE 29. Effect of time on syneresis and permeability of typical chemical grouts.

on calcium. As opposed to commercial alkaline sodium silicates that are aqueous solutions of colloidal silica particles dispersed in soda, the liquor is a true solution of activated silica. The activated dissolved silica, when mixed with the reagent, produces calcium hydrosilicates with a crystalline structure that is quite similar to that obtained by the hydration and setting of cement. The resulting product is a complex of permanently stable crystals. Hence, the reaction is not an evolutive gelation as in the case of silica gels which involves the formation of macromolecular aggregates and the possible loss of silicized water (syneresis). On the contrary, it is a direct reaction on a molecular scale. This type of mix, recently developed in France and successfully used in Italy, presents the same groutability range as common silica gels: medium to fine sands can be effectively treated. Even if larger voids or fissures are accidentally created by hydrofracturing, a permanent filling is assured without any syneresis risk. The activated silica mix has the stability of

a cement grout owing to the nature of the resulting products (insoluble crystals of calcium silicate) and to the absence of aggressive by-products, thus providing full safety against pollution.

Other outstanding features relative to silica gels of similar rheological properties are:

- the far lower permeability of 10^{-10} cm/sec for Silacsol, remaining constant over test periods of 15 days at hydraulic gradients of 100, as opposed to 10^{-5} cm/sec for silica gel
- the better creep behavior of treated sands for grouts of similar strength (2 N/mm² at a quick strain rate of 0.65 percent per minute): the data in Figure 30 show a much lower long-term deformability at the same stress level and creep rupture at about double the stress level

Instrumentation. Throughout the grouting industry, the use of computer-aided devices such as monitors and controls over grouting

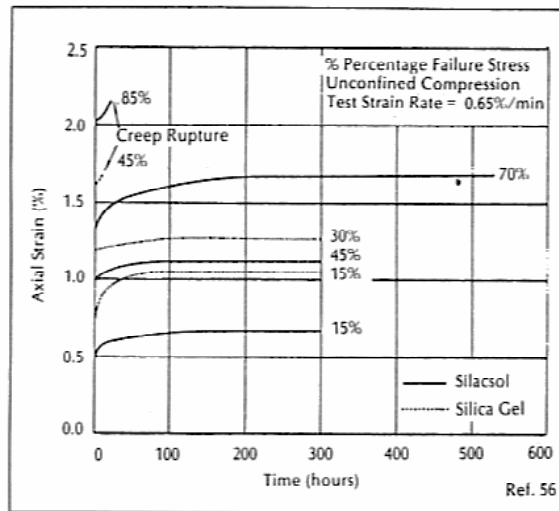


FIGURE 30. Unconfined creep results for silica gel and Silacsol grout (samples of grouted medium fine sand).

operations in the field is increasing. This growth is reflected in several of the papers presented at the "Issues in Dam Grouting," session of the ASCE Convention, Denver, 1985.⁷⁷ The most effective of these instrumentation systems, as far as injections are concerned, is similar to the electronic Paguro system of centralized monitoring and grouting control that was used in Italy. This system monitors in real time numerically and graphically the full injection characteristics of each pump (the setting of which remains under manual control). It thereafter gives a printout summary of each sleeve injected (including volume, maximum and average pressures, flow rates and time). Such data then provide the basis for the technical review of the grouting conducted (e.g., grout take analyses) and the quantities of work executed for payment purposes. Clearly, the investment in such sophisticated equipment is economically justifiable only in projects of appreciable scale and/or complexity such as the Milan subway.⁷⁶

Most recently, however, a major breakthrough has been made in Italy in the exploitation of instrumentation for soil investigation and grout parameter design. The sensors of the Papero system for drilling investigation continuously record the drill penetration rate, rotational speed, thrust,

torque and flush pressure encountered in drilling a certain exploratory hole. These data are combined to give a single unified factor — specific energy. Thereafter, the computer relates this factor to ground type, and prints out a geological log with boundaries at 10-cm intervals (see Figure 31). This geological log permits optimizing the subsequent drilling and grouting parameters as well as furnishing invaluable information to the tunnelling contractor in that potentially dangerous conditions (e.g., sand runs) can be closely predicted. The accuracy of the geological log has proved exceptional given the conditions of the Milan subway project (mixed gravels, sands and silts to over 25 m in depth) and groups of three investigatory holes have been routinely drilled at about 6-m intervals from the pilot tunnel along much of its length.

The key to the accuracy is obviously the ability of the computer to relate specific energy with ground type. This accuracy has been achieved by conducting statistical analyses of the specific energies recorded at discrete depth intervals, in correlation with visual observations (from core samples) of the ground type. In this way, the influence of depth on in-situ ground properties as well as other factors such as the hydrological regimes and borehole inclinations are accommodated, which is not the case in other, less successful systems of drilling parameter analyses.

New Development Trends in Jet Grouting. In contrast to the sensitivity and sophistication of some aspects of permeation grouting, the principle of jet grouting stands as a straightforward positive solution, using only cement-based grouts across the whole range of soil types. This opinion is enhanced by the very dramatic photographic evidence from excavated test sections (see Figure 32). However, it must be emphasized that any system that may involve the simultaneous injection of up to three fluids at operating pressures of up to 60 N/mm² must be handled with extreme care and only in appropriate applications, circumstances and ground conditions. The credentials, resources and methods of the specialist contractor

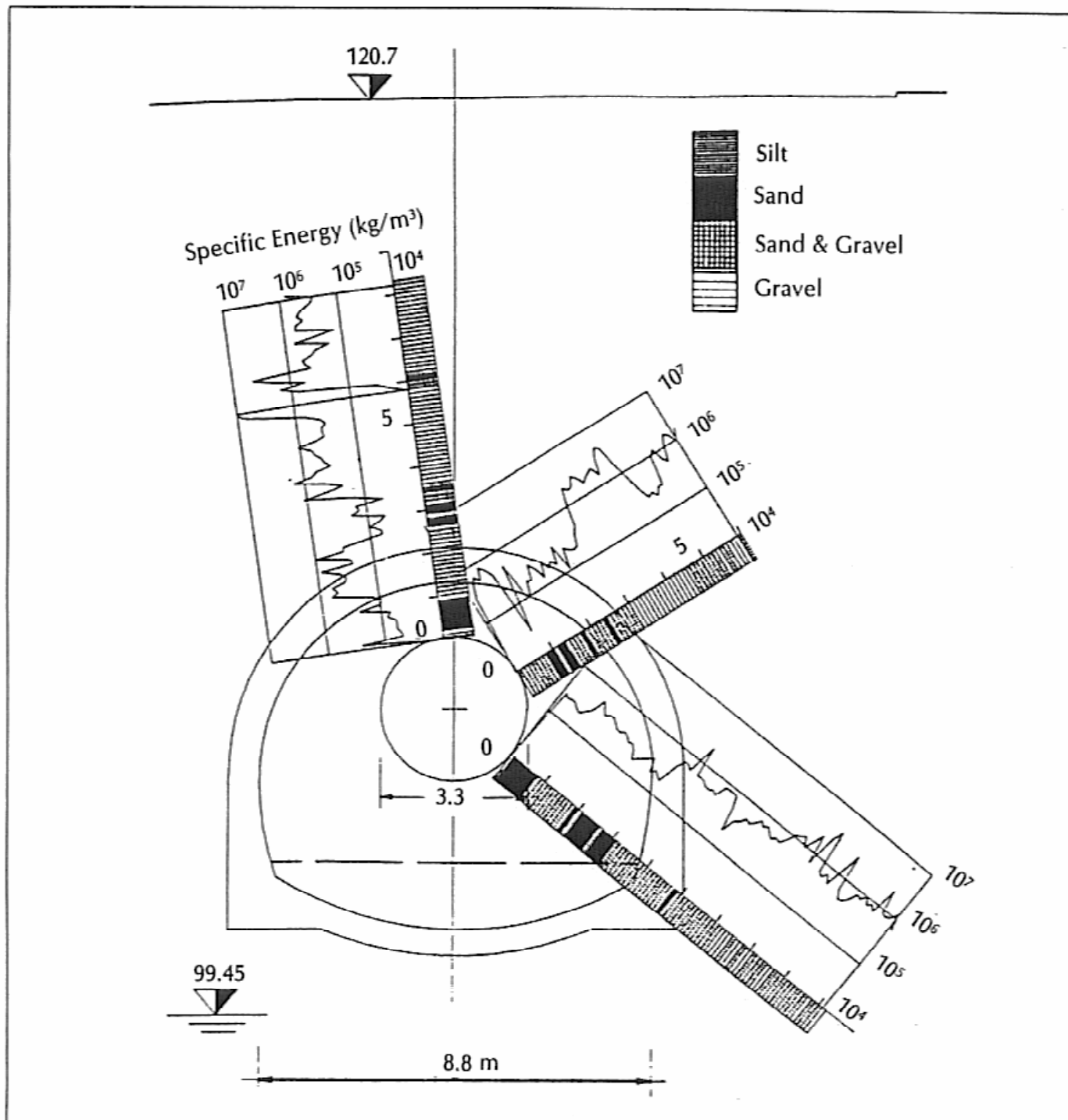


FIGURE 31. Soil profiles derived from the evaluation of electronically recorded drilling parameters in terms of specific energy (Milan Metro, Line 3).

must, therefore, be reviewed with special care.

The major research trends are heading towards refining operational methods and equipment, trying to obtain a closer understanding of the interaction of ground types and grouting parameters, and in developing monitoring and control systems.

Regarding equipment and methods, a two-fluid jetting system has been successfully developed. In this system, the high

pressure grout jet is concentrated in a compressed air envelope (see Figure 33). Thus, the radius of influence of the grout jet is considerably increased, even in saturated conditions. Another major advantage is that it is operationally far simpler than the three-fluid system described above, and much faster and less expensive.

Regarding design, ASCE noted that the fundamental aspects are grout mix, jet nozzle energy and flow rate, and grout pipe



FIGURE 32. Rodinjet 3 column in variable alluvial materials excavated 9 days after grouting.

rotation and withdrawal rates. Miki and Nakanishi reported on data recorded from full-scale tests conducted over many years, and quantified the relation between method, soil type and resulting column diameters for particular operating parameters.⁶⁷ Coomber stated that water jet pressure and monitor withdrawal rate are the most significant of these parameters (see Figure 34), while similar data were presented in different format by ASCE (see Figure 35).^{61,3}

Grout mix constituents and composition can be varied to meet the specific requirements and, for example, the addition of bentonite will reduce the soilcrete strength and decrease the soilcrete permeability. Mix viscosity should be low to promote uniform treatment to the greatest extent, and water/cement ratios (by weight) are rarely less than 1.0. In permeable granular materials, much of the injection water may be expected

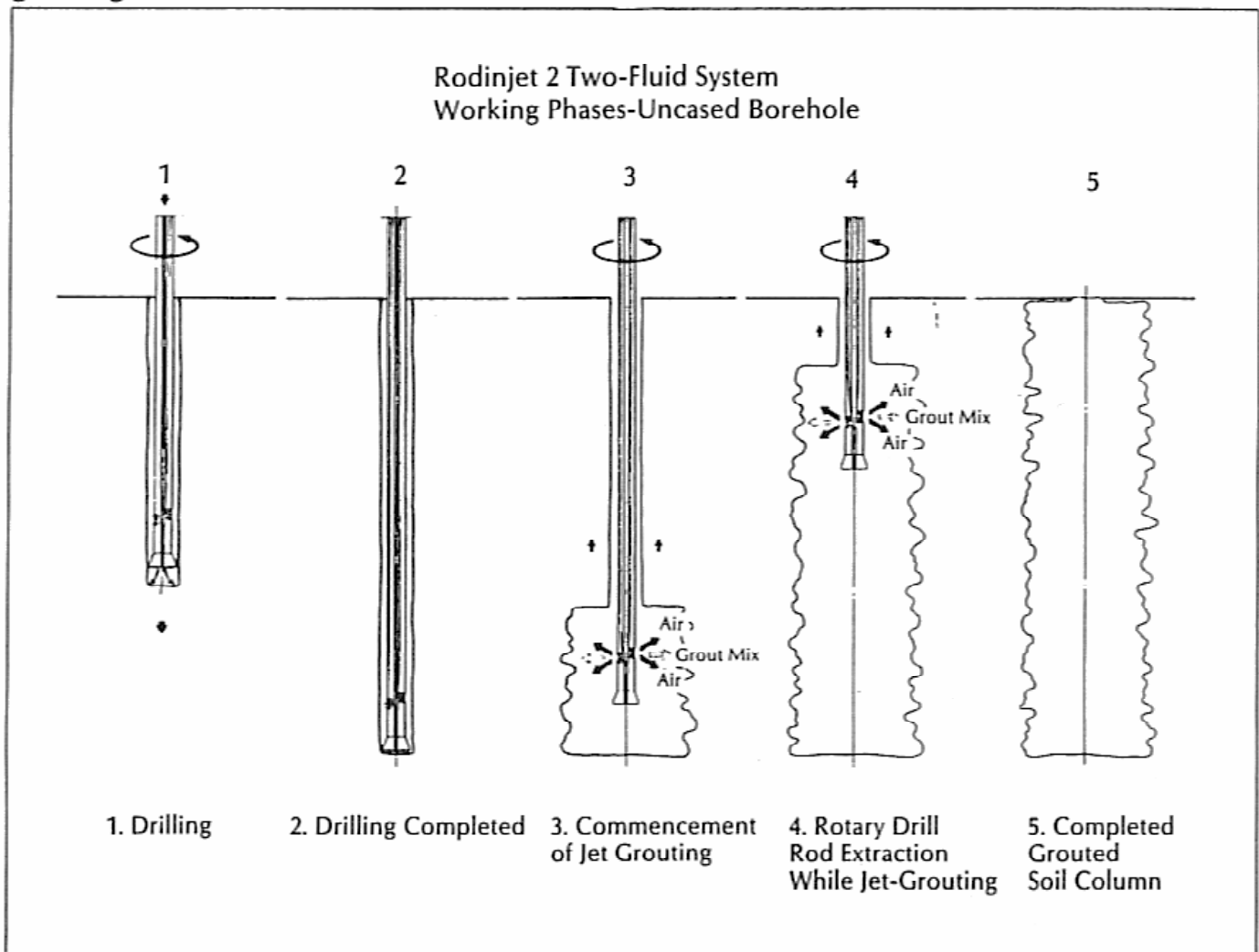


FIGURE 33. Jet grouting with two fluid system (Rodinjet 2).

to be drained out both from soil and grout, whereas in a cohesive soil of low permeability, poor or no drainage is likely. This lack of proper drainage is a principal reason why the strength of the grouted column (depending primarily on the final water/cement ratio) is much lower in clay than in sand and gravel when all other factors are equal.

Regarding monitoring, recent developments in instrumentation include the Paperjet — a parameter recording system similar to Papero and Paguro. Nevertheless, it should remain an integral part of every jet grouting program to have a field trial prior to the commencement of production in order to verify and optimize operating parameters. Such trials should include visual inspection

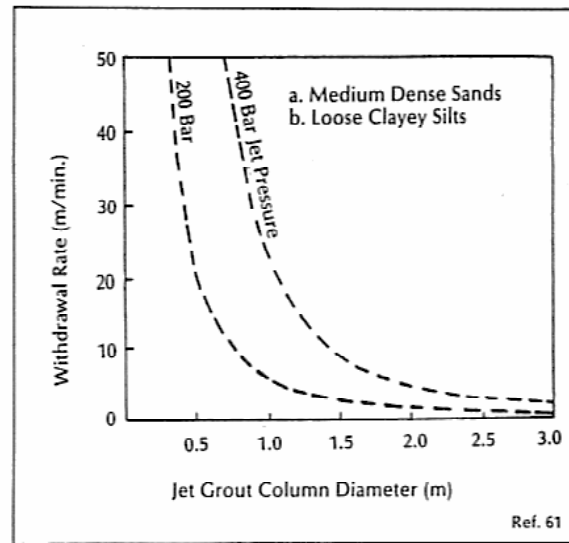


FIGURE 34. Typical design curve for jet grouting.

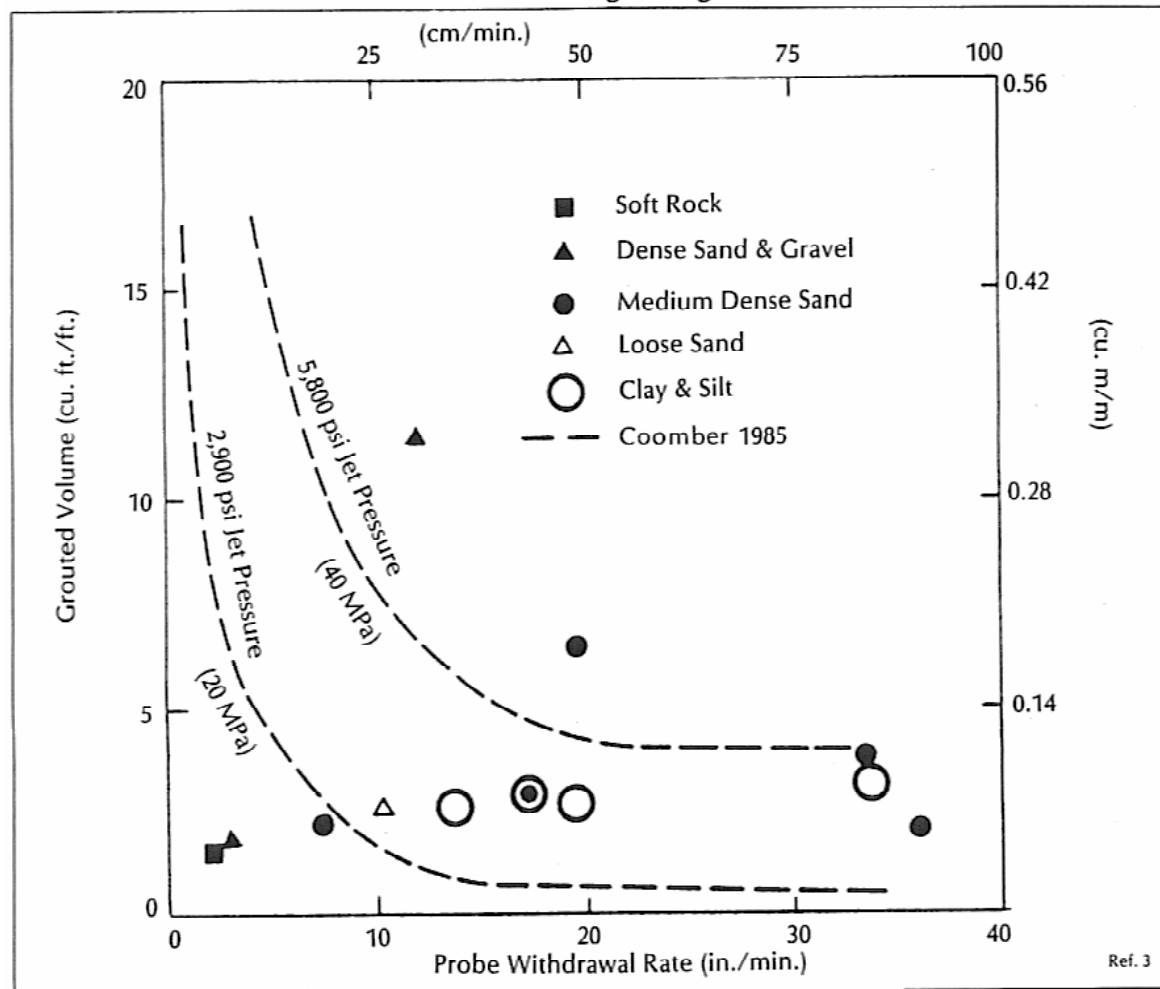


FIGURE 35. Probe withdrawal rate effects on jet grouted volume.

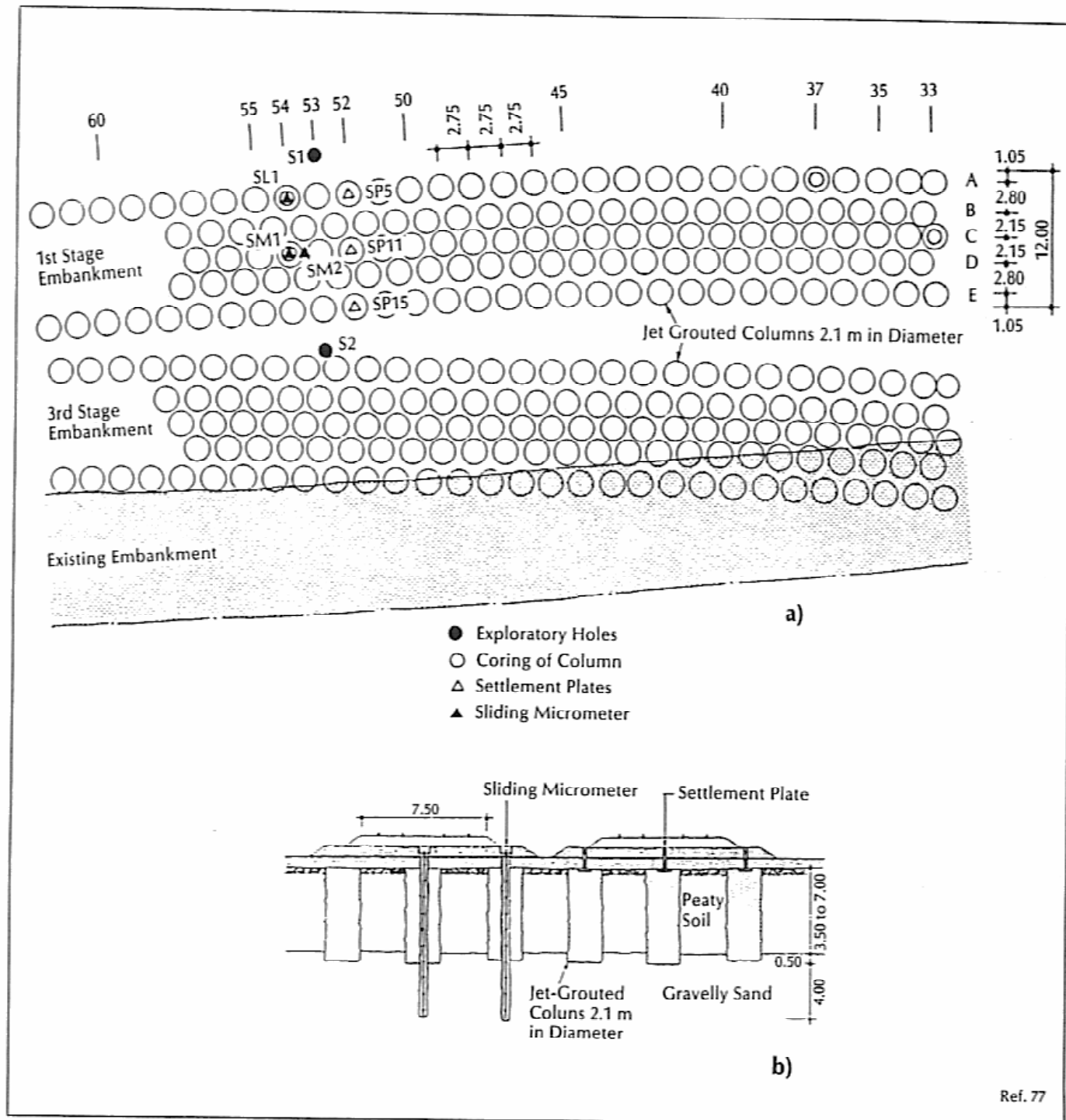


FIGURE 36. Typical plan, a), and cross section, b), of embankments, soil improvement and instrumentation, Como, Italy.

of the grouting by excavation wherever practical. An excellent example of the planning and execution of such a program is provided by DePaoli *et al.* in their description of the stabilization by the jet grouting of a highly compressible peaty zone up to 5 m thick under a railway embankment near Como, Italy.⁷⁷ A range of methods (one-, two- and three-fluids with and without pre-washing) and grouting mixes and parameters were systematically assessed prior to the

highly successful treatment of 25,000 m³ of soil with 1,300 columns (see Figure 36). Strengths of up to 7 N/mm² were achieved (typically 1 to 2 N/mm²) with "negligible" compressibility in service.

These remarks on verification and testing apply equally to projects executed with other forms of ground treatment. Experience, often bitter, has underlined the value of such testing, especially when set against the scale of the disruptions that may result "down

the line" due to inefficient or inappropriate treatment procedures however well-intentioned at the planning stage.

Case History. One of the first major applications of jet grouting for "block" consolidation from the surface, was the project executed on Contract 106 of the Singapore Mass Transit Railway in 1983-84. The job involved four tunnel sections with a total length of about 400 m, between Dhoby Ghaut and City Hall Stations, and was first described by Tornaghi and Cippo, and Mongilardi and Tornaghi.^{78,72}

The geology of Singapore Island is complex, but typically consists of beach sand and fill (3 to 5 m deep) that overlies very soft peaty clay, marine clay and fluvial soils to combined depths of 15 m or more. The base of this sequence consists of stiff to hard cohesive soils or soft rock. Clearly, the soil is too fine for permeation grouting. Groundwater levels are only 1 to 2 m below surface.

In these hydrogeological conditions, even shield excavation could be difficult and unsafe without soil improvement, since the permissible magnitude of vertical ground movements at the surface is restricted to a few centimeters. It was decided to use jet grouting from the surface, extended to the full excavation section above soft rock or very stiff to hard soil, and limited in the upper part in order to create an arch of consolidated soil typically 1.5 m thick. Figure 37 shows the arrangement for the first 70-m long section from Dhoby Ghaut Station, in which the annular thickness was increased to 3 m close to the station excavation where the shield could not operate.

In advance, a full scale field test was conducted. As shown in Figure 38, two different layouts of jet-grouted columns were tested: 0.6 and 0.8 m between centers of staggered elements. For each layout two different quantities of grout were injected (corresponding theoretically to 600 and 800 l/m³ of soil).

The four schemes that resulted (involving a total of 62 columns) were arranged to form the sides of a square area to be excavated subsequently for direct inspection. The

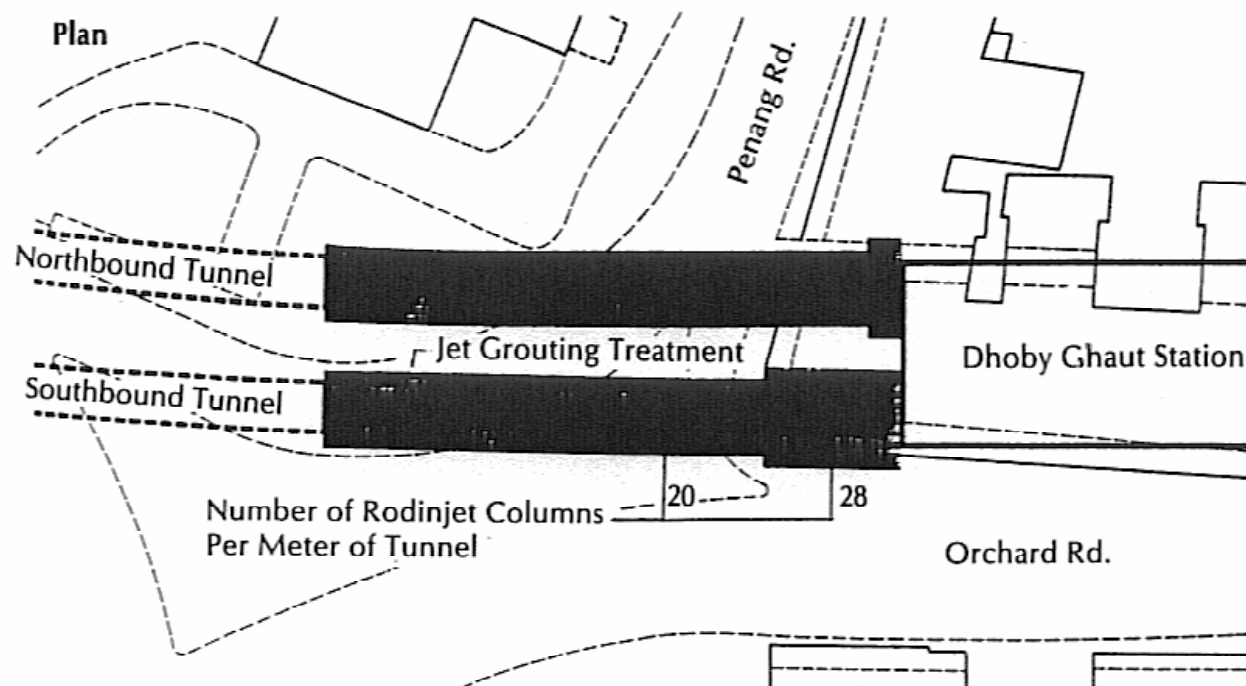
following general procedure was applied to each scheme: drilling to 10.5 m depth, treatment from the bottom to 0.5 m depth by the Rodinjet 1 technique, injecting a grout with a water/cement ratio of 1.6 and a grouting pressure of 40 N/mm².

The instrumentation previously installed consisted of two inclinometers to check horizontal soil displacements, two piezometers to record pore pressure build-up and dissipation, and nine datum points to check vertical soil displacements with reference to a fixed point 20 m distant from the perimeter of the test area.

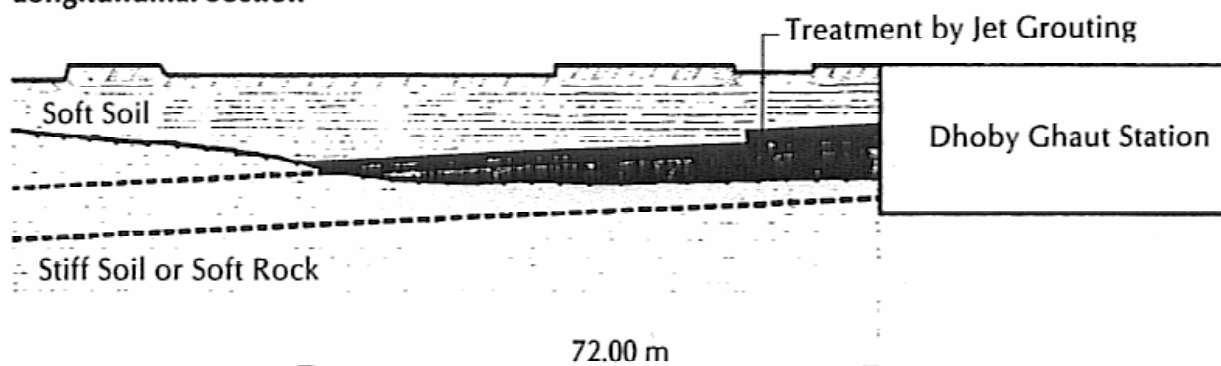
Inclinometer 1-1 (1 m from the axis of the external row of Scheme IV) recorded the maximum horizontal displacement of 23 cm at a depth of about 6 m; at a distance of 6 m, the displacements were less than 5 cm. The excess pore pressures recorded by the cells 6.5 m deep, and 3 m and 6 m from the perimeter of the treated area, were low (0.02 to 0.04 N/mm²) throughout. After completion of the treatment, the mean heave values were approximately 30 cm at 1 m distance, 17 cm at 3 m, 5 cm at 6 m and 1 cm at 10 m.

The total volume of injected grout was 190 m³, corresponding to 70 percent of the theoretically engaged soil volume (270 m³). About 70 m³ of the soil-grout mixture was ejected during grouting and it is estimated that the overall surface upheaval corresponded to about 60 m³ of upward displaced soil. Therefore, it may be inferred that the remaining 60 m³ of injected grout caused mostly radial displacement and compression effects.

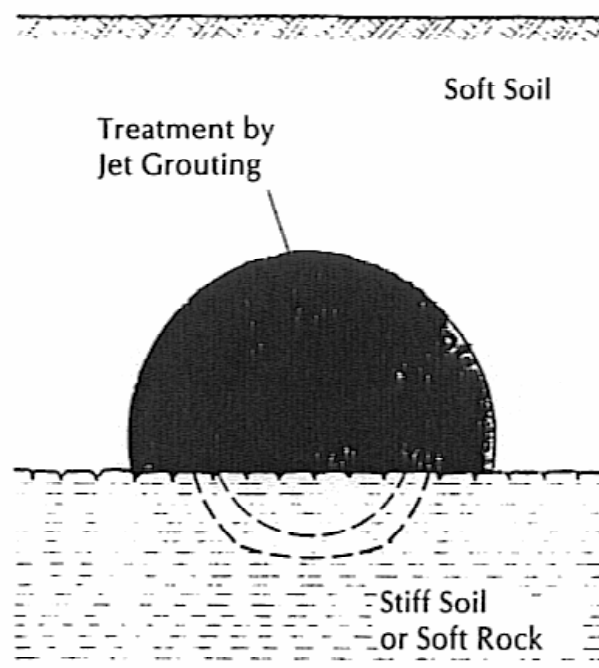
Some two weeks after completion of the treatment, eight control boreholes were drilled: two for each scheme (I to IV) and for each pair, one in the center of a column and one between the centers of two adjacent columns (see Figure 38). Laboratory tests were carried out on 40 representative samples; plots of mean values of bulk density and UCS versus depth are shown in Figure 39 both for native and jet grouted soil. Assuming a mean relation between one month's UCS (R in N/mm²) and the overall cement/water ratio, $R = 6(c/w)^2$, and full saturation, the actual composition of the



Longitudinal Section

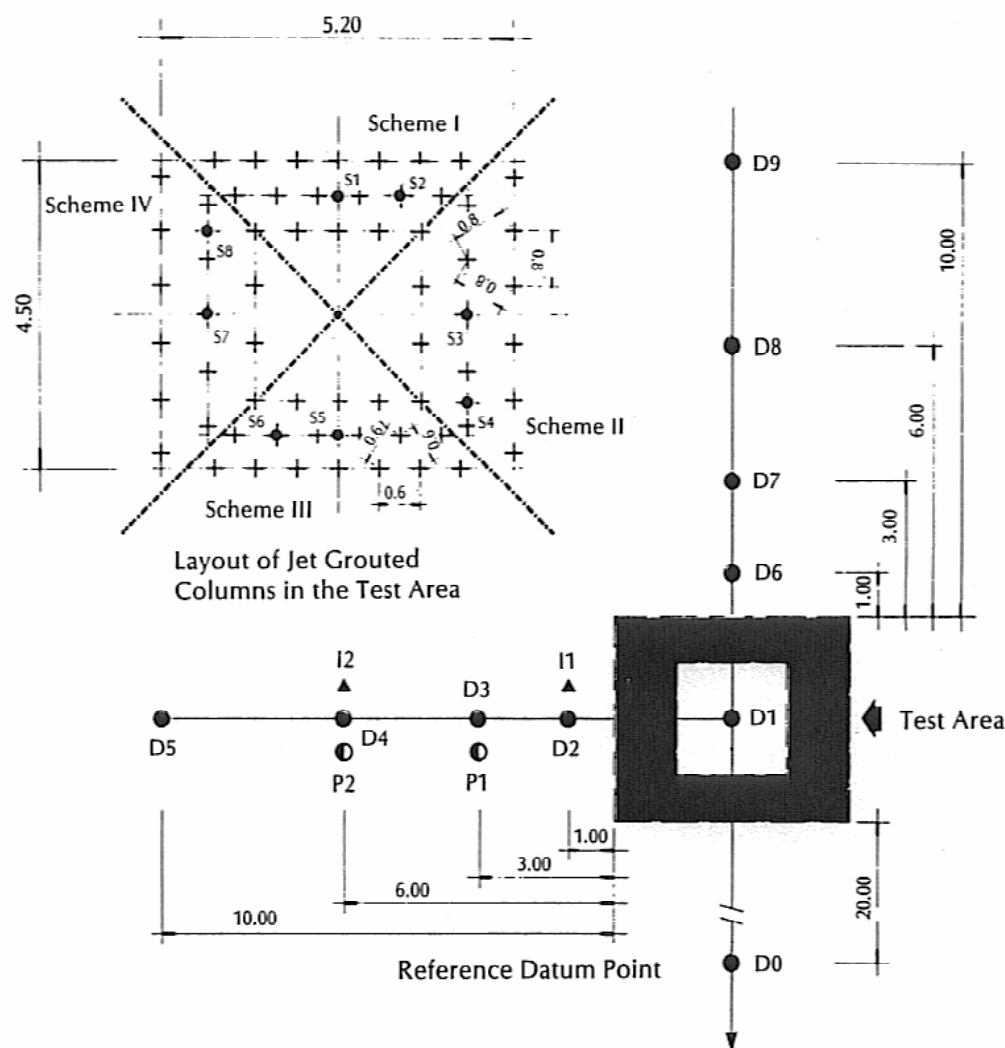


Cross-Section



Ref. 78

FIGURE 37. General arrangement of jet grouting MRT 106, Singapore.



Scheme	Spac. (m)	No. of Col. Per m ² of Soil	Volume of Mix Per l.m. of Col.	Volume of Mix Per cu. m of Soil
I	0.6	3.2	187	600
II	0.8	1.8	333	600
III	0.6	3.2	250	800
IV	0.8	1.8	444	800

- ▲ Clinometer
- Piezometer
- Borehole
- Datum Point

Ref. 78

FIGURE 38. Layout of test section, MRT 106, Singapore.

treated soil and the cement content can be estimated (see Figure 40). The average content of about 2 kN/m³ corresponds to 400 liters of grout per 600 liters of soil, close to the theoretical mean proportion of 700 liters per cubic meter of soil. The more detailed

data that are listed in the upper part of Table 5 permit comparison of the composition and properties of the samples recovered in the center and between the centers of columns, and those of treated soil and of ejected soil-grout mixture during the treat-

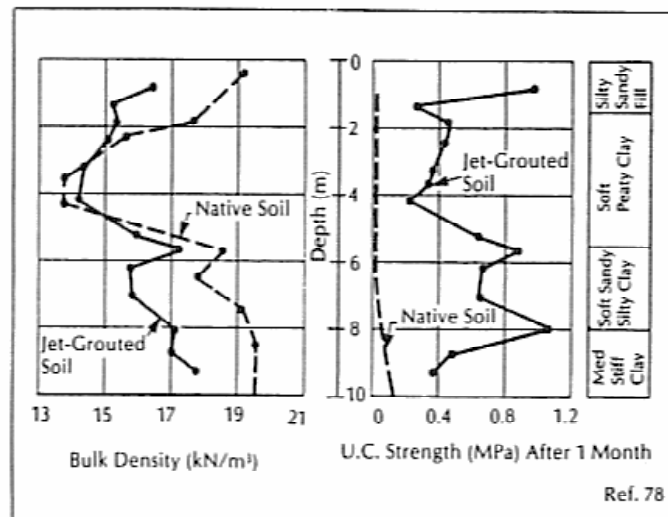


FIGURE 39. Plots of bulk density and strength versus depth (mean values recorded in Singapore test area on samples of jet-grouted soil).

ment. In statistical terms, the strength was above the specified minimum (0.3 N/mm^2) even midway between columns. The treated soil had a water/cement ratio twice that of the pure grout and the ejected mixture was richer in cement and water.

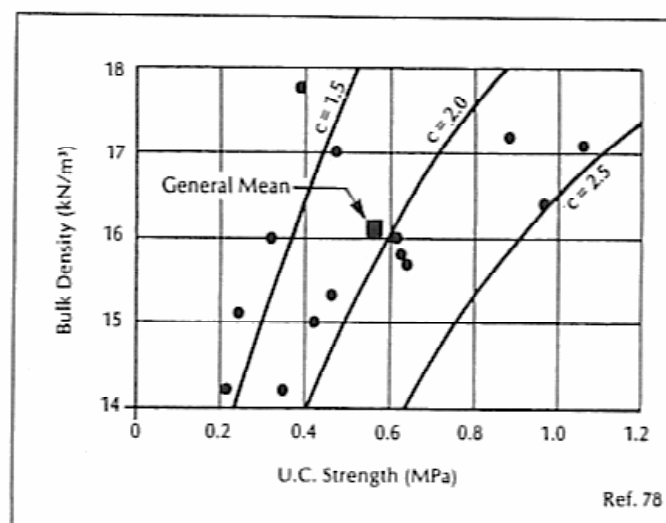


FIGURE 40. Estimate of cement content (kN/m^3) according to laboratory data recorded in Singapore Rodinjet test area.

A test pit was excavated inside the test area 15 days after the end of treatment. There was overlapping of columns with significant discontinuities for Scheme II only. A spacing of 0.7 m was selected for the final design.

The new field trial results were satisfactory

Table 5
Data Recorded During Field Trial & Production Grouting,
MRT Contract 106, Singapore

Field Trial		Unit Weight kN/m^3	U.C. Strength (N/mm^2)	Composition (kN/m^3)					
				C/W	C	dS	W	W_s (%)	V_g (l/m^3)
Jet Grouted Soil	Center Columns	15.91	0.604	0.294	1.93	7.41	6.57	45.3	386
	Between Columns	16.47	0.477	0.261	1.62	8.64	6.21	40.6	324
	Average	16.10	0.559	0.285	1.82	7.83	6.45	43.6	365
Ejected Mixture		14.91	0.641	0.303	2.18	5.54	7.19	(64.3)	435
Actual Treatment									
Jet Grouted Soil		15.31	1.090	0.395	2.76	5.56	6.99	43.0	550
Ejected Mixture		16.14	0.519	0.272	1.75	7.97	6.42	44.0	350

C = Cement W = Water W_s = Water content of native soil V_g = Volume of grout per unit volume of treated soil

General Assumptions: Fully saturated soil & no drainage

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in terms of the quality of soil improvement, but the magnitude of surface upheaval necessitated modification of the operating procedure mainly to increase the volume of ejected material. Further testing led to a satisfactory solution of the problem by various adjustments of the drilling and jet grouting parameters. About 9,400 m³ of soil were subsequently treated by more than 10,000 columns. Ground movements, recorded daily by a close network of datum points, were kept within the safe limits (average 2 cm) by adjusting the sequence of grouting. Systematic control by sampling, laboratory testing and static cone penetration tests were used in monitoring the quality of the work during execution. An excellent performance during the subsequent shield tunnelling was recorded.

Summary

It is a time of major change in the attitudes and demands of the geotechnical construction community in this country. The growing importance of the urban engineering marketplace, and the flood of "new techniques" being introduced from overseas have coincided to the benefit of all the parties involved. While discussion has been limited here to new developments in in-situ reinforcement, minipiling and grouting, it is possible to trace a large number of exciting new developments that can have a major impact.

One of the really attractive features of these developments is the availability of exhaustive and precise field observation data that have been obtained to verify design assumptions on construction and performance. It is to be hoped that the publishing of these data will make practitioners in this country more willing to consider employing the techniques, despite the litigious atmosphere that otherwise gives no encouragement to innovation.

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At Nashville, the excavation contractor was the M.R. Dillard Utility Construction Co., and the geotechnical engineers, Geologic Associates, Inc. The client was St. Thomas Hospital. At Boston, the construction manager was the Perini Corp., and the geotechnical engineers, Goldberg, Zoino and Associates, Inc. The client was Pilgrim Management Trust.



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REFERENCES

1. Nicholson, P.J., "Importing Geotechnical Technologies," Conference on New Technology in Geotechnical Engineering, organized by Central Pennsylvania Section ASCE and PennDOT, Hershey, Pennsylvania, April 14-15, 1987, 10 pp.
2. O'Rourke, T.D., "Are Foreign Engineers Miracle Workers?" *Civil Engineering*, January 1987, p. 6.
3. ASCE Committee on Placement and Improvement of Soils, *Soil Improvement — A Ten-Year Update*, Proceedings of Symposium at ASCE Convention, Atlantic City, New Jersey, April 28, 1987.
4. F.I.P., *Corrosion and Corrosion Protection of Prestressed Ground Anchorages: State of the Art Report*, published by Thomas Telford, London, 28 pp.
5. Transportation Research Board, *Reinforcement of Earth Slopes and Embankments*, NCHRP Report 290, June 1987.
6. Bruce, D.A., and Jewell, R.A., "Soil Nailing: Application and Practice," *Ground Engineering*, Vol. 19, No. 8, pp. 10-15, and Vol. 20, No. 1, pp. 21-32.
7. Bruce, D.A., and Boley, D.L., "New Methods of Highway Stabilization," Proceedings of the 38th Annual Highway Geology Symposium, Pittsburgh, Pennsylvania, May 11-13, 1987, pp. 57-72.
8. Rabcewicz, L.V., "The New Austrian Tunnelling Method: Parts 1-3," *Water Power*, London, December 1964 and January 1965.
9. Hovart, C., and Rami, R., "Elargissement de l'Emprise SNCF pour la Desserte de Saint-Quentin-Yvelines," *Revue Travaux*, January 1975, pp. 44-49.
10. Stocker, M.F., "Bodenvernagelung," *Vorträge der Baugrundtagung*, Nürnberg, 1976, pp. 639-652.

11. Banyai, M., "Stabilisation of Earth Walls by Soil Nailing," Proceedings of the 6th Danube Conference on Soil Mechanics and Foundation Engineering, Budapest, 1984, pp. 459-465.
12. Friedlaender, E.A., personal communication, 1987.
13. Woodward, S., personal communication, 1987.
14. "Sprayed Concrete Wall Cuts Overall Cost by 30% in Underpinning Shoring," *Engineering News-Record*, August 19, 1976, p. 26.
15. Shen, C.K., Herrmann, L.R., Romstad, K.M., Bang, S., Kim, Y.S., and De Natale, J.S., "An In Situ Earth Reinforcement Lateral Support System," Dept. of Civil Engineering, University of California at Davis, Report No. 81-03, National Science Foundation Report No. NSF/CEE-81059, March 1981, 130 pp.
16. Nicholson, P.J., and Boley, D.L., "Soil Nailing Supports Excavation," *Civil Engineering*, April 1985, pp. 45-47.
17. Juran, I., and Elias, V., "Soil Nailed Retaining Structures: Analysis of Case Histories," Proc. Symp. at ASCE Convention, Atlantic City, NJ, April 29, 1987, pp. 232-244.
18. Louis, C., "Nouvelle méthode de soutènement des sols en déblais," *Travaux* 553, March 1981, pp. 67-75.
19. Nicholson, P.J., "Insitu Earth Reinforcement at Cumberland Gap, US 25E," ASCE/PennDOT Conference, Harrisburg, PA, April 17, 1986, 21 pp.
20. P.T.I., *Recommendations for Prestressed Rock and Soil Anchors*, Phoenix, 1986, 41 pp.
21. Louis, C., "Theory and Practice in Soil Nailing Temporary or Permanent Works," ASCE Annual Convention, GEOTECH 1986, Boston, October 27-31, 1986.
22. FHWA, "Corrosion Susceptibility of Internally Reinforced Soil Retaining Structures," Federal Highway Administration Report No. FHWA/RD 83/105, 1985.
23. Juran, I., and Beech, J., "Theoretical Analysis of Nailed Soil Retaining Structures," Int. Symp. on In Situ Soil and Rock Reinforcement, Paris, October 9-11, 1984, pp. 301-307.
24. Schlosser, F., "Analogies et différences dans le comportement et le calcul des ouvrages de soutènement en terre armée et par clouage du sol," *Annales ITBTP No. 418, Sols et Foundations* 184, October 1983, pp. 8-23.
25. Stocker, M.F., Korber, G.W., Gassler, G., and Gudehus, G., "Soil Nailing," Int. Conf. on Soil Reinforcement, Paris, Vol. 2, 1979, pp. 469-474.
26. Gassler, G., and Gudehus, G., "Soil Nailing — Some Aspects of a New Technique," Proceedings of the 10th Int. Conf. Soil Mech. and Found. Eng., Stockholm, Vol. 3, 1981, pp. 665-670.
27. Goulesco, N., and Medio, J.M., "Soutènement des sols en déblais à l'aide d'une paroi hurpinoise," *Tunnels et Ouvrages Souterrains*, Vol. 47, September/October 1981, pp. 9-17.
28. Cartier, G., and Gigan, J.P., "Experiments and Observations on Soil Nailing Structures," Proceedings of the 8th Eng. Conf. Soil Mech. and Found. Eng., Helsinki, October 1983, pp. 473-476.
29. Plumelle, C., special lecture for the French National Committee on Soil Mechanics, 1986.
30. Triplett, R.E., personal communication, 1987.
31. Lizzi, F., *The Static Restoration of Monuments*, Sagep Publisher, 1982, 146 pp.
32. Attwood, S., "Pali Radice: Their Uses in Stabilising Existing Retaining Walls and Creating Cast-In-Situ Retaining Structures," *Ground Engineering*, Vol. 20, No. 7, 1987, pp. 23-27.
33. Lizzi, F., "The Pali Radice (Root Piles)," Proc. Int. Symp. on Soil and Rock Improvement, AIT, Bangkok, November 29 - December 3, 1982, Paper D-1, 21 pp.
34. Dash, U., "Pin-Pile Wall Evaluation," Research Report RR84-36, prepared for U.S. Dept. of Transportation, FHWA under Contract DTFH 71-85-501-PA-14, March 1987, 174 pp.
35. Koreck, H.W., "Small Diameter Bored Injection Piles," *Ground Engineering*, Vol. 11, No. 4, 1978, pp. 14-20.
36. Weltman, A., "A Review of Micro Pile Types," *Ground Engineering*, Vol. 14, No. 3, 1981, pp. 43-49.
37. Bruce, D.A., Ingle, J.L., and Jones, M.R., "Recent Examples of Underpinning Using Minipiles," Proc. Second Int. Conf. Structural Faults and Repairs, London, April 30 - May 2, 1985, pp. 13-28.
38. Johnson, E.G., and Schoenwolf, D.A., "Foundation Considerations for the Expansion and Renovation of the Hynes Auditorium," *Civil Engineering Practice*, Vol. 2, No. 2, Fall 1987, pp. 35-62.
39. Munfakh, G.A., and Soliman, N.N., "Back on Track at Coney," *Civil Engineering*, December 1987, pp. 58-60.
40. Mascardi, C.A., "Design Criteria and Performance of Micropiles," Proc. Int. Symp. on Soil and Rock Improvement, AIT, Bangkok, November 29 - December 3, 1982, Paper D-3, 18 pp.
41. Anon., "Core Drilling Through Concrete Pier Bases Preparatory to Minipiling," *Geodrilling*, August 1987, pp. 16-18.
42. Littlejohn, G.S., and Bruce, D.A., *Rock Anchors — The State of the Art*, Foundation Publications Ltd., Brentwood, Essex, England, 1977, 50 pp.
43. Bjerrum, L., "Norwegian Experiences With Steel Piles to Rock," *Geotechnique*, Vol. 2, No. 2, 1957, pp. 73-96.
44. Mascardi, C.A., *Il comportamento dei micropali sottoposti a sforzo assiale, momento flettente e taglio*, Verlag Leemann, Zurich, 1970.
45. Gouvenot, D., "Essais de chargement et de plambement de pieux aiguilles," *Annales de ITdB et de Travaux Publics*, No. 334, December 1975.
46. BSI, *CP 2004 Foundations*, London, 1972.
47. Plumelle, C., "Amélioration de la portante d'un sol par inclusions de groupe et réseaux de micropieux," Int. Symp. on In Situ Soft Rock Reinforcement, Paris, October 9-11, 1984, pp. 83-90.

48. Lizzi, F., "Reticulated Root Piles — to Correct Landslides," ASCE Convention, Chicago, Oct. 16-20, 1978, Preprint 3370, 25 pp.
49. Jones, D.A., and Turner, M.J., "Post-grouted Micro Piles," *Ground Engineering*, Vol. 13, No. 6, 1980, pp. 47-53.
50. Bruce, D.A., "Aspects of Minipile Practice in the United States," to be published in *Ground Engineering*, July/September 1988.
51. American Society for Testing and Materials, *Testing of Piles Under Static Axial Compressive Load*, D1143-81, 1981.
52. Commonwealth of Massachusetts, *State Building Code*, Fourth Edition, 1984.
53. Ewart, F.K., *Rock Grouting, With Emphasis on Dam Site*, Springer Verlag, New York, 1985, 428 pp.
54. Bruce, D.A., and Gallavresi, F., "The MPSP System: A New Method of Grouting Difficult Rock Formations," Proc. ASCE Convention, Nashville, Tennessee, May 9-13, 1988, and published in ASCE Geotechnical Special Publications No. 14, pp. 97-114.
55. Clough, G.S., "Innovations in Tunnel Construction and Support Techniques," *Bulletin Assoc. of Eng. Geol.*, Vol. 18, No. 2, 1981, pp. 151-167.
56. Tornaghi, R., Bosco, B. and DePaoli, B., "Application of Recently Developed Grouting Procedures for Tunneling in Milan Urban Area," Proc. 5th Int. Symp. Tunneling '88, London, April 18-21, 1988, 11 pp.
57. Baker, W.J., Cording, E.J., and MacPherson, H.H., "Compaction Grouting to Control Ground Movements During Tunneling," *Underground Space*, Vol. 7, 1983, pp. 205-212.
58. Bruce, D.A., and Joyce, G.M., "Slabjacking at Tarbela Dam, Pakistan," *Ground Engineering*, Vol. 16, No. 3, 1983, 5 pp.
59. ASCE Committee on Grouting, "Slabjacking — State of the Art," *Jour. Geol. Eng. Div., ASCE*, Vol. 10, No. 3, G.T. 9, September 1977, pp. 987-1005.
60. Warner, J., "Compaction Grouting — The First Thirty Years," Proc. ASCE Conf. Grouting in Geotechnical Engineering, New Orleans, February 10-12, 1982, pp. 694-707.
61. Coomber, D.B., "Groundwater Control by Jet Grouting," Proc. 21st Reg. Conf. Engr. Group of Geol. Soc., Sheffield, England, September 15-19, 1985, pp. 485-498.
62. FHWA, "Grouting in Soils," Report No. FHWA-RD-76-26, prepared by Haliburton Services, June 1976, 2 Vols.
63. Cambefort, H., "The Principles and Applications of Grouting," *Quart. Journ. Eng. Geol.*, Vol. 10, No. 2, 1977, pp. 57-95.
64. Karol, R.H., "Grout Permeability," Proc. ASCE Conf. Issues in Dam Grouting, April 30, 1985, pp. 27-33.
65. Littlejohn, G.S., *Chemical Grouting*, South African Inst. of Civil Engineering (Geotech. Div.), Univ. Witwatersrand, Johannesburg 4-6, July 1983, 14 pp.
66. Bruce, D.A., "The Drilling and Treatment of Overburden," *Geodrilling*, August & October 1984.
67. Miki, G., and Nakanishi, W., "Technical Progress of the Jet Grouting Method and its Newest Type," Proc. Int. Conf. In Situ Soil and Rock Reinforcement, Paris, October 9-11, 1984, pp. 195-200.
68. Miki, G., personal communication, 1984.
69. ASCE Committee on Grouting, "Preliminary Glossary of Terms Relative to Grouting," *Jour. Geol. Eng. Div., ASCE*, Vol. 10, No. 6, G.T. 7, July 1980, pp. 803-815.
70. Andromalos, K.B., and Pettit, P.J., "Jet Grouting: Snails Pace of Adoption," *Civil Engineering*, December 1986, pp. 40-43.
71. Kauschinger, J.L., personal communication, 1988.
72. Mongilardi, E., and Tornaghi, R., "Construction of Large Underground Openings and Use of Grouts," Proc. Int. Conf. on Deep Found., Beijing, September 1986, 19 pp.
73. Tokoro, T., Kashima, S., and Muraka, M., "Grouting Method by Using Plastic Setting Grout," Proc. ASCE Conf. Grouting in Geotechnical Engineering, New Orleans, February 10-12, 1982, pp. 738-752.
74. Tan, D.Y., and Clough, G.W., "Ground Control for Shallow Tunnels by Soil Grouting," *Jour. Geol. Eng. Div., ASCE*, Vol. 10, No. 6, G.T. 9, September 1980, pp. 1037-1057.
75. ASCE, Baker, W.H., ed., "Issues in Dam Grouting," Proc. Session at ASCE Convention, Denver, Colorado, April 30, 1985, 167 pp.
76. Fairweather, V., "Milan's Model Metro," *Civil Engineering*, December 1987, pp. 40-43.
77. DePaoli, B., Maggioni, F., Tornaghi, R., and Laudiero, C., "Stabilisation of a Peaty Layer for the Construction of a Railway Embankment," Proc. 2nd Baltic Conference on Soil Mech. and Found. Eng., Tallin, USSR, May 1988, 10 pp.
78. Tornaghi, R., and Cippo, A.P., "Soil Improvement by Jet Grouting for the Solution of Tunneling Problems," Proc. 4th Int. Symp. Min. and Met., Tunneling, Brighton, March 10-15, 1985, pp. 265-276.