39th
Highway Geology Symposium
on
"Construction to Minimize Environmental Impact"

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ABSTRACT

In the field of urban construction, engineers are being increasingly forced to resolve difficult problems in situ, and in a timely, cost effective manner. Such problems have been addressed out of necessity in densely populated areas in other continents for many years, and as a result, there is available a wide range of effective and sophisticated techniques from European and Japanese sources. The paper describes salient details of some of these "New Technologies", as applied to minimize the environmental impact of major urban projects involving tunnelling, retaining walls and direct structural support. As the trend towards redevelopment of our major cities gathers momentum, these techniques should have a major impact on all aspects of project performance.

1. BACKGROUND

The contracting skills of an engineering community develop in response to the times and the environment. Many of our 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 transport and sewage projects in both developed and developing countries: Cairo's current resewerage 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 Baltimore'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, and 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 (1987) 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." Furthermore, the competition between the growing number of highly competent specialist contractors is intense, and so cost considerations are 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 (1987) and Nicholson (1987) 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. 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 a net importer of new technologies need not be restated. Rather, the situation can be 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.

This paper introduces a number of newer developments in geotechnical construction which have been fostered by the demands of urban engineering. Six topics are described briefly, for applications in soft ground tunnelling, retaining walls and structural support. The list is not comprehensive, being limited to the direct experience of the author and his company and by the space available. The following techniques are introduced:

A. Soft Ground Tunneling

(i) Premilling
(ii) Grouting

B. Retaining Walls

(i) Soil nailing
(ii) Post grouted anchorages
(iii) Hydromill diaphragm wall excavation

C. Structural Support

(i) Minipiling

The range of techniques and applications is great, but they do share the common goals of minimizing environmental impact, and optimizing construction time and safety.

2. NEW TECHNIQUES IN SOFT GROUND TUNNELLING

2.1. Premilling

Papers describing the successful development of "predecoupage mecanique" (Premill) as a specialized tunnelling technique in rock and soils have appeared sporadically in the French technical press over the last few years, and a review was presented recently by Bruce and Gallavresi (1988).

In essence, the system comprises a track-mounted frame, of shape corresponding to the tunnel extrados (Figure 1). Mounted on the frame and projecting out in front is a large band saw type milling machine about 3m long. This can be moved around the frame to cut a slot about 120-200mm wide into the ground around the volume to be excavated. In competent rock this slot is left open, to optimize the subsequent blasting parameters and performance. In soft ground the slot is filled immediately with high strength, fast setting concrete, so forming an insitu arch to minimize decompression effects during subsequent excavation.

The system was evolved in response to the need to absolutely minimize construction related effects in urban areas involving large diameter tunnels close to the surface under old and delicate structure. Under such conditions even the excellent performance afforded by the standard New Austrian Tunneling Method was not acceptable. The early examples for railroad and Metro construction in France have been followed by similar contracts for example in Belgium, Spain, and Italy.

In competent rock formations of up to 250 bar (3500 psi) compressive strength, the premill is used only to provide a continuous slot around the volume to be excavated - usually with a drill and blast method. Premilling provides the following major benefits:

- less explosives (and blast holes) are required, rendering the entire blasting operation safer, faster and environmentally more acceptable.

- fissuring or decompression occurs in the surrounding rock mass, thus preserving its virgin properties, and so reducing the demand for subsequent reinforcement, e.g., with bolts.

- there is no overbreak and, therefore, there are associated cost savings in time, effort and materials.

- the smooth profile makes the placing and performance of arches more efficient.

- less contact or consolidation grouting is needed behind the final tunnel lining.

- following blasting, there is a greatly reduced danger from rock falls due to chimneys of fractured ground developing above the excavation.

- the magnitude of vibrations transmitted upwards towards nearby surface structures is greatly attenuated.

Developments of the technique continue, for example, in special diamond tools, high pressure water jetting, and increased cutting power, to permit its use in harder rock formations, faster, and with increased safety.

In soft ground as noted above, the major difference is that the cut slot is filled with a special concrete mix as early as possible. The advantages are a identified above for the rock premill, although the prime target is the elimination of surface settlements induced by the tunnelling.

Each cover, up to 3.5m long, depending on the soil, is inclined slightly outwards and overlaps the preceding one by 300 to 500mm. The cone is cut in discrete segments so that the concrete can be placed in each segment, as early as possible and without having to wait for the whole arch profile to be first completed. Cutting times for a typical 3m long segment may be as low as one minute.

The concrete may be placed by dry or wet shotcrete methods. A typical mix reported by Bougard et al (1979) comprises, per cubic meter of mix:

Cement	450Kg	Coarse gravel	650Kg
Sand	560Kg	Accelerator (Sigunite)	27Kg
Fine gravel	650Kg	Water, as appropriate, typically w/c = 0.25 - 0.30	

This gives a strength of up to 100 bar (1400 psi) at eight hours. Spraying th mix into the premilled slot ensures that none of the fine aggregate is lost, is the case in conventional NATM applications of shotcrete on open faces. Th concrete in place is, therefore, of superior quality, further enhancing the performance of the system.

In comparison with the NATM, there are certain similarities, notably the overall concept of the support, and the common construction elements such as shotcrete, bolts, and arches. However, the major dissimilarity is that with premilling the primary lining is placed up to 3m ahead of the face before excavation, whereas in NATM the lining follows 1 or 2m behind the excavated face. This greatly impacts the generation and scale of tunnel deformations, and so the effect on overlying structures. Goer (1982) described a monitored case history of the relative performance of the two methods in the same material - Argenteuil marl (Figure 2). Typical properties of this material were listed as:

Density = 2
Effective ϕ = 20^0
Effective cohesion = 0.5 - 1.0 bar
Undrained cohesion = 1.5 bar
Deformation modulus = 500 bar

Three times less settlement was achieved in the tunnel protected by premilling.

Most of the earlier applications have been carried out with the conservative "divided section" profile. However, excellent results with the "full section" profile (i.e., cutting a 270^0 arc and excavation in one pass) in a shallow circular collector tunnel 3.50m diameter (Departement de la Seine-Saint-Denis, France), in very difficult ground, encouraged its use in Lot 7 of the Lille Metro, Belgium. As evident in Figure 3 the performance of the full section profile was superior, with surface settlements no more than 1mm. Prefabricated base slabs were connected structurally to the premill cover by shotcrete, and the steel ribs then placed, bearing on the slabs. This system also proved faster than the divided section approach.

2.2. Grouting

Ground treatment by grouting is hardly a new technique: Charles Berigny repaired a harbor sea lock in Dieppe, France, in 1802 using basic cement grouting techniques, whilst the first major US application dates from 1910 and shaft sinking for the Catskill Aqueduct, New York. Applications in dam foundation sealing and strengthening were for many years the principal American market. However, dam grouting in this country stultified as a direct consequence of restrictive construction practices and unimaginative contractual procedures. As a result, our industry could not cope with the geotechnical challenges issued by dam construction on the less favorable sites left in the last twenty or thirty years. Consequently, major projects were completed wherein the standard of the grouting executed was, simply, poor and the effectiveness highly questionable. Such imperfections soon became common knowledge - although the causes remained uninvestigated: the word soon emerged that grouting "doesn't work."

Today major Federal dams are being repaired against seepage around or under the structure by using concrete diaphragm walls - often constructed by the hydromill excavator, described below. This approach can guarantee an efficient cutoff, but at significant financial premium over grouting, properly executed.

This poor opinion of grouting as an engineering tool has permeated the thinking of those involved in tunneling and deep foundations too, to the extent that grouting is still regarded by many as a last resort - to be attempted when everything else has failed. Although understandable, this attitude is patently unfair and wholly unjustified, in the light of major strides made in the last decade on the execution and control of grouting works.

In rock grouting, significant advances have been made in techniques (e.g. the MPSP system of treating difficult rock masses - Bruce and Gallavresi, 1988) and in materials, (e.g. the use of microfine grouts to penetrate fine fissures Karol, 1985).

Soil grouting is in an even more dynamic situation, benefitting rapidly from technological advances made by chemists, physicists, instrumentation engineers, and geotechnicians. Many of these developments have been associated with tunneling in urban areas, principally for subway or sewer projects. The aim has been to provide better aids to speed progress, improve safety and minimize associated settlements. This aim has been so efficiently achieved in Europe and the Far East that grouting is there incorporated routinely, abinitio, as an integral part of the process.

Bruce and Boley (1987) summarized four categories of soft ground grouting (Figure 4), and for most purposes only the following three have validity for work in urban areas, in U. S. practice.

o Compaction grouting is a specialized "uniquely American" process that has been used since the early 1950s (Baker et al, 1983). Very stiff soil-cement mortar is injected at high pressures (up to 35 bar (500 psi)) 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 Joyce, 1983) and, indeed, the earlier applications were used exclusively for levelling slabs and light buildings on shallow foundations (Warner, 1982). 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 (Baker et al, 1983). 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 devleoping at the source. Although compaction grouting has practical 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 is then necessary.

The techniques involved in permeation grouting are the oldest and best researched. 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 5). Excellent reviews of the subject are provided by the FHWA (1976), Cambefort (1977), Karol (1983), and Littlejohn (1983). Other permeation grouting methods, principally from Japan, are described in an earlier study (Bruce 1984).

o Replacement grouting is the youngest major category of ground treatment. According to Miki and Nakanishi (1984), 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. Its development was fostered by the need to thoroughly treat soils from gravels to clays to random fills in areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements. As indicated in Figure 5 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 (1980) define 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 pumping grout through the horizontal nozzles that jets and mixes with foundation material as the drill bit is withdrawn." Figure 6 depicts one particular type in which the soil is jetted by an upper nozzle ejecting water at up to 600 bar (8400 psi) inside an envelope of compressed ai at up to 12 bar (170 psi). The debris is displaced out of the oversized hole by the simultaneous injection of cement-based grout through a lower nozzle (up to 70 to 80 bar). Other simpler variants utilize only grout jetting alone to simultaneously erode and inject, giving much more of a "mix in place" action.

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.

In permeation grouting major new trends are evident in

(1) Methods - e.g. powerful diesel hydraulic drilling rigs capable of drilling quickly in restricted conditions through difficult ground to depths of over 60m if necessary.

(2) Materials - new families of stable, microfine cement-based grouts, and high-strength low viscosity chemical grouts which do not creep or synerise (Tornaghi et al 1988).

(3) Instrumentation and Control - Throughout the grouting industry, the use of computer-aided devices as monitors and controls over grouting operatio 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. 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 has bee

developed in Italy. This system displays 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 (Fairweather, 1987).

Most recently, however, a major break-through 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 100mm intervals (see Figure 7). This geological log permits optimization of 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 6m 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.

Similar advances are being made in jet grouting to the extent that major Metro tunnels have been constructed through soft marine clays in Singapore (Mongilardi and Tornaghi, 1986) and railway embankments have been founded on highly compressible peats, stabilized by grouting (De Paoli et al 1988).

In this country, perhaps about 50 projects involving jet grouting have been completed to date by a handful of specialists. If some of these projects have not had entirely satisfactory outcomes, this reflects the decision to employ the technique for the wrong application, and occasionally it reflects the inexperience of the operators and engineers involved.

Nevertheless, it can only be concluded that the significant advances made in grouting over the past ten years offer U. S. engineers an extremely versatile, controllable and effective tool in minimizing environmental impact in urban construction.

3. NEW TECHNIQUES IN RETAINING WALLS

3.1. Soil Nailing

Soil nailing is one of the family of insitu soil reinforcing techniques summarized by Bruce and Jewell (1986-1987). It comprises steel reinforcing elements grouted into horizontal or sub-horizontal holes drilled into the cut face of the excavation as it proceeds downwards in stages. The inserts improve the shearing resistance of the soil by being forced to act in tension. They clearly differ from the nature and mode of action of the other members of the family (Figure 8), namely reticulated micropiles and large diameter soil dowels.

The history of development in the three principal countries of origin, namely France, Germany and the United States, is fascinating and encompasses almost two decades of often erratic progress. Suffice it to note that today soil nailing is one of the fastest growing geotechnical construction techniques within North America, and has enjoyed a boom since the execution of the foundation excavation for the PPG Building in Pittsburgh, Pennsylvania, in 198[illegible] (Nicholson and Boley, 1985).

This upsurge in interest is reflected in the publishing of the National Cooperative Highway Research Program (NCHRP) Report 290 in 1987 on all types o[illegible] insitu reinforcement and the attention paid to it at the ASCE Atlantic City Ten-year Update Symposium, also in 1987. 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 in 1989.

The standard sequence of construction is illustrated in Figure 9 - basically it is a cycle of excavate-spray with shotcrete - nail, and so on. Detailed case histories are provided by Louis (1987) and Bruce (1988) amongst others. Special attention must be paid to drainage of the face and soil mass during an[illegible] after construction, and to corrosion protection of the inserts in permanent applications.

Design is complex and there seems to be no "best" method, although the kinematical limit analysis proposed by Juran and Beech (1984) seems consistent with observed structural performance.

There is a wealth of experimental and construction data on the performance of soil nailed excavations in urban settings. Most significantly, Juran and Elia (1987) concluded that post-construction observations in non-plastic soils have shown that after the end of construction, ground movement and facing displacement (maximum at the top) do not typically exceed 0.3% of the total excavation depth and one rapidly stabilized. This puts soil nailing very firmly in Pecks (1969) Category I of excavation performance.

Soil nails differ from ground anchorages in several aspects:

a) Nails are fully bonded to the soil over their entire length.
b) Nails are not prestressed.
c) Nails are relatively closely spaced e.g. 1 per 1.5 to 2.5 square meters of face, to generate the soil mass-insert interaction.

d) High loads are not transmitted to the head of the nail at the cut face. Therefore, there is no need for elaborate bearing plate arrangements.
e) Nails are shorter (typically 50-100% of excavation depth depending on the soil) and so need only relatively light and mobile drilling equipment capable of drilling up to 150mm diameter holes. However, if overall stability calculations indicate a problem to be deep seated then prestressed anchorages will most probably be required either as the only retaining element or compositely with the nailed structure.

It is important to note well the benefits and limitations. In the former category, there are

- Economic advantages: where nailing is possible technically, it is typical to find that the cost savings for excavations on the order of 10m deep are 10 to 30 percent, relative to the use of an anchored diaphragm or Berlin wall. These projected savings are supported by the reported savings of 30 percent on a soil nailed excavation in Portland, Oregon, (ENR) 1976.

- 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 (1981), nailed structures can withstand both static and dynamic surcharge loading without excessive settlements if properly designed.

Major practical limitations are:

- Soil nail construction requires the formation of cuts generally 1 to 2m 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 ground water 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.

3.2. Post-grouted Anchorages

Prestressed ground anchorages have been employed throughout the world since 1934. They were introduced into the States during the 1960s and were used initially only as temporary excavation support, although many of the applications were in major urban excavations in very variable soil (Figure 10). Such ground conditions may give rise to very low grout/ground bond values or significant creep amounts in service. In addition, erratic anchor behavior can occur, due to subtle local variations in the soil.

To increase bond capacity (and, therefore, reduce anchor length and cost), to reduce creep, and to regularize anchor performance in such soils, post-grouted, or regroutable, anchorages have been developed in Western Europe. Their relationship to conventional anchorage types is best illustrated in the British Code of Practice BS 8081 which defines four types (Figure 11).

Type A anchorages: consist of tremie (gravity displacement), packer or cartridge grouted straight shaft boreholes, which may be temporarily lined or unlined depending on hole stability. This type is most commonly employed in rock and very stiff to hard cohesive deposits. Resistance to withdrawal is dependent on side shear at the ground/grout interface.

Type B anchorages: involve low pressure (typically grout injection pressure $p_i < 10$ bar (140 psi) grouted boreholes, where the diameter of the fixed anchor is increased with minimal disturbance as the grout permeates through the pores or natural fractures of the ground. This type is most commonly employed in weak fissured rocks and coarse granular alluvium, but the method is also popular in fine grained cohesionless soils. Here cement-based grouts cannot permeate the small pores but under pressure the grout compacts the soil locally to increase the effective diameter and enhance the shearing resistance. Resistance to withdrawal is dependent primarily on side shear in practice, but an end bearing component may be included when calculating the ultimate capacity.

Type C anchorages: feature boreholes grouted to high pressure (typically $p_i > 20$ bar (280 psi)), via a lining tube and packer (i.e. sleeved pipe system). The bond zone is enlarged by hydrofracturing of the ground mass to give a grout root or fissure system beyond the drilled diameter of the borehole after initial stiffening of primary grout placed as for Type B anchorages (Figure 12). A relatively small quantity of secondary grout is needed. Continuous flow or a sudden drop on initial injection pressure might indicate hydrofracture after which only relatively limited pressures can be achieved.

Post-grouted anchorages of this type are commonly applied in fine cohesionless soils, whilst increasing success has also been achieved in stiff cohesive deposits. Design is based on the assumption of uniform shear along the fixed anchor.

Type D anchorages: consist of tremie grouted boreholes in which a series of enlargements, either bells or underreams, have previously been formed. This type is employed most commonly in firm to hard cohesive deposits. Resistance to withdrawal is dependent on side shear and end bearing, although, for single or widely spaced underreams, the ground restraint may be mobilized primarily by end bearing. Such anchorages are becoming less popular, having been superseded by the post-grouted types.

For the design of post-grouted anchorages in cohesionless soils, calculations are based on design curves created from field experience in a range of soils rather than relying on a theoretical or empirical equation using the mechanical properties of a particular soil. In alluvium, for example, test results (Jorge 1969) have indicated in boreholes of 100 to 150mm diameter, ultimate load holding capacities 9 to 13 tons/m of fixed anchor at a grouting pressure of 10 bar (140 psi) and 19 to 24 tons/m at a pressure of 25 bar (350 psi).

In more recent years, design curves for post-grouted anchorages have been extended through proving tests in Germany (Ostermeyer, 1974). For sandy gravels and gravelly sands, it has been found that the ultimate load increases with density and uniformity coefficient. The results of a large number of fundamental tests are shown in Figure 13, which can be used as a design guide for borehole diameters of 80 to 160mm. Skin friction increases with increasing consistency and decreasing plasticity. The technique of post-grouting is also shown to generally increase the skin friction of very stiff clays by some 25% to 50%, although considerably greater improvements are claimed for stiff clay of medium to high plasticity. Ostermeyer also found that there was a steady increase in skin friction as the post-grouting pressure was increased up to 30 bar (420 psi).

3.3. Hydromill Diaphragm Wall Excavation

Diaphragm walling (slurry trenching) is another technique which can hardly be described as new to these shores. For example, Saxena (1974) described the construction of the massive wall built for the World Trade Center foundation excavation in New York in 1968/69. There are a number of reputable specialist companies operating throughout the country, although most of the activity has so far been in the Northern and Eastern States.

The principle of the technique is well-known: a trench, typically 600-1000mm wide and 2.5 to 3m long, is excavated vertically by grab, to the required depth. The trench is maintained filled with bentonite slurry to keep it open during excavation. After excavation is complete a steel reinforcing cage is usually placed in the panel and the bentonite slurry then displaced out of the trench by concrete tremied into the trench from the base up. In this way, insitu reinforced concrete panels are formed in the ground. During construction of the wall, alternate panels are excavated and cast (Primaries). When concrete has reached a certain strength, the intermediate (Secondary) panels are formed contiguously, thus forming a continuous wall in the ground.

Removal of the soil on one side of the wall follows, leaving the diaphragm exposed, to act as a retaining wall or as the support of a deep excavation. Most commonly prestressed anchors are installed through the wall to provide shoring.

Excavation is carried out by a clamshell bucket. either rope suspended or operated from a KELLY bar. Diaphragm walling in this way has always experienced problems with:

- The presence of major boulders.
- The difficulty of "toeing" into rock.
- Making joints between adjacent panels watertight.
- The handling and disposal of bentonite slurry and waste material - especially in city sites.
- Restricted depth capacity (say 50m).

In addition, the drive for increased productivity and the need to have a technological 'edge' on the competition are always constant spurs to developments.

Less than 15 years ago, the hydrofraise excavating machine began to be developed in France. (Figure 14). Called the hydrofraise by the French specialists Soletanche, it comprises a steel frame also serving as a guide, on which are mounted two cutting wheels. hydraulically powered. and rotating in opposite directions. The wheels have tungsten carbide teeth. A third down-the-hole motor operates a reverse circulation mud pump located above the wheels which carries the excavated debris. in the bentonite slurry. to the surface. The "dirty" mud is cleaned there and returned out the top of the trench. The guide frame is attached to the crane operated cable from which it is suspended by a hydraulic feed cylinder which can be controlled a) to give a constant rate of advance or b) to maintain a constant weight on the cutters (16-20 tons maximum).

The cutters can readily chew into the concrete of adjacent primary panels thus ensuring excellent joint properties. They can penetrate all kinds of soil and rock with compressive strengths of up to 1000 bar (14.000 psi). The absence of vibration and shocks, plus the self-contained debris and slurry handling system makes it ideal for urban sites. Overbreak is also less than for conventional systems (less than 10%) and verticality can be controlled and corrected to less than 0.2% if required.

Standard panel sizes are shown in Figure 15. About 20% of the 600.000 square meters excavated by hydrofraise throughout the world has been conducted in the US, with the greatest proportion being on three major Federal dams. On one - Navajo Dam, New Mexico - a world record depth of almost 120m was reached, including up to 60m in bedrock (Fairweather. 1987). Other excellent and typical case histories include shafts for the Channel Tunnel. France (Evers and Hovart 1988) and deep excavations for a nuclear power station in England (Anon 1988) and for Baltimore-Harbor Place Building (Soletanche 1987).

Production rates can be very high and quoted figures run from 120 square meters/shift in tough alluvium and hard limestone in Paris (Soletanche. 1980). to 40 square meters/hour in dense sands and clay in England (Anon. 1988).

The hydromill is an expensive machine to buy and operate but has clearly several major advantages over conventional diaphragm walling methods. Its speed reduces unit costs, and any cost premium still remaining can often be offset against the environmental bonuses it offers.

4. NEW TECHNIQUES IN STRUCTURAL SUPPORT

4.1. Minipiling

It is now almost 40 years since the technique of minipiling was first applied in Italy (Koreck 1978, Weltman 1987, ASCE 1987). Following the lapse of the early patents, there has been a tremendous growth in the market volume particularly in the cities and industrial centers of W. Europe and S. E. Asia. In this country the start was much later but the expansion has been equally dramatic in the last five years or so, as rebuilding and redevelopment of our older cities picks up momentum. Indeed it is the author's observation that there is probably a greater intensity of minipile activity in Boston, Massachusetts, and New York City than in any other cities in the world. (Bruce 1988)

Minipiles are cast in situ bored piles rarely more than 300mm in diameter and 30m deep. A fundamental feature is their ability to be constructed by equipment of the type used for anchoring and grouting works, as opposed to that needed for conventional bored or driven piles.

Minipiles can be constructed to considerable depths through all types of soil, rock and obstructions, and in virtually any direction. They have a high slenderness ratio and so transfer load almost wholly by shaft friction, eliminating any requirement for underreaming at the base to enhance end bearing. All feature substantial steel reinforcing elements and so can sustain axial loading in both senses. The reinforcement can also be designed to resist bending stresses safely and with minimal displacement.

The construction steps (Figure 16) are characterized by equipment ensuring minimum vibration, ground disturbance and noise, and capable of operating efficiently in awkward and restricted access and working conditions. Thus, although their nature may result in them being lineally more expensive than conventional driven or large diameter piles, they may be the only guaranteed solution given a particular set of ground, site, program and performance conditions.

Regarding their service behavior, minipiles exhibit relatively high carrying capacity (for their diameter) and very small settlements. Piles installed wholly in soils can be constructed to provide safe working loads approaching 100 tons, whereas recent work in Boston (Johnson and Schoenwolf, 1987) and New Jersey, (Bruce 1988) shows that when founded in rock, safe working loads two or three times that figure can be sustained.

Load holding capacity can be improved substantially by the post-grouting techniques described earlier as used for anchors. Settlements to structures being underpinned can be almost eliminated by preloading the piles to working load - by prestressing - so that no further pile movement occurs when the structural load is finally applied.

Another very significant feature involves the consideration of interpile spacings with respect to pile group performance. For example, the British Code of Practice (CP2004, 1972) states that for "friction piles, the spacing center to center should not be less than the perimeter of the pile." On the other hand, test information (Plumelle, 1984) shows that closely spaced minipiles, especially when inclined, interfere positively, as illustrated in Figure 17. Undoubtedly there is a soil structure reaction, as in insitu reinforcing, which is being exploited with excellent results in tunnel related applications (Figure 18).

Overall, therefore, minipiles are an excellent option for upgrading or replacing existing foundation to sustain increased structural loadings, or to help them resist additional settlements arising from adjacent new constructions such as tunnels or deep excavations. They are also finding increasing application as support for new foundations bearing on very difficult geologic profiles which would render conventional piles or caissons exceptionally difficult or very costly to install.

5. FINAL REMARKS

These brief introductions to various ground engineering techniques which have been developed primarily for construction in urban areas highlight that we have considerable power at our fingertips. These techniques cannot be described merely as "having potential" - their potential has been realized and exploited in the countries of origin to the extent that in this country we now have access to tried and proven systems of major relevance.

It can only be hoped that practitioners in this country will be more willing to employ "new technologies" than they have generally been in the past. It is recognized, however, that such innovation must be pushed through in the face of our litigious atmosphere which certainly gives no encouragement in this respect

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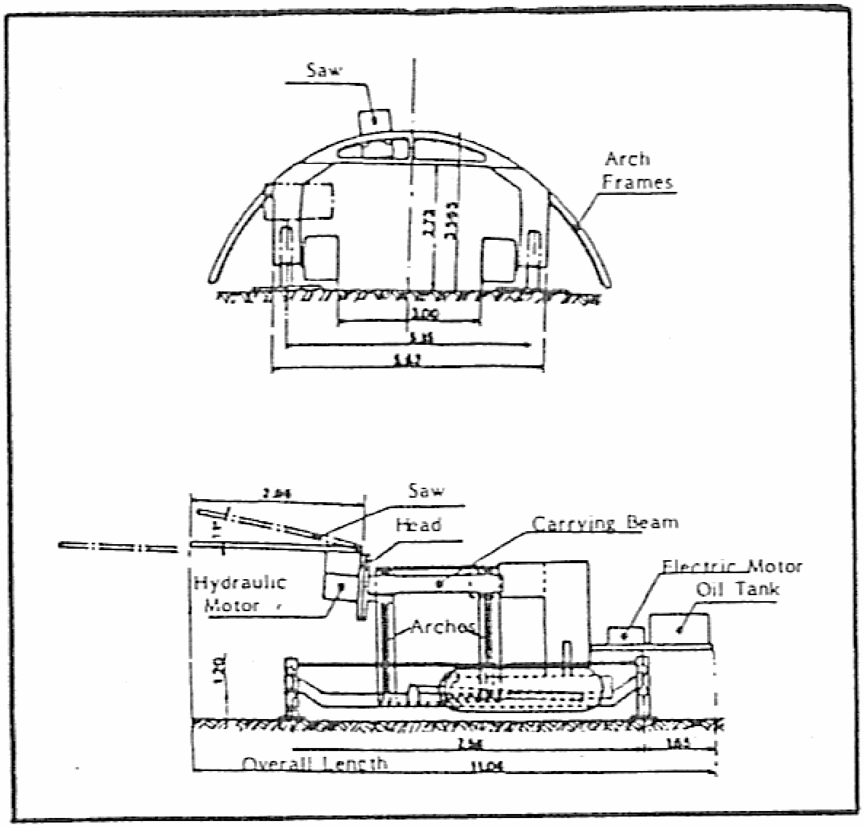


Figure 1. General layout of typical premill machine (Hydraulic arm for shotcreting not shown).

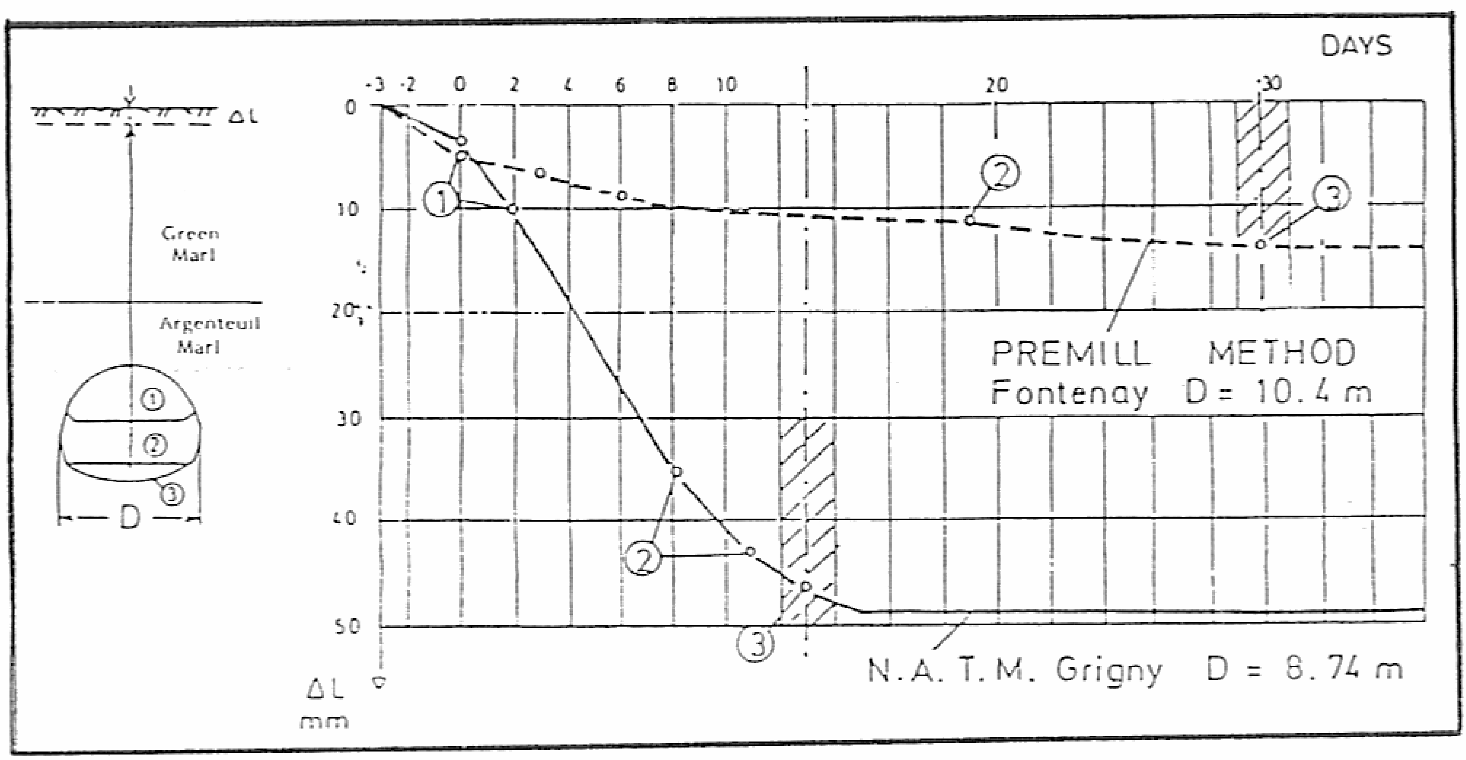


Figure 2. Comparison of settlements generated with time in identical geological conditions, by Premill and NATM. (LeGoer, 1982)

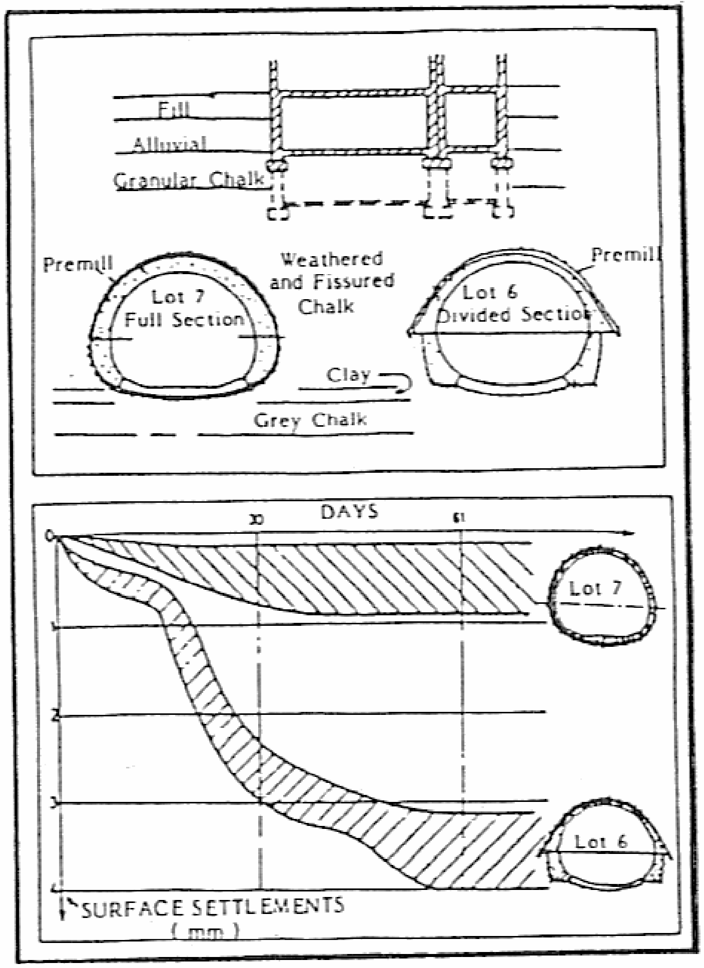


Figure 3. Comparison of settlements generated by Premilling in Divided Section and Full Section, Lille, Belgium (LeGoer, 1982)

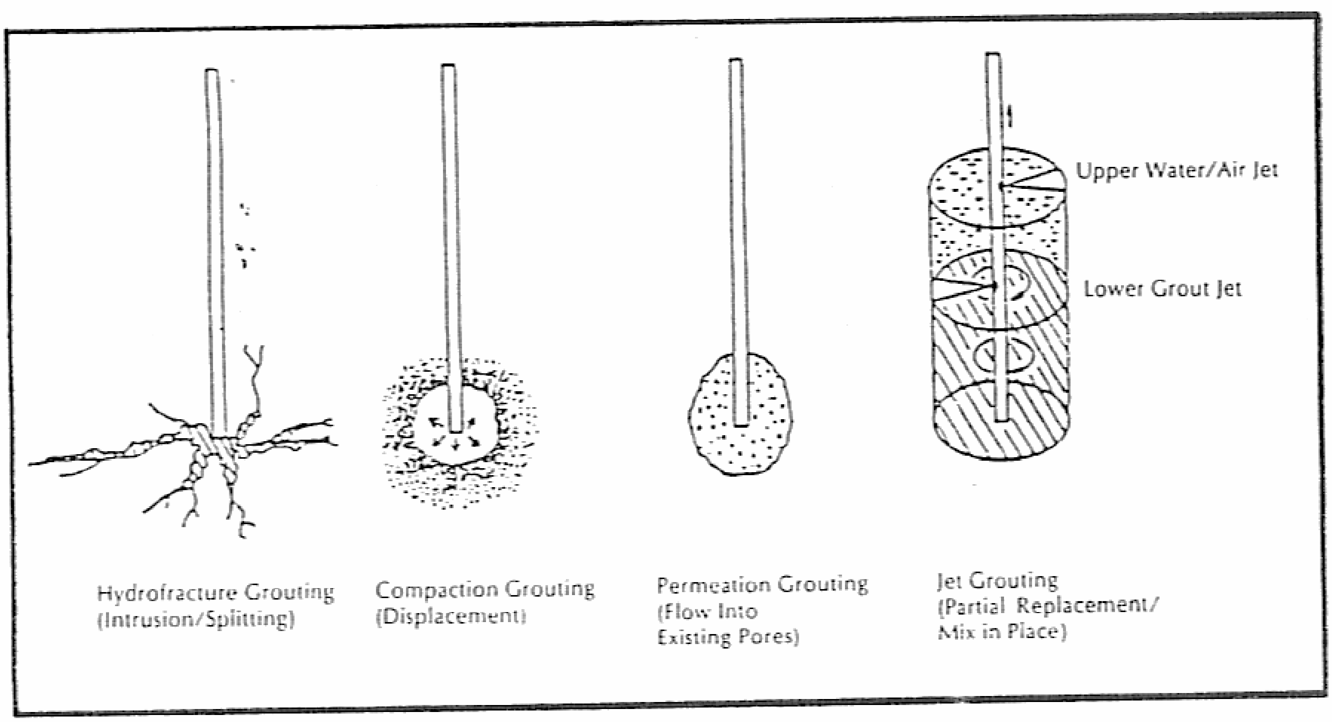


Figure 4. Basic categories of soil grouting.

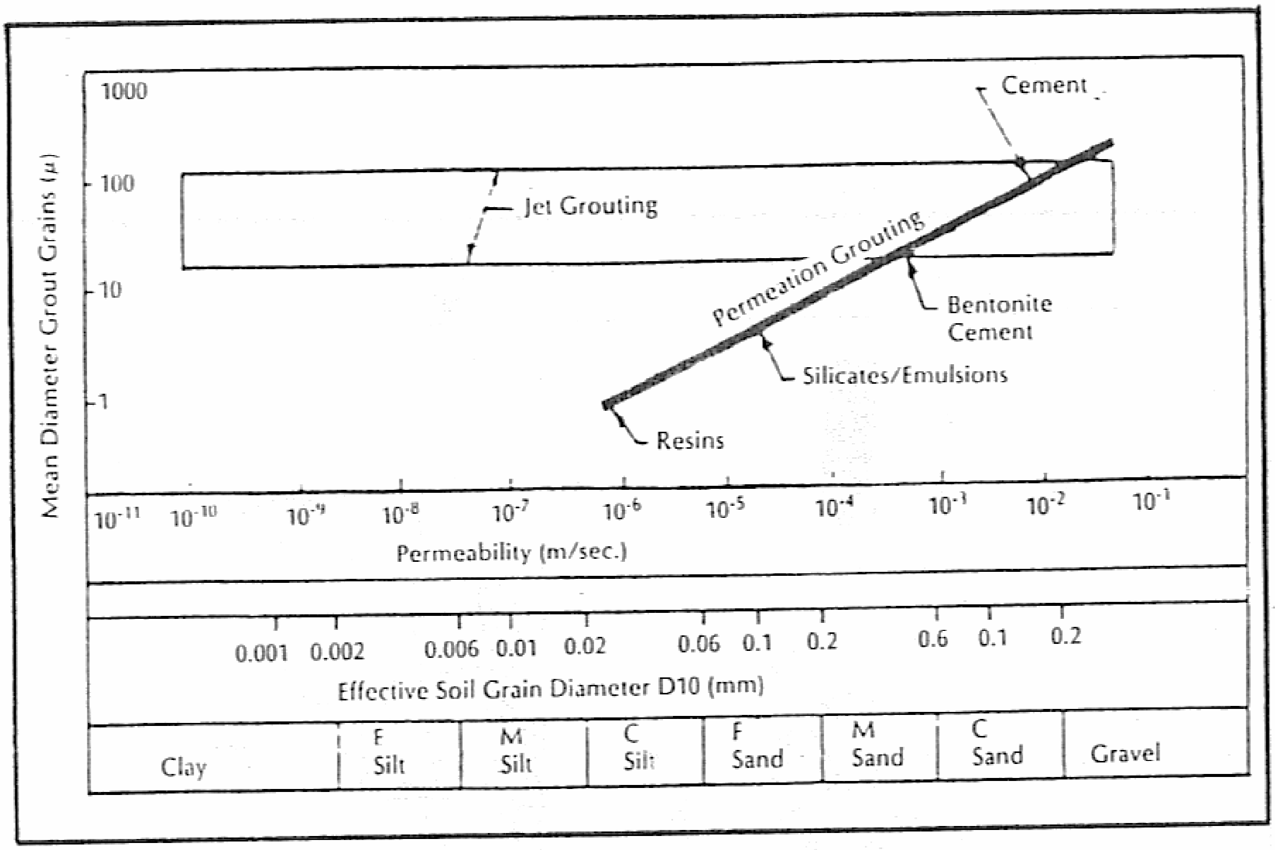


Figure 5. Groutability of soils in relation to grout and soil properties (After Coomber, 1985)

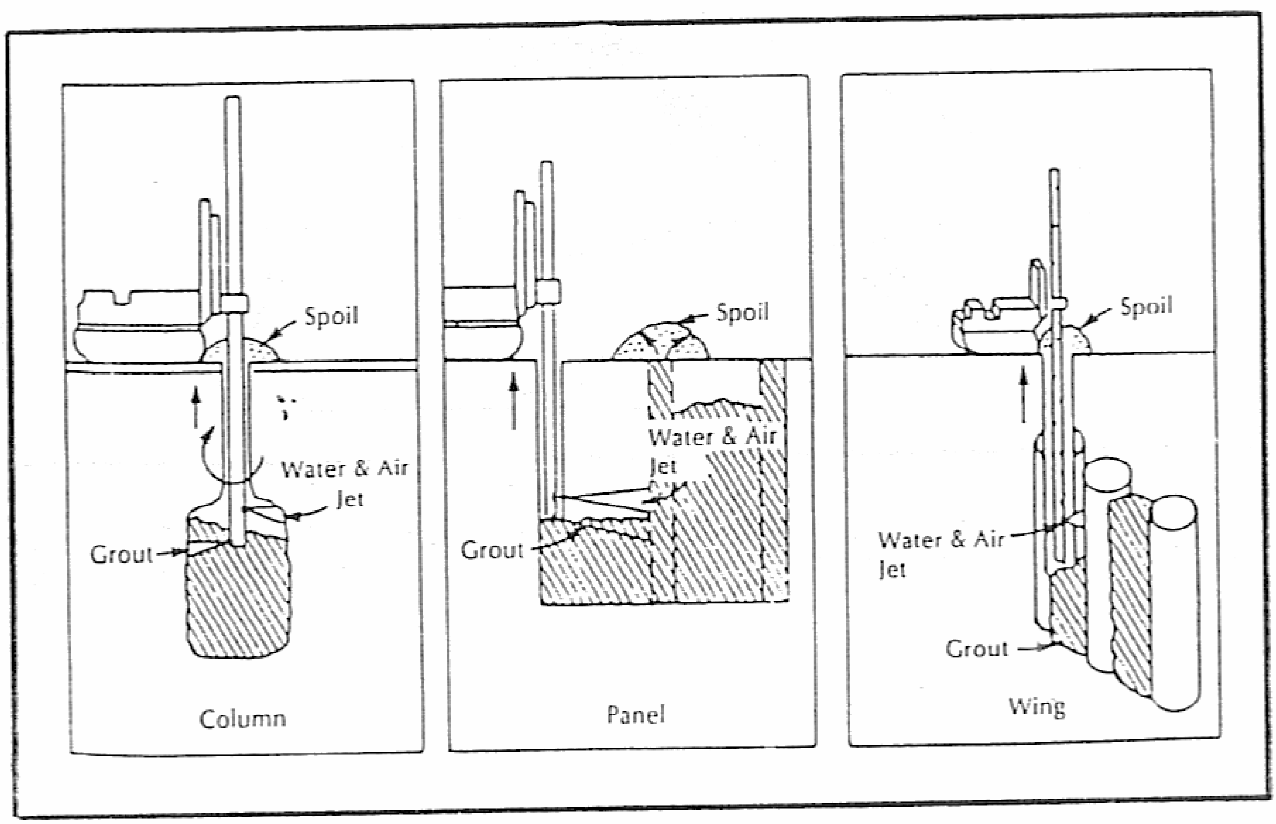


Figure 6. Jet grouting options using the three fluid system (i.e. air, wa
and grout) (After Coomber, 1985)

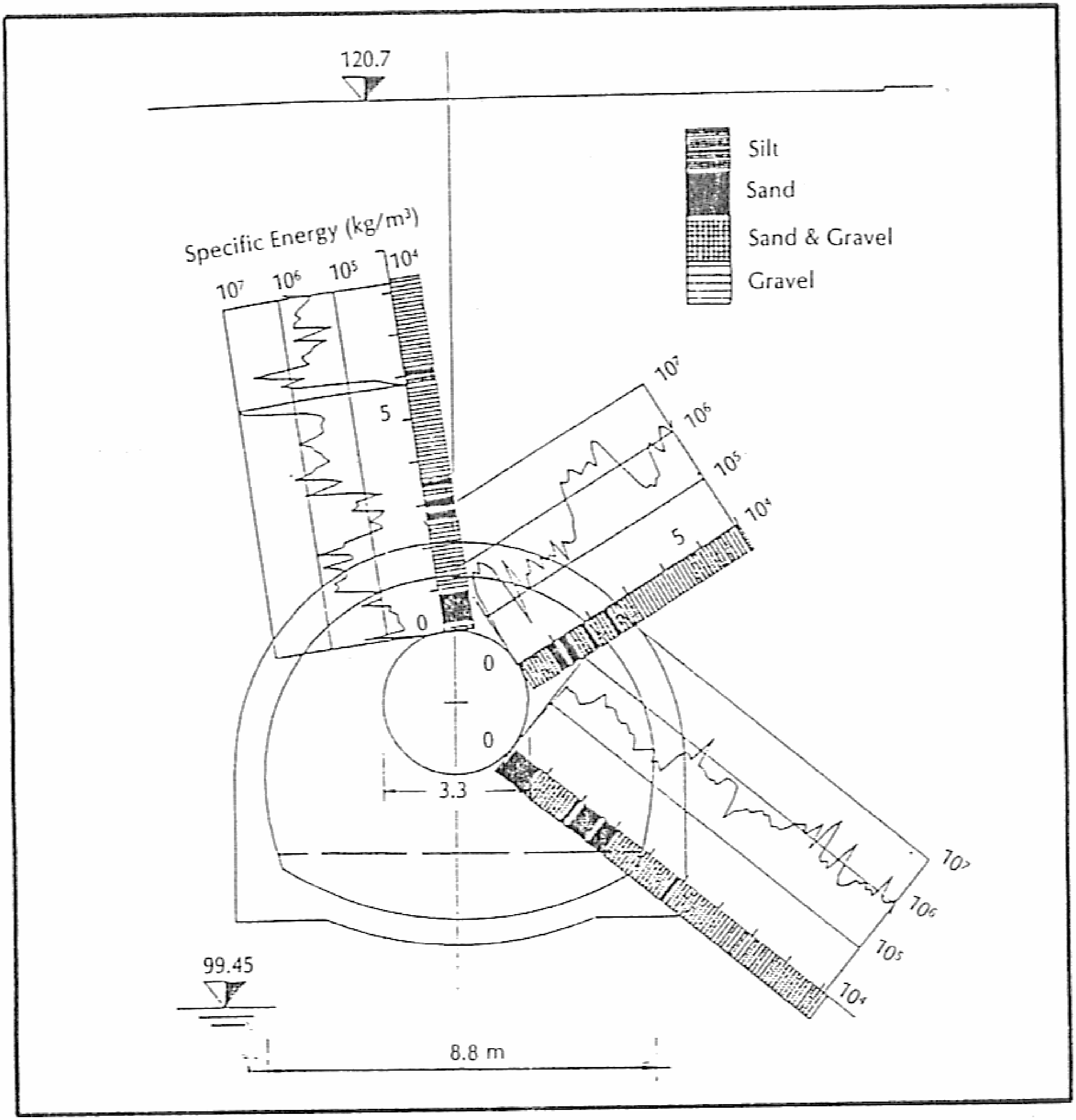


Figure 7. Soil profiles derived from the evaluation of electronically recorded drilling parameters (PAPERO) in terms of specific energy (Milan M Line 3)

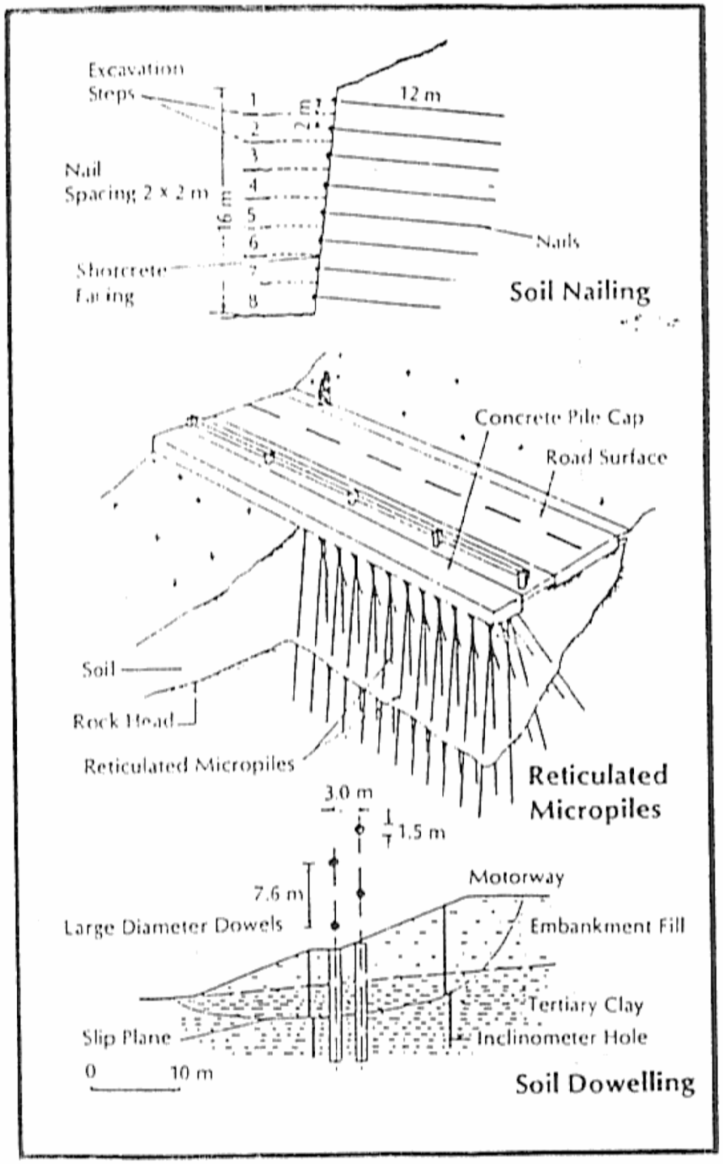


Figure 8. The family of insitu soil reinforcing techniques (Bruce and Jewell, 1986)

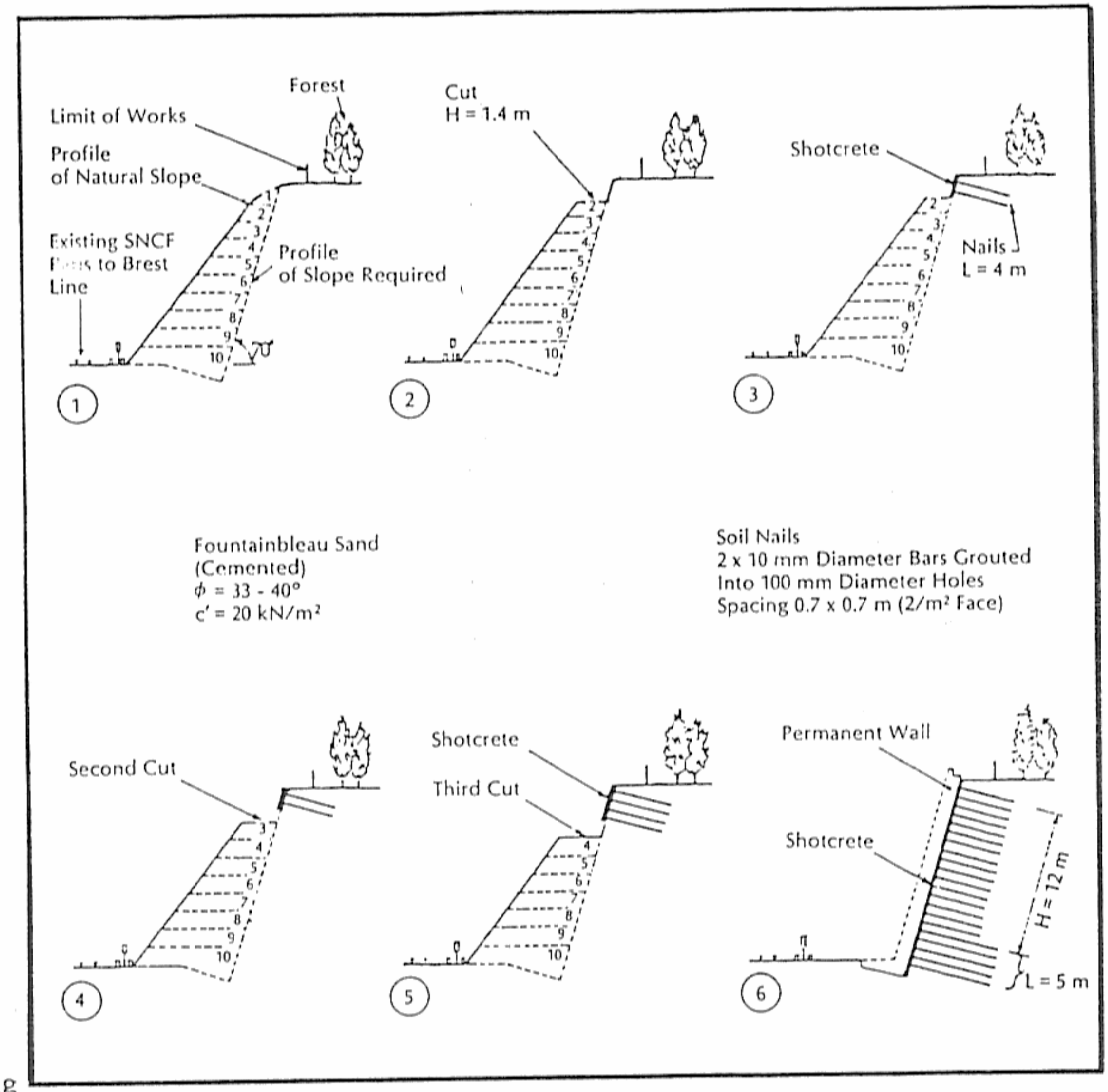


Figure 9. Standard sequence of soil nail construction, as used at Versailles, France.

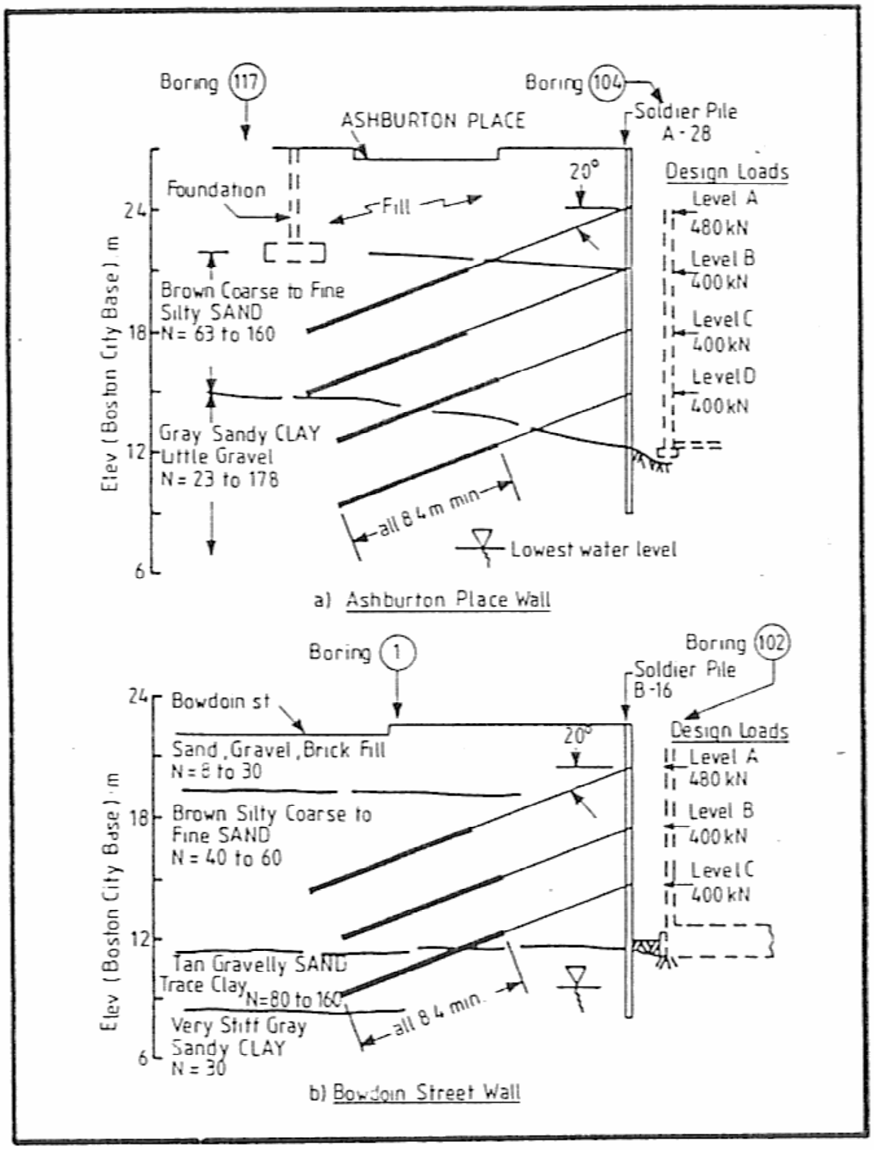


Figure 10. Typical anchored retaining wall, Boston (Oosterbaan and Gifford 1972)

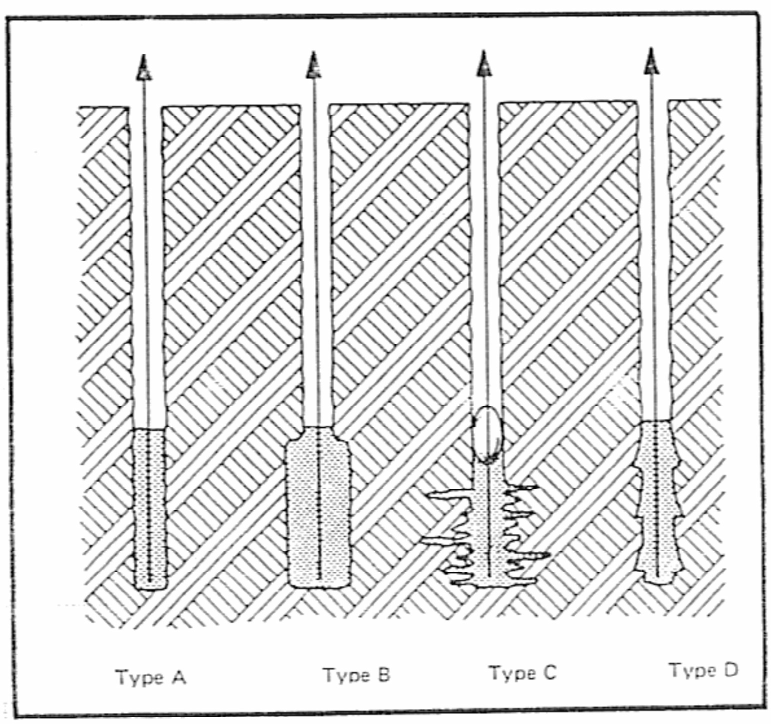


Figure 11. Main types of cement grouted anchorages (B.S.C.P. 8081. 1988)

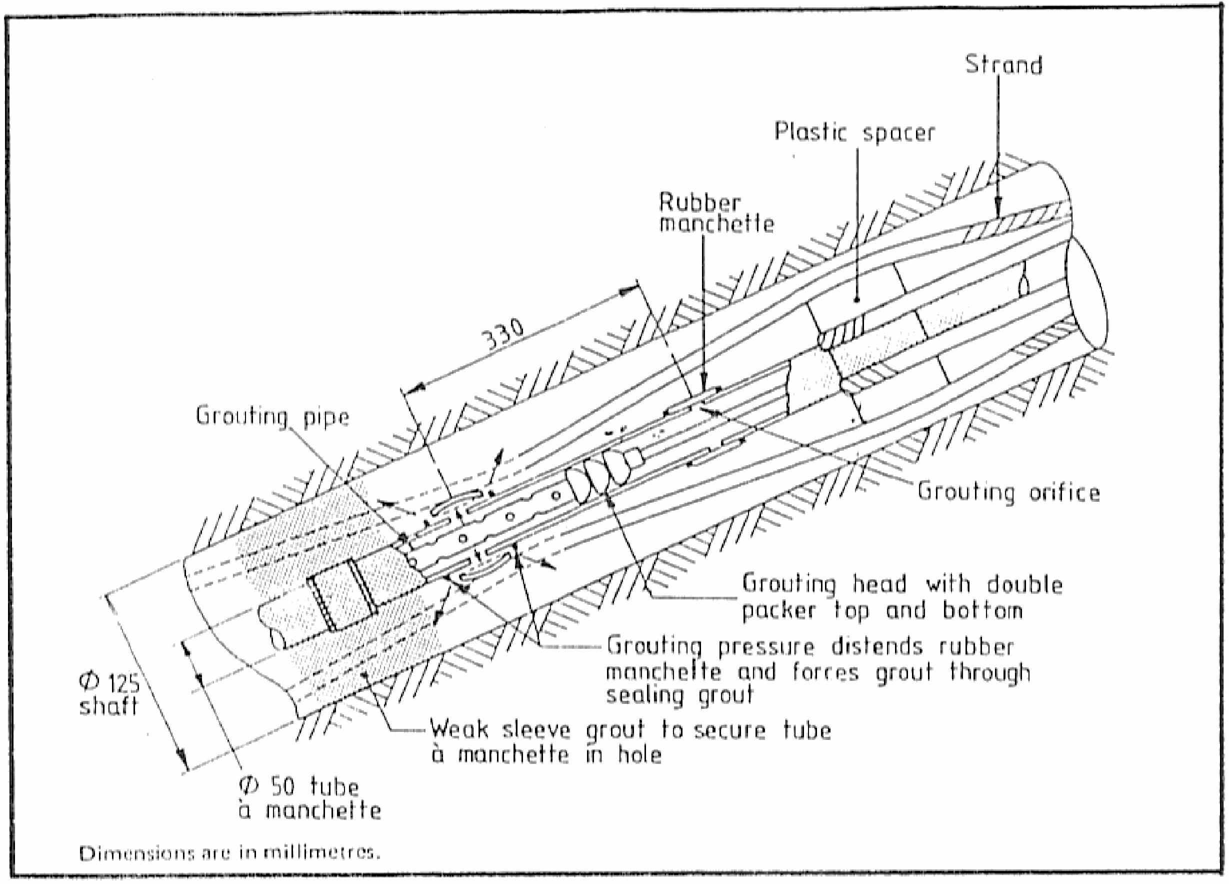


Figure 12. Detail of sleeved tube used in post-grouted anchorages (B.S.C.P. 8081, 1988)

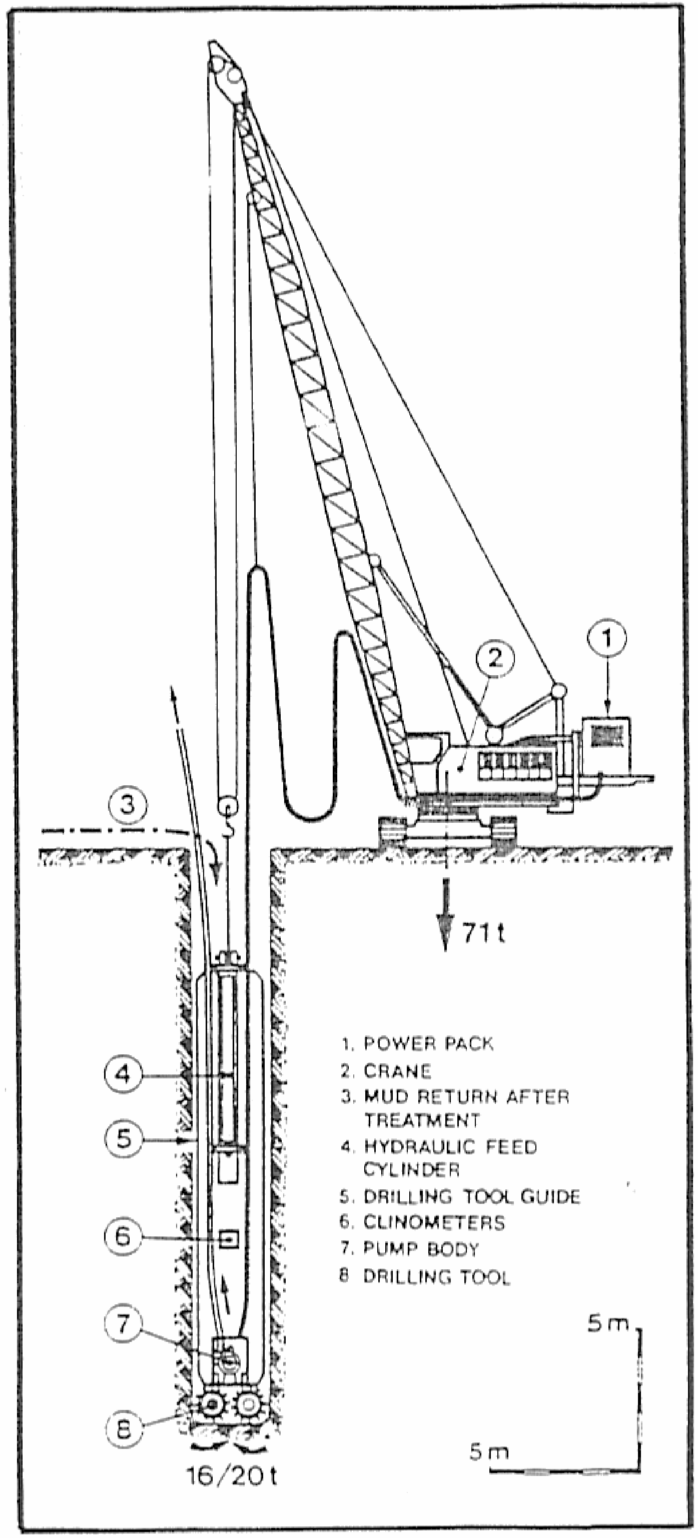
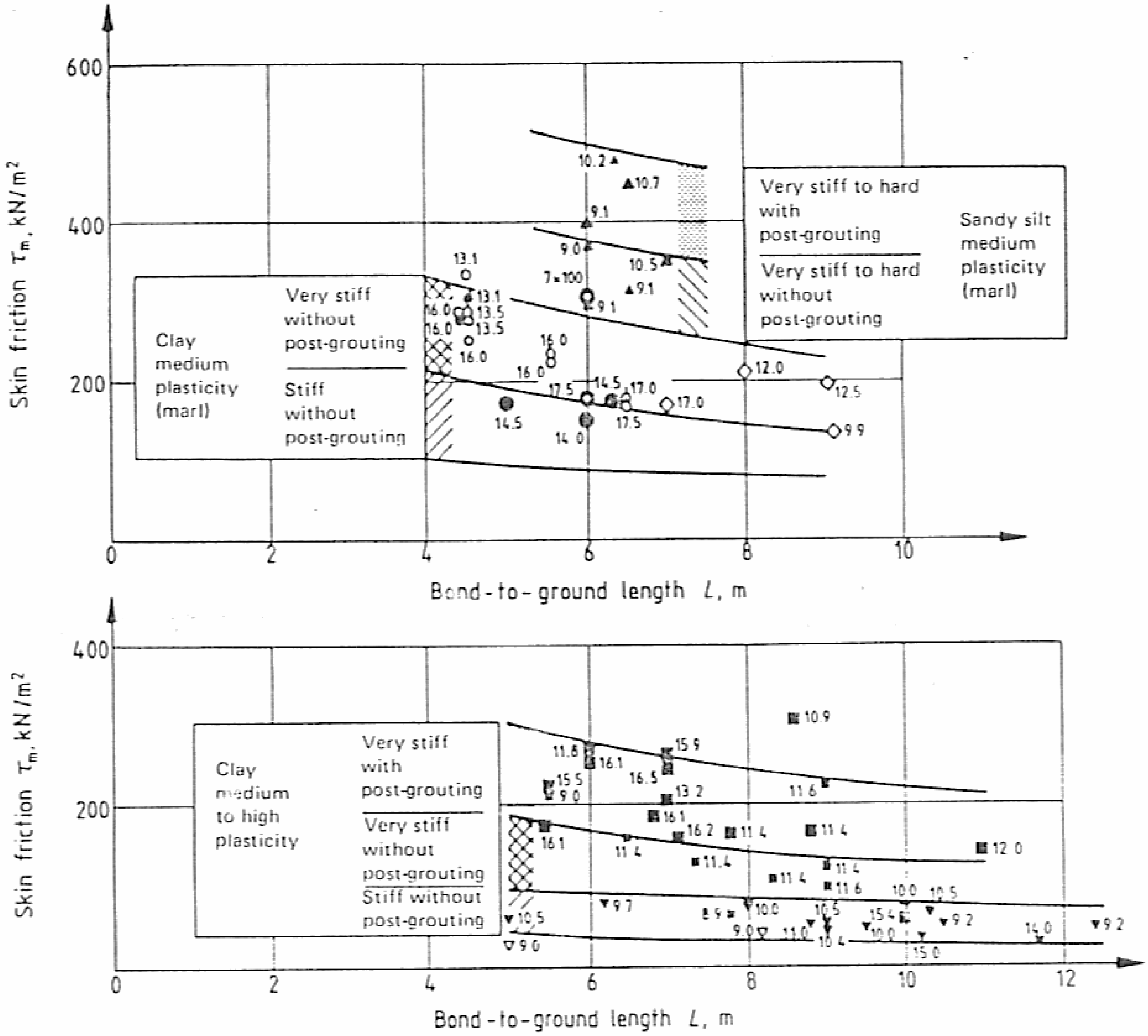


Figure 14. Principle of operation of the hydrofraise (hydromill) excavator. (Evers and Hovart, 1988)



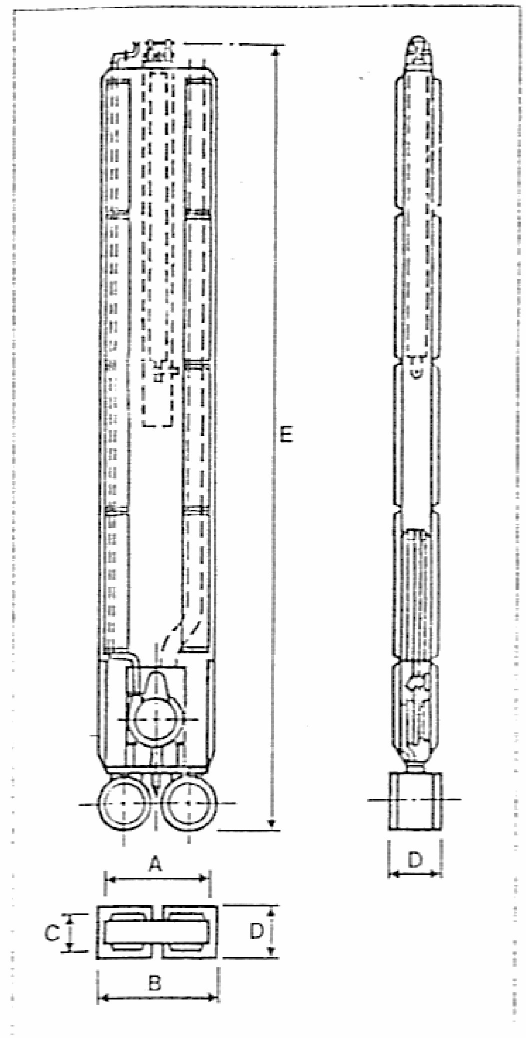
Failure load was reached	Failure load was not reached	Post-grouting	Type of soil	L_L %	P_L %	I_c %
▲	△	Without	Silt, very sandy (marl) medium plasticity	~45	~22	~1.25
▲	△	With				
●	○	Without	Clay (marl) medium plasticity	32 to 45	14 to 25	1.03 to 1.14
●	○	With				
●	○	Without		36 to 45	14 to 17	1.3 to 1.5
●	○	With				
	◇	Without	Silt medium plasticity	23 to 28	5 to 11	0.7 to 0.85
■		Without	Clay medium to high plasticity	48 to 58	23 to 35	1.1 to 1.2
■		With				
▼		Without		45 to 59	16 to 32	0.8 to 1.0

Figure 13. Skin friction in cohesive soils for various fixed anchor lengths, with and without post-grouting (Ostermayer, 1974)

Figure 15. Standard dimensions for hydrofraise (hydromill) excavation. (Soletanche, 1987)

A	2.36	2.36	2.36	2.36	2.36
B	2.40	2.40	2.40	2.40	2.40
C	0.59	0.76	0.96	1.16	1.46
D	0.63	0.80	1.00	1.20	1.50
E	15.00	15.00	15.00	15.00	15.00

METRES



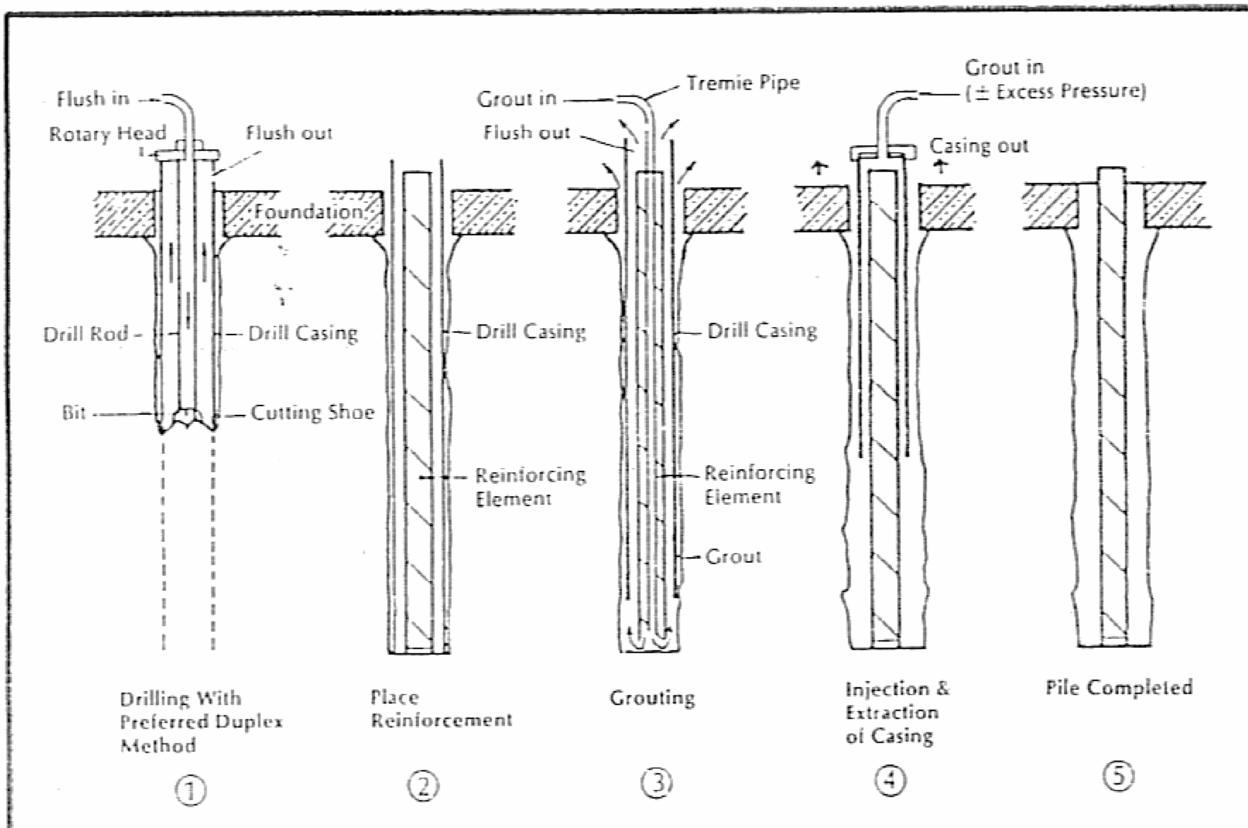


Figure 16. Standard sequence of minipile construction (After Koreck, 197

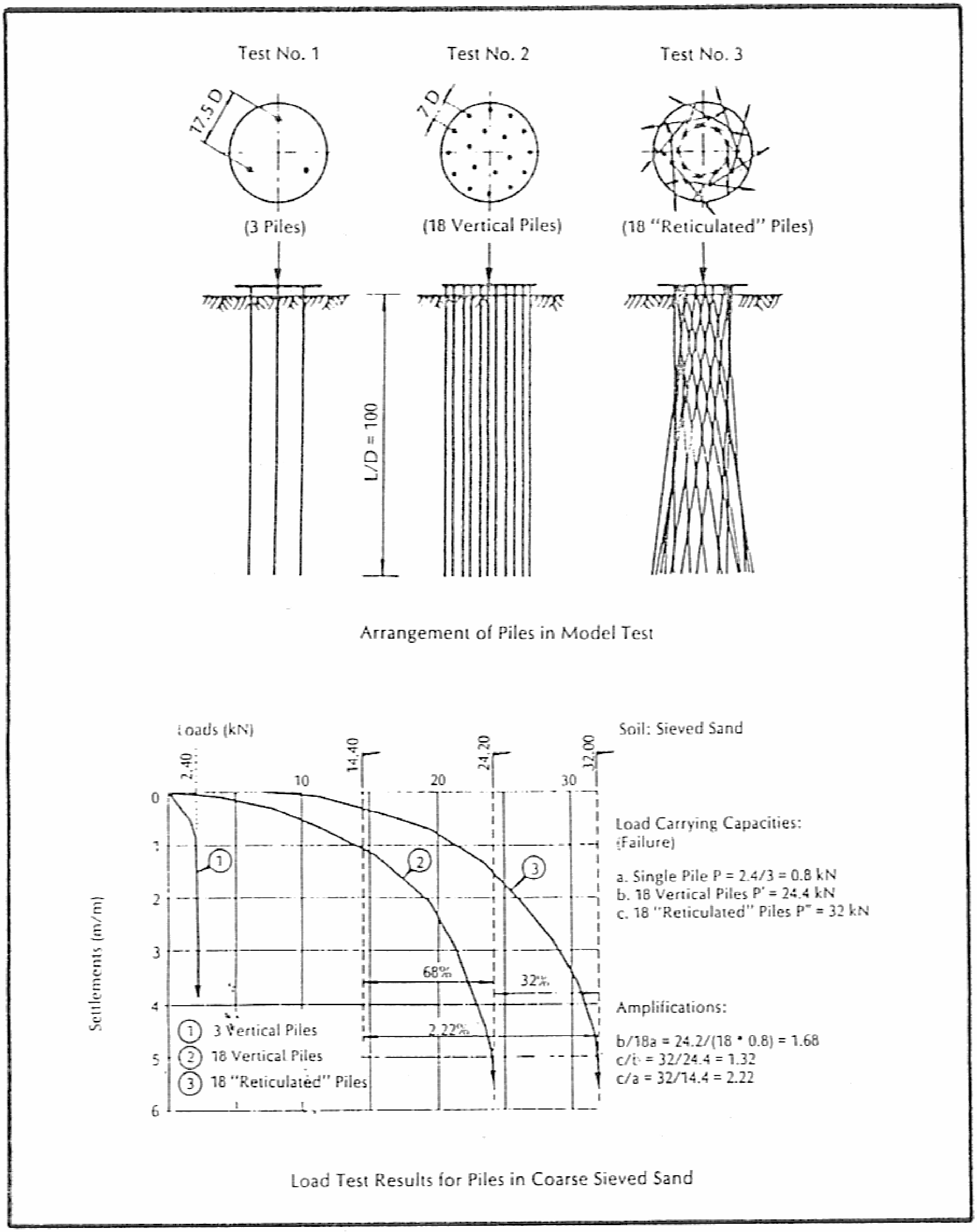


Figure 17. Model test data for different minipile arrangements in coarse sieved sand (Lizzi, 1978).

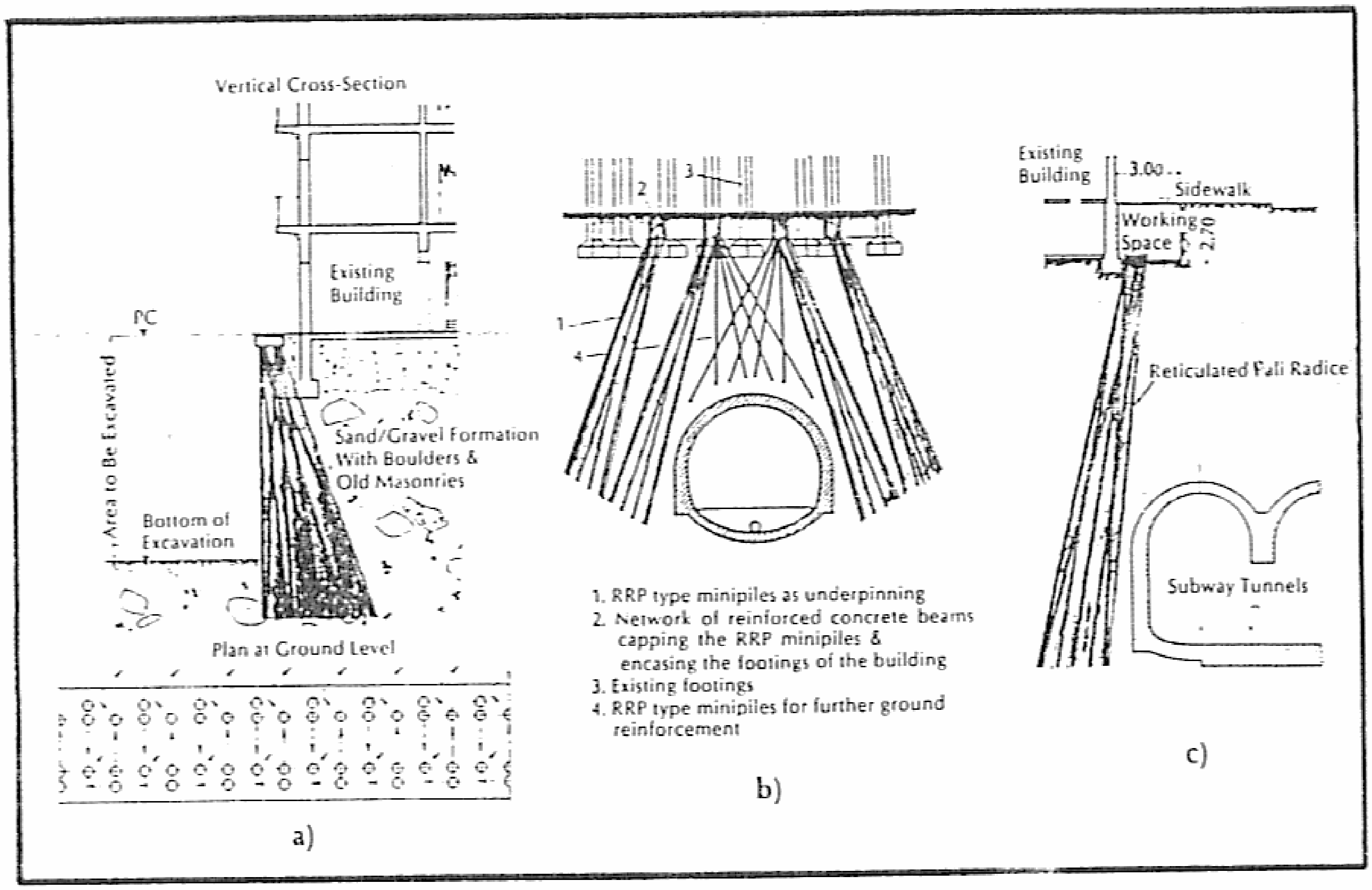


Figure 18. Applications of reticulated micropiles as used for in situ reinforcement: a) for cut and cover excavation: and. b) and c) around bo
tunnels. (After Lizzi, 1982)