TUNNEL GROUTING:
"AN ILLUSTRATED REVIEW
OF RECENT DEVELOPMENTS
IN GROUND TREATMENT"

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TUNNEL GROUTING AN ILLUSTRATED REVIEW
OF RECENT DEVELOPMENTS IN GROUND TREATMENT

During the last few years, major developments have impacted on the processes of soft ground treatment by grouting. These advances have been less spectacular in the fields of hydrofracture and compaction grouting, but quite fundamental as far as permeation and jet grouting are concerned. Major new trends in processes, methods, materials and control are outlined and illustrated later in the paper by reference to seven important case histories drawn from around the world, in a wide range of ground conditions.

1. INTRODUCTION

Within the scope of this review, ground treatment is synonymous with grouting: the injection of a foreign, fluid phase in order to improve in some way the ground's natural properties. Grouting in association with tunnelling works is conducted to improve the strength and/or reduce the permeability of the virgin ground, and may also be used to redensify and displace disturbed soil. Under this definition, backgrouting around tunnel linings is excluded from consideration, as is the alternative soil stabilization technique of soil freezing (Gallavresi, 1982).

Given the theme of the paper--a concentration on novel aspects and recent developments--the scope is limited to the grouting of soils. With the exception of the development of the MPSP system for efficient curtain grouting in collapsing rock strata (RODIO, 1983), the author is aware of no fundamental new developments in rock grouting following the data presented at New Orleans Grouting Conference (1982), and the consequent overview of Houlsby (1985). Thus, recommended "state of the art" type case histories would still include, for example, Restelli's (1978) paper on the acrylic grouting of microfissures to eliminate toxic gas infiltration (Figure 1), and Black et al's (1982) description of the sealing of the fissured limestones and weakly cemented sandstones for the sinking of two deep shafts in the Selby coal field.

In contrast, major developments are occurring in soil grouting, largely in response to the demands placed on and by tunnelling contractors involved in major schemes, under strenuous environmental and geological circumstances throughout the world. The extremely competitive value of bidding on such projects adds further pressure to the evolutionary process.

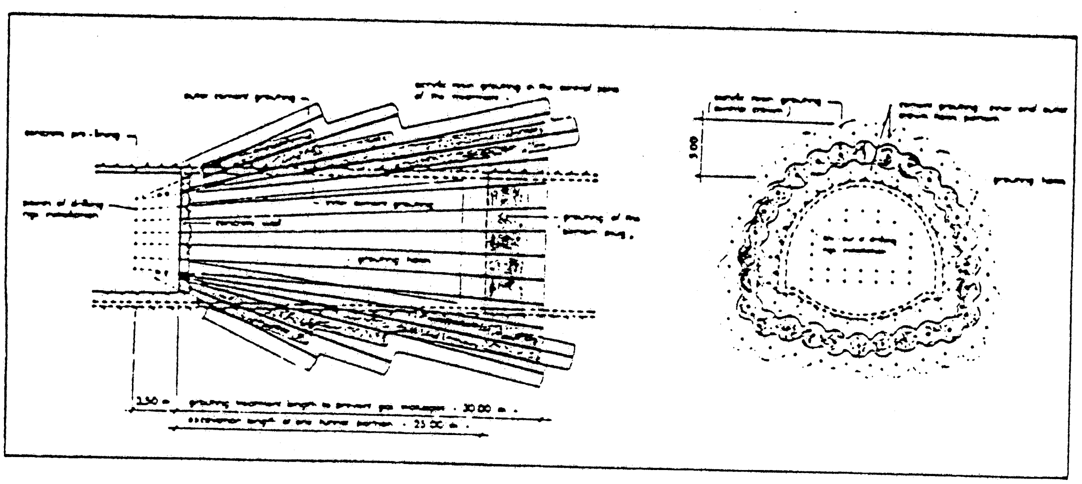


Fig 1 - CAPO CALAVA TUNNELS, ITALY: Longitudinal Section Shows the Grouting Arrangement for a Standard Tunnel Portion. Cross Section Shows Details of the "Sandwich" Grouting Treatment to Eliminate the Toxic Gas. (Restelli, 1978).

Most tunnelling engineers have some form of experience with grouting for ground treatment, as defined above. However, in many cases, it may be guessed that the experience has not been successful. Reasons--and excuses--range from unsuitability of the method or materials, to poor workmanship or control by the grouting contractor. In addition, it must be acknowledged that grouting has often been selected as a final remedial option in difficult situations where "conventional" techniques have also failed, instead of being regarded as the "design tool, as it should be from the onset " (Clough, 1981). In such conditions, the ground is invariably disturbed, and may be in a dynamic regime. Such factors do not improve the prospects of efficient ground treatment.

The modern trends in ground treatment described in this paper have been selected with such potential criticisms in mind. They emphasize (i) the wider range of grouting materials which are now available, (ii) the newer techniques and systems which have been developed to combat the problems of soft ground tunnelling, (iii) the advantages of integrating such techniques as routine construction methods, and (iv) the high degree of operational control which can now be exercised, and demonstrated, by an experienced and efficient grouting contractor.

This paper does not attempt to provide a comprehensive survey of soil grouting, for tunnels or otherwise. The literature is replete with such fundamental reviews (e.g., Littlejohn, 1983; Karol, 1983), while the ICE Works Construction Guide "Ground Stabilization: Deep Compaction and Grouting" (1984) provides a fine introduction. The author, therefore, assumes the reader to have some acquaintance with grouting. This paper first details the innovations before referring to the most modern case histories by way of illustration.

2. INNOVATIVE TRENDS IN SOIL GROUTING

2.1 BASIS FOR SELECTION - Overviews of ground treatment conventionally identify four basic categories of grouting:

(i) Hydrofracture
(ii) Compaction
(iii) Permeation
(iv) Replacement

(i) In hydrofracture grouting, the ground is deliberately split by injecting stable but fluid cement based grouts at high pressures (say up to $40M_{pa}$). The lenses and sheets of grout so formed increase total stress, fill unconnected voids, possibly consolidate the soil under injection pressure, and conceptually constitute impermeable barriers, mainly horizontal. 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 as is described below, some hydrofracture phenomena accompany most permeation grouting contracts either accidentally or in conjunction.

(ii) Compaction grouting is a specialized "uniquely American" process (Baker et al., 1983) that has been used since the early 1950's. Very stiff soil-cement mortar is injected at high pressures (to$35M_{pa}$) at discrete locations to compress and densify soft, loose or disturbed soil. Unlike the case of hydrofracture grouting, the grout forms a very dense and coherent bulb which does not extend far from the point of injection. Near surface injections result in the lifting of the ground surface--the technique of slabjacking as described for example by Bruce and Joyce (1983)--and indeed the earlier applications were exclusively for levelling slabs and light buildings on shallow foundations (ASCE, 1977). Prior to the Bolton Hill Tunnel project, described below, compaction grouting had been used in Baltimore to correct settlement problems caused by Metro tunnel construction--but only after the tunnel had been completed and structural damage to the overlying buildings had occurred. 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 source. Although compaction grouting has naturally practical and technical limitations, its application is being popularized, mainly as the result of the well researched (and publicized) Bolton Hill contract. Thus it is the preferred solution in the new Los Angeles Metro, has been used recently in Caracas, and is currently under consideration for certain situations in London. For such reasons, it is regarded as an innovation in the context of this presentation, and is detailed in the Case History section below. The interested reader is referred to the definitive paper by Warner (1982), in addition to those publications cited above.

(iii) In certain ways, the techniques involved in permeation grouting are the oldest and best researched. The aim 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, is clearly a major determinant of the method of grouting and the materials which may be used (Figure 2). This paper details several new developments in procedures, monitoring and materials as related to injection exclusively by the tube'a manchette (sleeved pipe) system already well known throughout the industry.

(iv) Replacement grouting is the youngest major category of ground treatment. According to Miki and Nakanishi (1984) and Miki (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 have approached their full economic and operational potential (Figure 3 and Table 1). Its development was fostered by the need to treat thoroughly soils from gravels to clays in areas where major environmental controls were strongly exercised over the use of chemical (permeation) grouts and allowable ground movements. In one particular form (Figure 4), the ground around the drill string is fragmented by a very high pressure (up to $50M_{pa}$) horizontally directed water and air jet, and partially expelled from the hole. The cavitated zone so formed is simultaneously filled from below with a cement based grout, which does, of course, incorporate some of the native ground. A simpler variant eliminates the air/water cavitation and instead uses only the high pressure grout jet to cavitate and eject, as well as inject. Popularly, the technique is called jet grouting, while the grouted product in situ is appropriately referred to as "soilcrete" by the GKN group of companies, which introduced the technique into the United Kingdom in 1982.

Virtually all ground treatment conducted in Japan (over $160,000 m^3$ of treated ground in 1983: Miki (1984)) and Germany, features some type of jet grouting whereas its use by Italian and French contractors is increasing fast, supplementing or replacing other forms. Successful case histories are also reported in Brazil, Taiwan, Korea and Singapore. For various reasons, development is at an earlier stage in the UK and the USA, although recent applications in dam sealing in both countries augur well

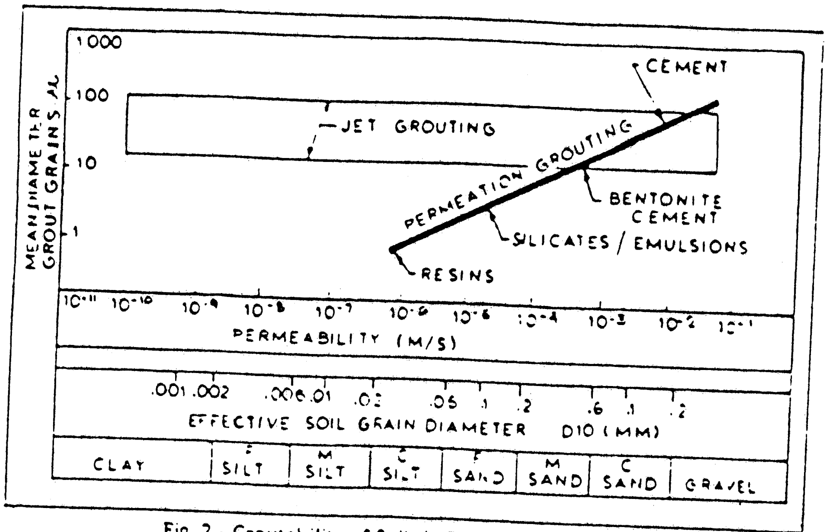


Fig. 2 - Groutability of Soils in Relation to Grout and Soil Properties (after Coomber, 1985).

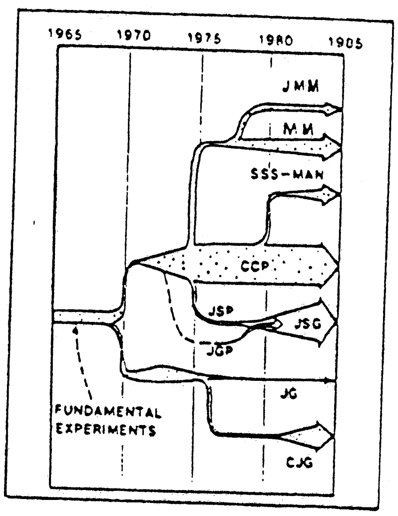


Fig. 3 - Development of Jet Grouting Methods in Japan (after Miki and Nakanishi, 1984).

for the immediate future.

Case histories from current contracts are described below. Further general data are provided in 1985 papers by Coomber, and Tornaghi and Cippo, amongst the rapidly growing technical literature.

2.2 Innovations in Permeation Grouting by Tube a Manchette Method

2.2.1 Equipment - Each of the numerous companies executing such work has its own specialties in terms of installing the tubes and executing the grouting. However, the following trends are evident on a worldwide basis:

- use of long mast "one stroke" drilling rigs, permitting fast installation programmes and usually operating with bentonite flush or a self hardening drilling mud, further reducing operating costs.
- use of more flexible grouting tubes, delivered in one piece to site, to ease installation, especially from within restricted tunnel access conditions, and reduce risk of malfunction due to leakage at joints. Alternatively, steel grouting tubes are being used for certain surface installations: these double as insitu reinforcement or underpinning for very delicate structures (Figure 26).
- use of hydraulically or pneumatically inflatable double packers for grouting, thus reducing labour effort, ensuring efficient sealing, and permitting any deviated or damaged hole to be "rescued" and still used for grouting.

2.2.2 Grouting Materials - In order to appreciate more fully the significance of the new developments, it is necessary to first consider the broad categories of grouts used in permeation. Mongilardi and Tornaghi (1986) observed the following classification on the basis of rheological performance--in order of increased penetrability (and cost):

(a) particulate suspensions (Binghamian fluids)
(b) colloidal solutions (evolutive Newtonian fluids)
(c) pure solutions (non-evolutive Newtonian fluids)

(a) Suspensions of solids in water are termed unstable when water loss by bleeding is significant. This is the case of pure cement grouts used at high w/c 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 increasing with time at a rate which may be considerably accelerated by drainage under pressure (Figure 6a) i.e., "pressure filtration."

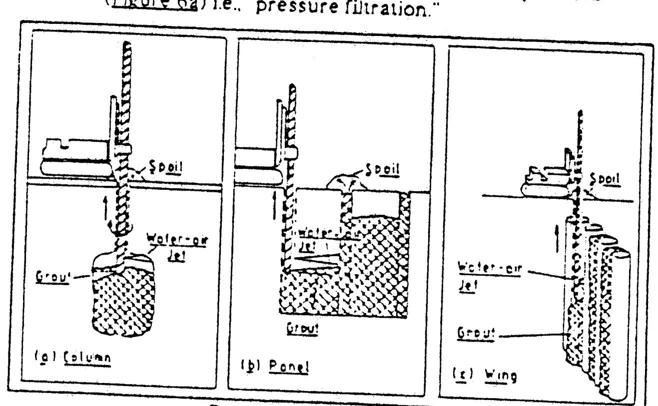


Fig. 4 - Jet Grout Construction (Coomber, 1985).

Original Japanese Name	Principle of Operation	Jetting Pressure (M_{pa})	Jetting Nozzle Dia (mm)	Revolving Rate (rpm)	Anticipated Column Dia (cm)	Notes
JG: Jet Grout	Upper water and lower grout jet	20	?	None	-	Panels only - soon obsolete
CCP: Chemical Churning Pile	Single grout jet	20 to 40	1.2-3.0	20	30-60	1. Chemicals now replaced by cement 2. Equivalent to RODIO's RODINJET 1.
JSG: Jumbo Special Grout	Single jet of grout enveloped in air	20	3-3.2	6	80-200	Originally called JSP (Jumbo Special Pile) but name changed for patent reasons
CJG: Column Jet Grout	Upper water and air jet and lower grout jet	40-50	1.8-3.0 (upper) 3.0-5.0 (lower) [8-9mm in KAJIMA System]	5	150-300	1. Referred to as "half Replacement" 2. Equivalent to RODIO's RODINJET 3, or KAJIMA/GKN KELLER system
MM: Mini Max	Like CCP but uses special "chemi-colime" cement	20	1.2	20	80-160	Specially for very weak soil and organics (e.g., soft peaty clays under water)
JJM: Jumbo MiniMax	As for MM except for addition of 20-40cm wing jet	20	1.2	20	100-200	"
3SMAN: Super Soil Stabiliza-tion Manage-ment	Air water jet used to excavate volume completely underwater. This is then surveyed ultrasoni-cally. If OK then tremied full of desired material. (See Figure 5)	20-60	2-2.8	3-7	200-400	1. To provide absolute control over shape and composition of column. 2. Effective to over 70m depth 3. "Complete replacement" 4. Most expensive technique, but ensures desired performance

TABLE 1 - Major Categories of Jet Grouting Variants (After Miki, 1984)

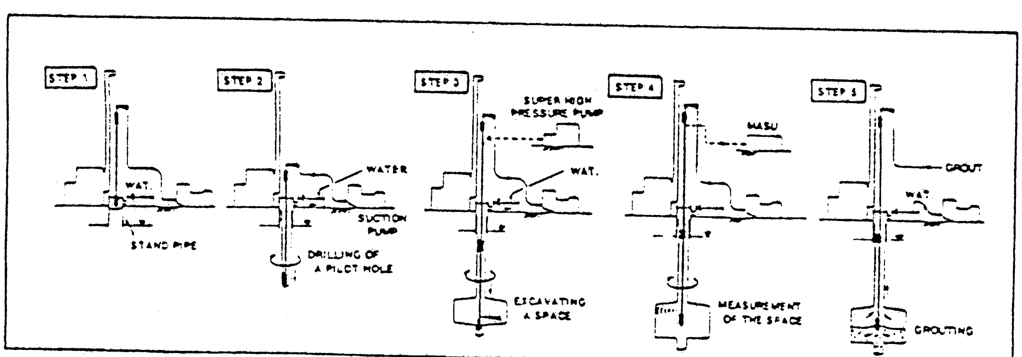


Fig. 5 - Operational Steps of New SSS-MAN (Super Soil Stabilisation Management) Application of Jet Grouting (Miki & Nakanishi, 1984).

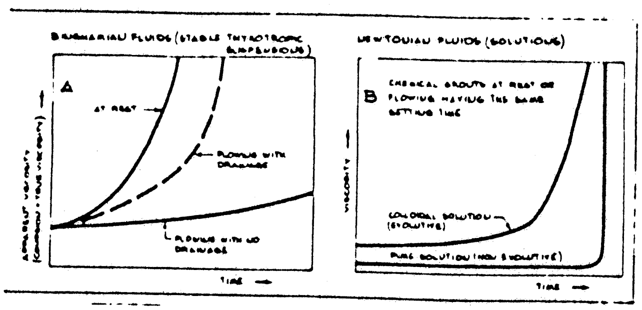


Fig 6 - Rheological Behaviour of Basic Grout Types (Mongilardi and Tornaghi, 1986).

Though the addition of colloidal products can minimize bleeding, filtration must be always considered as an important design factor with respect to penetrability and 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^{-3}m/s involves the additional or alternative use of chemical solutions in order to minimize hydrofracturing effects.

The chemical solutions exhibit quite different rheological features. The following properties are related to the viscosity/time relationship at a given temperature, without any other factors influencing penetrability. The chemical grouts can be grouped in two rheological sub-classes: colloidal "evolutive" and pure "non-evolutive" solutions.

(b) By far the best known colloidal solutions consist of diluted sodium silicate with inorganic or organic reagents producing relatively soft to hard silica gels. The term evolutive means that viscosity increases before setting (Figure 6b upper curve) at a rate mainly depending on concentration.

(c) 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 (Figure 6b lower curve). This outstanding property, associated to 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 permeablity not lower than 10^{-6}m/s).

Regarding the particulate suspensions, main obstacles to penetrability are related to (i) the maximum particle size of the solid components in the grout; v.v., the pore sizes in the soil, and (ii) the rate of pressure filtration which may induce rapid clogging even under low pressures.

The first problem is being tackled by generally introducing finer cements, for example, MC500 (Karol 1985) 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. In contrast, the filtration problem has represented the main obstacle in the past, since in conventional "stable" grouts 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.

However, this problem has very recently been resolved by RODIO, which has developed an entirely new class of cement-bentonite grouts exhibiting extremely important and advantageous properties:

- very low filtration rates (considering a relative scale with bentonite mud being 1, then these new grouts are 2.5, compared with 10 for conventional cement-bentonite grouts, and 20-30 for cement grouts).
- no bleeding
- low values of yield point and plastic cohesion over an adjustable period time
- higher long-term strength and lower permeability in comparison with conventinal grouts having similar contents of cement and bentonite.

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
- a 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 treat medium-coarse sands with refined products, minimizing hydrofracturing effects.

Chemical grouts based on sodium silicate solutions and inorganic reagents (e.g., sodium aluminate, sodium bicarbonate) have long been used to provide soft gels for waterproofing sands. In order to increase strength, silicate concentrations must be raised, but this ensures almost instantaneous gelling. Thus, early higher strength requirements (say up to 0.5-1.0M_{pa}) could only be satisfied by a two shot system such as the Joosten process. Most recently the Japanese have developed specialized drill and grout systems such as LAG (Tokoro et al. 1982) and DDS (Bruce 1984) which can handle the problems of injecting flash setting grouts of this type, but these systems are only practical in the softer uniform deposits.

The use of organic reagents (e.g., Rhone Poulenc 600) capable of matching these higher strength targets but still offering long gel times has grown over the last 20 years. However, in certain areas including Japan and Germany, such organically based reagents are not environmentally acceptable. In addition, creep effects may be a significant problem for silicate gel stabilized soils if the design involves a high and permanent loading (Tan and Clough 1980), while questions of durability under certain conditions may be valid. (The author, however, supports the view of Mongilardi and Tornaghi (1986) that there are certainly "overconservative prejudices against organic materials in some important urban areas and below the water table.")

In light of these problems, potential and real, the newest developments have led to the evolution of a new type of chemical grout, composed of (a) a silica liquor, and (b) an inorganic reagent. As opposed to commercial alkaline sodium silicates, which are aqueous solutions of colloidal silica particles dispersed in soda, the liquor is a true solution of activated silica. The activiated dissolved silica when mixed with the reagent produces calcium hydrosilicates with a crystalline structure 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 no more an evolutive "gelation" as in the case of silica gels, involving the formation of macromolecular aggregates and possible loss of silicized water (syneresis). On the contrary, it is a direct reaction on a molecular scale.

This type of mix (Silacsol), recently developed in France and successfully used by RODIO 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. In fact, 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.

2.2.3 Instrumentation - Throughout the grouting industry there is an increasing employment of computer-aided devices as monitors and controls over grouting operations in the field. This 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, as far as injections are concerned, will be similar to the RODIO PAGURO System of remote centralized monitoring. This monitors numerically and graphically the full injection characteristics of each pump (the setting of which is incidentally still under manual control) in real time. It thereafter gives a hard copy summary of each point injected (including volume, maximum and average pressures and flow rates and time). Such data then provide the basis for technical review of the grouting conducted (e.g., grout take analyses) and quantities of work executed, for payment purposes. Clearly the investment in such sophistication is economically justifiable only in projects of appreciable scale and/or complexity, (e.g., Milano Metro, Cairo Wastewater Tunnels).

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 continuously record the penetration rate, rotational speed, thrust, torque and flush pressure encountered in drilling a certain exploratory hole. These 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 10cm intervals (Figure 7). This geological log, conducted in advance of grouting and tunnelling, permits optimization of the subsequent drilling and grouting parameters as well as giving invaluable information to the tunnelling contractor in that potentially dangerous conditions (i.e., sand runs) can be closely predicted. The accuracy of the geological log has proved exceptional in the conditions of the Milano Metro--mixed gravels, sands and silts to over 25m depth--and groups of three investigatory holes have been routinely drilled at about 6m intervals 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 has been developed by conducting statistical analyses of 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 insitu ground properties, and 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.

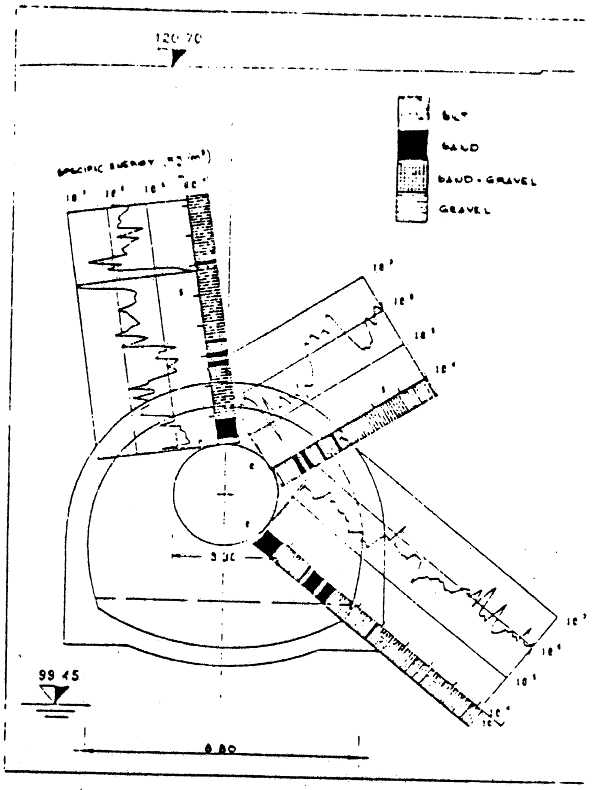
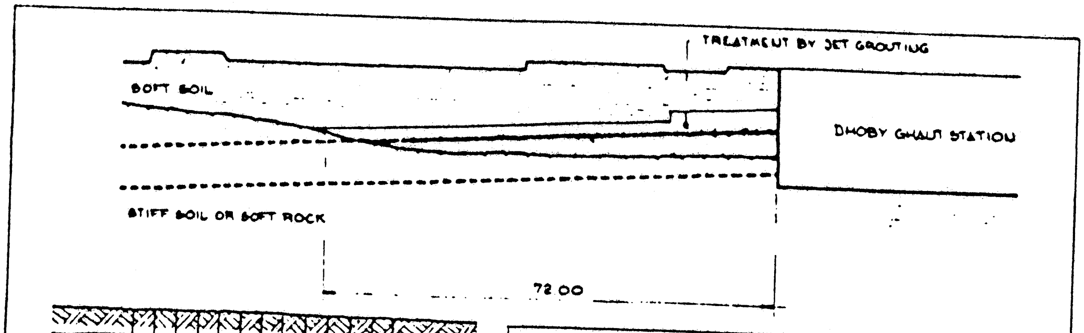


Fig. 7 - Soil Profiles Predicted by the PAPERO Method of Electronic Sensing, Milano Metro, Line 3, (Bruce et al., 1987).

Fig. 8 - View of Excavated RODINJET Columns, in Trial Site, Milano, Italy.

2.3 INNOVATIONS 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 safe cement-based grouts across the whole range of soil types. This opinion is enhanced by the very dramatic photographic evidence from excavated test sections (Figure 8). However, it must be emphasized that any system which may involve the simultaneous injection of up to three fluids at operating pressures of up to $60M_{pa}$ must be handled with extreme care and only in appropriate applications and ground conditions. Many readers will already be aware of unhappy experiences in this country (due to inefficient mix and inject procedures and inappropriate application of the technique), and in Singapore (where excessive ground heaves severely upset surface structures and disrupted subsurface services). The credentials, resources and methods of the specialist contractor must therefore be reviewed with special care.



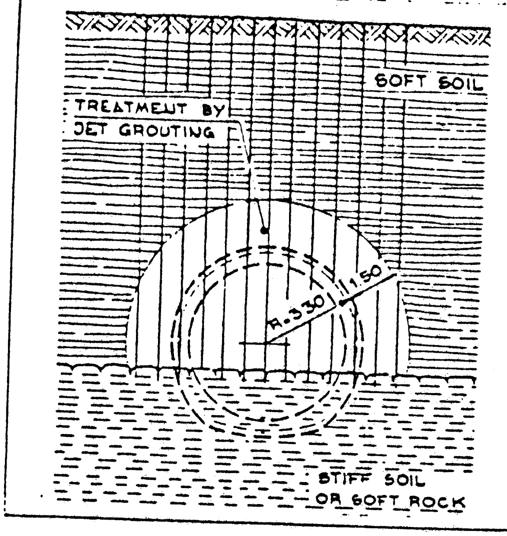


Fig. 9 - Jet Grouted Blocks for Tunnel Excavation, Contract 106, Singapore MRT, (after Lunardi, et al., 1986).

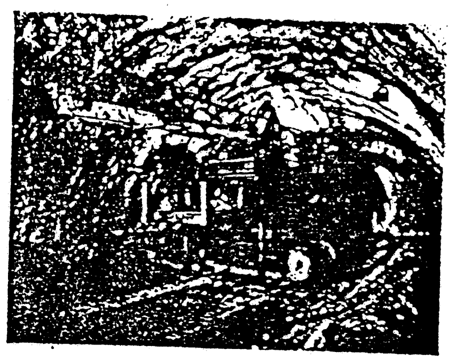


Fig. 11 - RODIO SR510 Tunnel Drilling Rig With Long Mast for Drilling Horizontal Micropiles ("infilaggi"), and Jet Grouting.

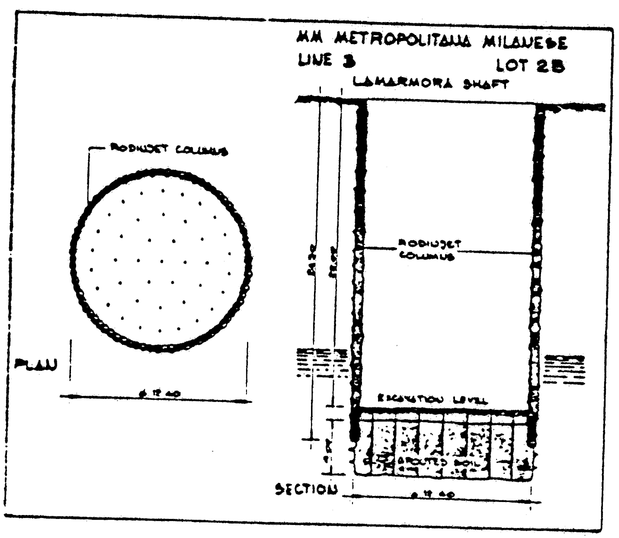


Fig. 10 - Jet Grouting for Shaft Construction, Milano Metro, Line 3, (Mongilardi and Tornaghi, 1986).

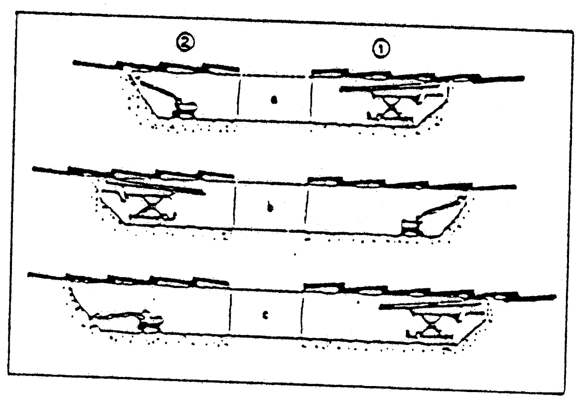


Fig. 12 - Typical Work Sequencing in a Jet Grout Tunnel With Two Simultaneous Faces (Cippo et al., 1986).

The RODIO group of companies currently have two main types of jet grouting options: Rodinjet 1 (similar to the mix in place CCP system, based on grout jetting only) and Rodinjet 3 (similar to CJG system employing air and water jetting, and grout replacement). Almost 500km of jet grouted columns have been installed in the last 6 years, in various countries in Europe (including USSR and the Far East). Most of the applications have been for tunnel and shaft excavations (Figures 9 and 10), while much of the work has been executed horizontally by purpose built drilling rigs (Figure 11), to form "umbrellas" for subsequent phased excavation (Figure 12). In conjunction with base grouting (to counteract boiling, piping or heave) the weight of the grouted plug (and ungrouted zone above to formation level) alone is inadequate to resist hydrostatic uplift forces (Coomber 1985). The majority of the resistance is developed by peripheral contact stresses arising from (a) friction generated by active earth pressures on the side walls and/or (b) adhesion developed between the soilcrete

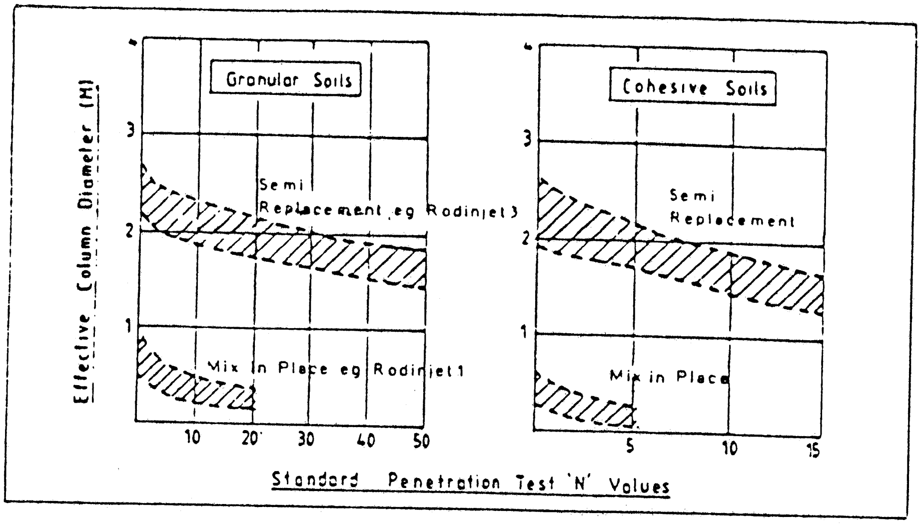


Fig. 13 - Relation of Effective Column Diameter to Grouting Method and N Value (Under Equivalent Operating Parameters), (after Miki and Nakanishi, 1984).

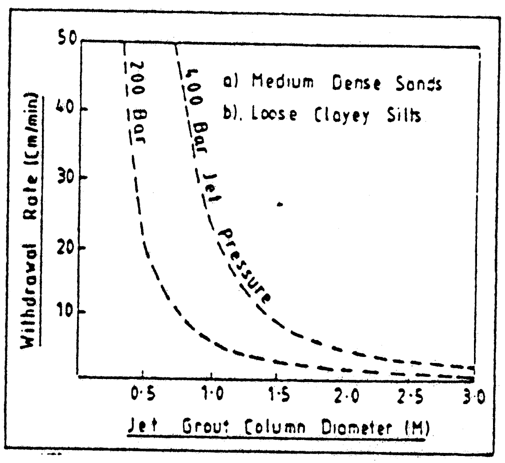


Fig. 14 - Influence of Withdrawal Rate on Column Diameter (3 Fluid System), (Coomber, 1985).

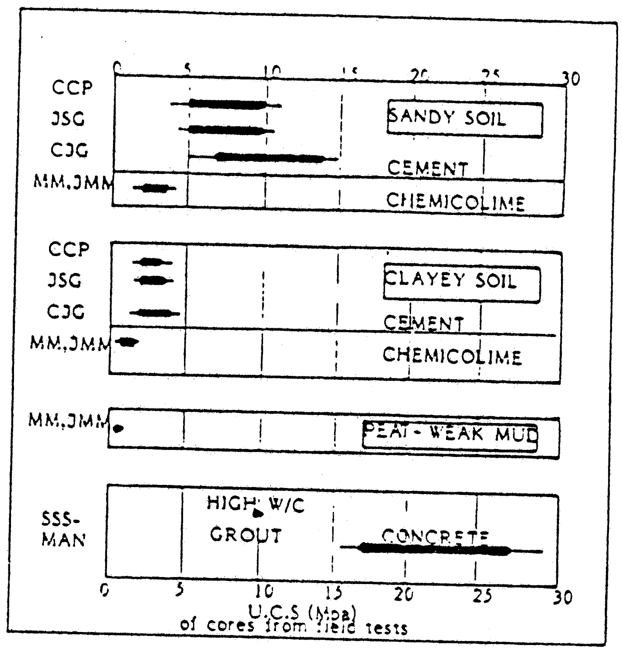


Fig. 15 - Unconfined Compression Strengths of Cores Extracted from Various Systems of Jet Grouted Columns (after Miki, 1984).

and the walls. For this to be valid, the plug must be considered as a monolithic slab: its thickness can then be checked in relation to resistance to lateral compression and bending.

Miki and Nakanishi (1984) reported on data recorded from full scale tests conducted over many years. For example, the relation between method, soil type and resulting column diameters has been quantified (Figure 13) for particular operating parameters. The influence of these factors, which include fluid flow rates and pressures, nozzle sizes and shapes, monitor rpm and rate of withdrawal, and volumes and compositions has also been investigated in Japan and Europe. Coomber (1985) states that water jet pressure and monitor withdrawal rate are the most significant of these parameters (Figure 14).

Grout mix constituents and composition can be varied to meet the specific requirements and for example, pfa is a common addition in Britain. Mix viscosity should be fairly 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 to be drained out both from soil and grout, whereas in a cohesive soil of low permeablity, poor or no drainage is likely. This is a principal reason why the strength of the grouted column (depending primarily on the final w/c ratio) is much lower for clay than for sand and gravel, all other factors being equal. (Figure 15)

PROJECT LOCATION	NATURE/SIGNIFICANCE	APPLICATION	MAJOR SOIL TYPE	MAJOR REFERENCE
A' Cut & Cover				
1. Cairo Metro, Egypt	Permeation grouting with soft silica gel & hydrofracture grouting.	Base slab between twin diaphragm walls.	Fine-medium alluvial cohesionless sands.	Abdel Salam (1984)
2. Torre Annunziata, Napoli, Italy	Jet grouting for vertical & horizontal sealing.	Pile sealing & base slab for new sewer.	Fine, soft sands & silts of volcanic origin.	Unpublished (current project)
B' Underground Excavation				
3. Bolton Hill Tunnel, Baltimore, USA	Compaction grouting.	Reduction in surface settlements over tunnel.	V. dense sands & gravels & silts.	Baker et al (1983)
4. Hong Kong MTR Tunnels	Permeation grouting with higher strength silica gels.	Many applications to enable tunnelling, underpin buildings & reduce settlements.	Completely decomposed granites.	Bruce & Shirlaw (1985)
5. Cairo Wastewater Tunnels, Egypt	Permeation grouting medium-higher strength silica gels with very close quality controls.	Grouting round tunnels & shafts for strength & water tightness.	Fine-medium alluvial cohesionless sands.	Greenwood et al (1987)
6. Singapore MRT Tunnels	Jet grouting with trial block.	Treatment of soft deposits to safeguard tunnelling under water table.	Soft marine clays.	Tornaghi & Cippo (1985)
7. Milano Metro Tunnels	Jet grouting & permeation grouting with new high strength chemicals & special cement grouts.	Many applications: as for Project 4 above.	Alluvial gravels, sands, & silts.	Tornaghi & Mongilardi (1986)

Table 2 - Introductory Details of Case Histories Described in the Text

Although there clearly exists a wealth of experimental data in conditions worldwide, it should remain an integral part of every jet grouting programme to have a field trial, prior to the commencement of the production works, in order to verify and optimize operating parameters. Such trials should include visual inspection of the grouting, wherever practical, by excavation.

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 which may result "down the line" due to inefficient or inappropriate treatment procedures, however well intentioned at planning stage.

3. ILLUSTRATIVE CASE HISTORIES

Seven case histories have been selected on grounds of technical significance, application and scale. They represent "state of the art" solutions in soils from gravels to clays, as related to tunnelling activities. Only projects which have been executed or described within the last four years have been considered (Table 2).

3.1 TUNNELLING BY CUT AND COVER

3.1.1 Cairo Metro, Egypt - Cut and cover methods were adopted to allow the construction of the new north-south underground line through the busiest parts of Cairo. This section is 4.5km long and includes five underground stations, as well as numerous ancillary structures.

The basic method is illustrated in Figure 16, the bottom seal to the twin diaphragm walls being provided by injecting cement bentonite, and silica gel, grouts from the surface via the tube'a manchette system.

The grouting was conducted in Nile valley medium sands--varying vertically and laterally into fine/silty sand. The design of the grouting was based on the following parameters:

Grain size distribution
D_{10}:0.2-1.0mm
Porosity : 35% to 45%
Insitu permeability:
10^{-3}-10^{-4}m/s

The groundwater was highly alkaline: sulphate up to 3884mg/litre, carbonate up to 2077mg/litre, and chloride up to 2000mg/litre.

Following initial trials and field observations, which optimized the construction and grouting methodologies, the following sequence was followed:

A split spacing grouting system (Figure 17) was used in three phases:

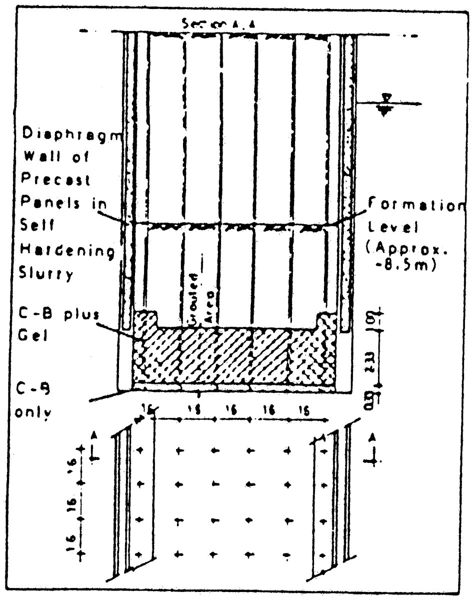


Fig. 16 - General Arrangement of Grouted Plug, Cairo Metro (Abdel Salam, 1984).

1. Cement-bentonite grout in alternate holes with total grout volume equivalent to 5%-7% ground volume at pressures less than $1.5M_{pa}$ (250kg cement and 50kg bentonite per cubic meter of mix). The purpose of this phase was to seal major voids or potential seepage paths and act as a "skeleton" for the later chemical injections.
2. Chemical grout first in the holes not previously grouted, then in the others (i.e., Phase 1 holes). Target volumes of 35%-40% ground volume were set, with a pressure target of $2M_{pa}$. A mix of sodium silicate and water (weight ratio 1:5-6) was reacted with a sodium bicarbonate reagent (12%-14% by weight of silicate) to give a relatively low strength, but penetrative and economic grout with a gel time of about 40 minutes and an initial viscosity of 5cp.
3. Cement-bentonite grout in holes not previously injected in Phase 1. Further volumes equivalent to 6%-8% were injected at up to $3.5M_{pa}$. This claquage grouting was intended as a final sealing, bearing in mind that most hydrofractures run horizontally. A final "control" phase (3bis) was conducted to tighten up suspect areas.

Upon satisfactory checking (by pump tests and piezometers) that the target average permeability of 10^{-6}m/sec had been achieved, the tubes were backfilled with cement grout then left to set. Thereafter each tube was severed at a level 1m below excavation level with a special "in hole" cutter so that subsequent excavation would not "pull" an old tube and so cause a direct pipe to the surface.

Excavation was thereafter conducted in short (15-20m) lengths as fast as possible to avoid the risk of major blowins occurring due to time related failure of the grouted zone. Blinding concrete was also promptly applied to further seal the base and give additional self weight and structural strength to the plug.

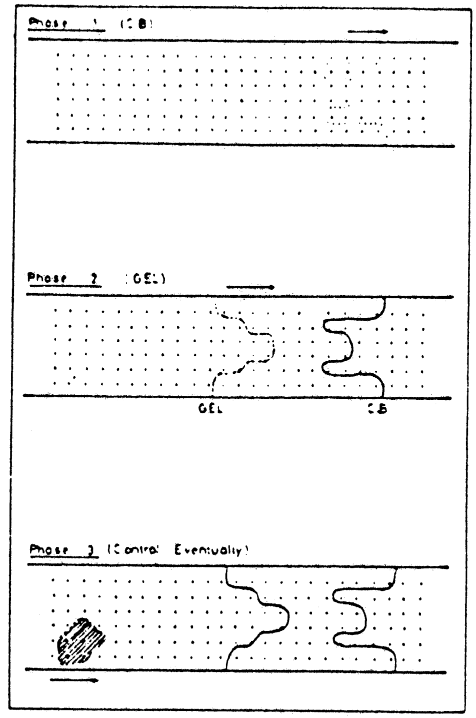


Fig. 17 - Details of Grouting Sequence, Cairo Metro (Abdel Salam, 1984).

3.1.2 Torre Annunziata, Napoli, Italy - A new major sewage line is being laid through an intensely urbanized area of the town of Torre Annunziata, to the south of Naples, Italy. The scheme involves a 900m long section where a reinforced concrete collector approximately 4.5m square is to be cast. The surface cover over this structure varies from 1.5 to 10.0m.

The subsoil conditions are mainly soft silty deposits of volcanic origin, with 1-2m of variable fill and rubble above (containing a mass of domestic service pipes). Groundwater level is at about-5m. The route of the sewer follows an extremely busy street, with domestic and commercial buildings of several stories on each side. Clearly, minimal settlements could therefore be tolerated outside the construction boundaries, while the scale and time of environmental upheaval also had to be minimized. In addition, the very restrictive access conditions precluded the consideration of certain techniques otherwise offering potential solutions (e.g., diaphragm walls).

A solution was developed which addressed each of these environmental and technical problems while still being financially attractive:
- where the cover to the crown was to be less than 4m, the excavation was created in cut and cover. The excavation support was provided by reinforced bored piles, the lateral continuity between each being guaranteed by jet grouting. Resistance to base heave or leakage was provided by a "floor" of jet grouting (Figure 18).
- where the cover was greater, hand tunnelling was feasible assuming the same pile and grouting scheme as before, but supplemented by a jet grouted roof to ensure the support of the overlying roadway.

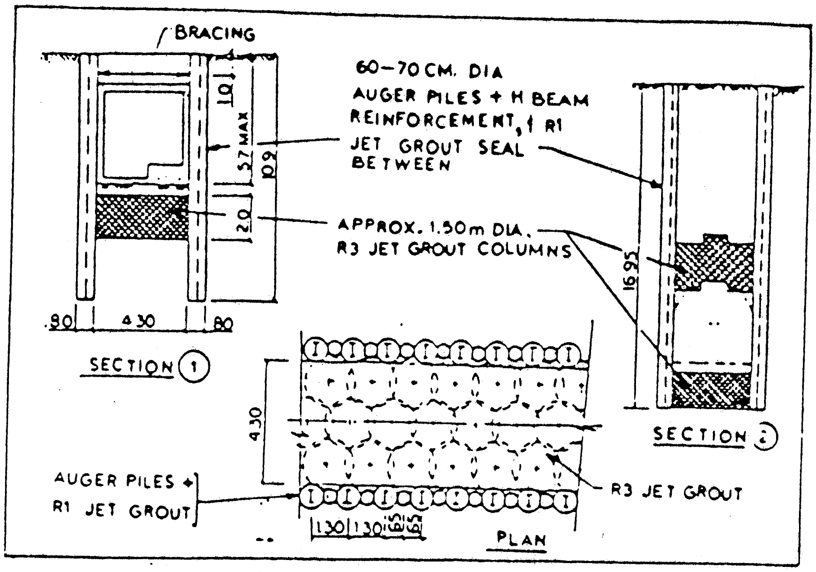


Fig 18 - Jet Grouting Applications to Enable Cut and Cover (Section 1) and Tunnelled Excavation (Section 2) for New Sewer Construction, Torre Annunziata, Napoli, Italy.

The 2118Nr 800mm diameter piles, totalling over 24,000 lineal metres were augered to depth and were reinforced with steel H-sections. They were spaced at a maximum of 1.2m centres in one row each side of the excavation. The ground between the piles was treated by the Rodinjet 1 system, giving "mix in place" columns of 600mm notional diameter. Grout of w/c=1 was jetted at a pressure of $40M_{pa}$. The withdrawal of the monitor during grouting was accomplished by an electro-hydraulic control on the drill rig which automatically lifted the monitor a preset distance at preset time intervals. In this way the injection parameters were precisely controlled to ensure uniform, effective treatment and no surface displacement.

The grouted base (and where necessary, roof) of the excavation was achieved by the Rodinjet 3 method. Over 5500m of columns, 1.5m in diameter, were involved using jetting pressures up to $50M_{pa}$ for water and $0.8M_{pa}$ for air. The subsequent cement grouting pressures were around $10M_{pa}$.

Section by section excavation and casting sequences were followed by the main contractor to allow both interim verification of the grouting effectiveness, and as a further aid towards minimizing the environmental upheaval.

3.2 TUNNELLING BY UNDER-GROUND EXCAVATION

3.2.1 Bolton Hill Tunnel, Baltimore, USA - Bolton Hill Tunnel, part of Northwest Line of the Baltimore Region RTS, was built between 1977 and 1980. As noted above, compaction grouting had been already used for a quarter of a century for problems unrelated to tunnelling, or used to uplift structures well after tunnelling had completed and settlement became apparent. This case history is claimed to be the first application of compaction grouting for controlling ground movements during tunnelling. The authors also claimed the project illustrated "the improvements in the prediction and control of ground movements that are being achieved through cooperative efforts within the tunnelling industry." Very significantly, they confirmed that these advances had been made in the climate created by revoking the philosophy of ignoring the ground movement problem--"where a utility can accept no movement, specifications allow no movement, the contractor causes no movement, and the engineer measures no movement."

The tunnel has almost 3500m of 5.9m diameter steel lined tunnel 12-23m below the ground surface in very dense sand and gravel with occasional silt and clay lenses, overlying very dense residual soils. Compaction grouting was specified for the first time, as the procedure for uplifting 39 relatively highly loaded structures (2-5 stories high on brick bearing walls) if subjected to settlements over 6mm. In addition, special tunnelling procedures were specified to limit loss of ground in the tunnel (e.g., compressed air, careful shield configuration, and back grouting) estimated at around 1.9 $ft.^3/ft.$ ($0.016 m^3/m$) of tunnel (due to the 10mm overcutter bar at the leading edge of the shield, and 6.3 $ft.^3/ft.$ ($0.054 m^3/m$) in the 32mm wide annular gap left by the steel ungrouted shield tailskin.

It was estimated that the surface settlement trough volume (Vs) would be 1.3-5 $ft.^3/ft.$, (0.011-$0.042 m^3/m$) i.e., about half the volume loss around the tunnel. The corresponding maximum surface settlement would be 12-50mm for a trough whose width in cross section is defined by lines extending upward and outward from the tunnel springline at an angle of 30° from the vertical.

In contrast to typical practice (i.e., underpin individual footings) to that time, the injection was made from tubes placed just above the tunnel crown for the following reasons:

- this method attacked the source of the problem (holes usually terminated 1.5-4.5m above crown)
- grout tubes could be installed from open air (roadway) locations, as opposed to from restricted access basements

• grouting could be controlled more easily, through successive holes, just over or behind the shield as it advanced as opposed to attempting simultaneous uplifts from many points.

• grouting at depth, the influence of a single bulb was spread, thus avoiding local differential uplifts developing.

In order to finalize the details of the system, a Federally funded test section was first conducted. The compaction grouting programme was carried out by grouting through vertical or inclined 76mm diameter grout pipes placed from the street surface at 1.5m centres over each tunnel. Grouting was conducted at 8.5 to 9.8m depth at pressures of 2.5-4.3M_{pa}, as measured at the top of the grout pipe, and until extraction of the grout pipe by a metre did not result in lower pressures nor increase the rate of grout take. Grouting operations were begun at each grout pipe after the tail of the shield had passed below the end of the pipe. Grout was injected usually in one hole at a time, in sequence.

The cement-fine sand mix had a slump of 2-5cm and mixed such that it could be pumped at up to 4M_{pa} without sand blockages. Injected volumes ranged from 0.8 to 3.0m^3/hole averaging 12 $ft.^3$ per foot of tunnel (0.1m^3/m) for the first, and 6 $ft.^3$ per foot of tunnel (0.05m^3/m) for the second. (Grout takes per hole were reduced and hole spacings increased to 3m in later production work.) Surface settlements were negligible (less than 3mm) and the volume of the surface settlement trough for each tunnel was negligible (less than 0.5 $ft.^3$/ft. of tunnel (0.004m^3/m)). Thus the surface settlements had been virtually eliminated by using volumes of grout equivalent to 1-1.5 times volume loss around the tunnel. Detailed examination of surface settlement patterns with time showed that the grout bulb influences a cone-shaped zone above itself, extending upwards at an angle of about 30°. Thus grout bulbs 3-4.5m beyond the settlement cross section can reduce the shallow settlement at the cross section.

Figure 19 illustrates clearly the effectiveness of the grout in reducing settlements: above the bulb almost zero, whereas below the bulb large settlements were recorded (and actually increased by up to 25mm by downward pressure of the bulb).

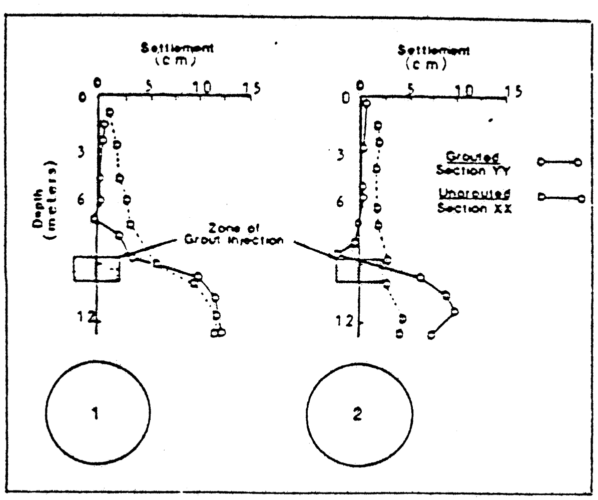


Fig 19 - Comparison of Displacement Depth Profiles for Grouted and Ungrouted Sections, Bolton Hill Tunnel, Baltimore (Baker et al., 1983).

Regarding the mechanisms of the bulb:
55% of the volume densified soils within 1.5m radius.
30% of the volume caused downward compression of loosened soil over the crown, below the 1.5m influence zone: 5% of the volume gave horizontal movement, towards the loosened soils over the shield tail:
10% of the volume gave heave in the soils to 6-9m above the crown.

Regarding the exact timing of the grouting operation, the authors recommended that compaction grout be pumped just after the tail of the shield passes the grout pipe location (1.5-2.0m beyond). Grouting immediately over the shield tends to deflect the skin of the shield or produce pressures on the shield that make it difficult to advance it.

The timing and frequency of grout placement should be decided after considering the location and magnitude of the potential ground losses over the shield and the degree of control required at the ground surface. The larger the ground loss over the shield and the smaller the differential movement required at the surface, the greater should be the frequency and volume of grouting and the less should be the lag time in injecting the grout.

In summary, they proposed the model of Figure 20 wherein:

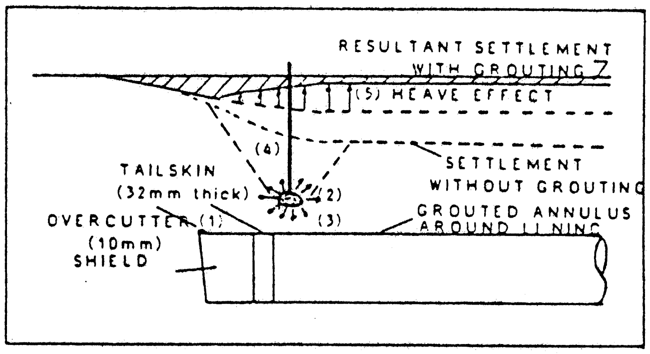


Fig 20 - Summary of the Effect of the Compaction Grout Bulb on Ground Movements Over the Tunnel (Baker et al., 1983).

Step 1 - Volume loss of 2-3 $ft.^3$/ft. (0.016 to 0.025m^3/m) over shield (overcutter void) plus 3-5 $ft.^3$/ft. (0.025-0.042m^3/m) at tail (tailskin void), gives total surface trough volume of 2 $ft.^3$/ft. (0.016m^3/m) without grouting.

Step 2 - Inject 6-12 $ft.^3$/ft. of grout over tunnel (0.05-0.10m^3/m).

Step 3 - Soil in immediate vicinity is redensified, soil ahead is displaced forward, soil below is recompacted, almost immediately replacing the 3-5 $ft.^3$ loss (0.08-0.14m^3) above the tail.

Step 4 - Increased pressures densify soils to above original value, shear deformations above bulb occur heaving soil up to 9m above crown. Zone is conical, extending 30°-45° from vertical.

Step 5 - Earlier surface settlements are reduced by heave from the bulb.

3.2.2 Hong Kong MTR Tunnels - Cut and cover methods were widely adopted during the construction of the first two lines of the Hong Kong Mass Transit Railway between 1976 and 1981. The third artery--the Island Line--was constructed between 1981 and 1986. Its route largely coincides with the densely populated fringe of the north foreshore of Hong Kong Island itself, and so the 10.5km of new subsurface track was created in 8m diameter bored tunnels 25-35m below ground surface. Throughout the MTR network, chemical grouting by the tube a manchette system has been a common feature of the construction, since its introduction during the construction of the trial tunnels in 1973 (McFeat-Smith et al., 1985). Ground treatment for strength and/or water tightness has thus been conducted in a variety of applications. These include lengths of ground adjacent to access shafts where tunnel drives were to commence in free air, prior to the application of compressed air. Ground treatment has been used to reduce compressed air losses (or eliminate its use altogether), in conjunction with both NATM and shield tunnelling. Similarly, at junctions to stations where diaphragm walls have been constructed, the method has been successful in preventing water ingress during junctioning. On the Island Line (Figure 21), ground treatment has been used extensively to minimize differential settlements to nearby multi-story buildings.

Much work has been conducted in medium coarse alluvial materials wherein methods and results have followed standard procedures, as outlined above. However, most of the treatment has been executed in the completely weathered granite whose geotechnical parameters would, by conventional standards, argue against the prospect of successful treatment by permeation techniques:

- initial permeablity of 10^{-5}m/s and less
- fines content over 40% (average 30%)

This success—McFeat-Smith et al., (1985) refer to "virtually impermeable" results—has been achieved through the following principal methods:

- high intensity of treatment: interhole spacings of 1-1.5m (av. 1.2m) were observed. In addition, most of the holes for tunnel grouting were inclined from the surface (for access reasons) and it is felt that there was an insitu reinforcing effect contributed by the rigid plastic or steel grouting pipes.
- routine use of stable cement grouts to "repair" any damage to the ground caused during drilling, permeate any coarse zones, and fill any major relict fissures in the decomposed mass. (Typical grout volumes: 6-10% of ground).
- chemical injection of large volumes (equivalent to over 30% of ground) of low viscosity (<5cp) relatively high strength (0.3M_{pa} in grouted sand) silicate gels with gel times up to 90 minutes to promote flow. In addition, the inherent arching strength of the decomposed granites permitted relatively high pressures to be exercised (over twice overburden) without risk of surface heave.

With respect to the performance of the grouting for curtains, Morton and Leonard (1980) reported on "reductions of between 300-1200 times for a 3-row curtain, over 40m deep. These measurements were calculated from water inflows into shafts, and not from borehole permeability tests, as it was reasoned that at such low permeabilities (of the order of 10^{-7}m/s) the borehole method was not sufficiently precise.

The uniformity of treatment was reported by Bruce and Shirlaw (1985) and Table 3 summarizes the results from one particular site. It is highly significant that 32 of the 43 grouted specimens had at least 20% fines, whereas none that did not react had less than 20%. Despite an average fines content of 31%, 78% of the samples showed positive reaction; i.e., had been thoroughly permeated with chemicals. This excellent performance--mirrored on other sites in the Territory--was achieved by the intensity and the thoroughness of the grouting, as detailed above. In addition, however, it did highlight a major anomaly in the way in which groutability, based on grain size analysis, is judged. In completely

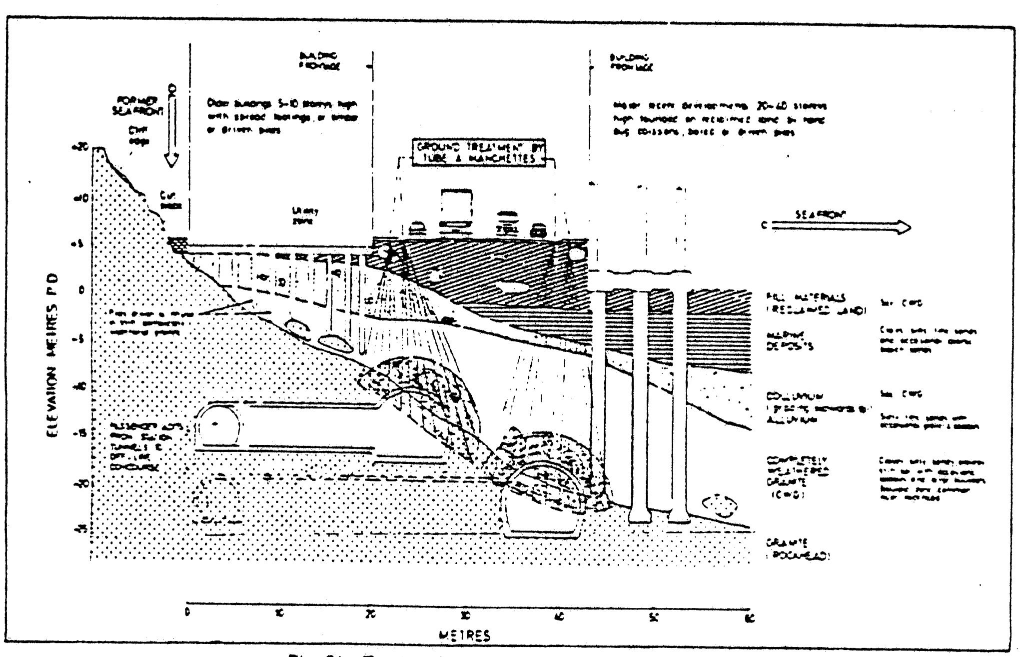


Fig. 21 - Typical Cross Section Showing Treatment of "Soft Ground" Sections of Rock Tunnels, Hong Kong (McFeat-Smith et al., 1985).

Fines in Sample,%	0-10	10-20	20-30	30-40	40-50	50-60
Reaction to test for presence of chemical grout:						
Positive, *(total, 43)	2	9	14	15	3	0
Negative, *(total, 13)	0	0	5*	2	4	2

* Includes two samples in region not regrouted following initial low pressure attempt.

TABLE 3 - Summary of Results Obtained on 56 Samples of Treated CWG (Bruce and Shirlaw, 1985)

decomposed granite, the fines result from the chemical weathering of the feldspars and micas. Insitu, these tend to remain in agglomerations, reflecting the original crystalline structure. Chemical grout attempting to permeate the cwg cannot flow into such agglomerations, and so treats each collection as a single large grain and flows around its boundary. On the contrary, these agglomerations break down when subject to the mechanical action of a sieve analysis, which then registers not one single "clump," as the grout encounters, but a large number of silt and clay particles. Such a grading curve would, therefore, indicate by convention an ungroutable soil.

[illegible]

The vast amount of ground treatment in Hong Kong has contributed much towards the current state of refinement of the materials and methods of tube'a manchette grouting. Equally, however, it highlighted that the traditional reliance placed by grouting engineers on grading curve analyses alone must be tempered by supplementary examinations of soil genesis as related to particle size distribution insitu.

3.2.3 Cairo Wastewater Tunnels, Egypt - Contract 3 of the Cairo Wastewater Project involves the construction of 16 circular shafts over a 5.8km length of bored tunnels. The general stratigraphy of this part of N.E. Cairo is described by Coe and Kay (1985):

- Fill (rubble in clayey matrix)
- silty clay and clayey silt
- dense interbedded clays, silts and sands
- dense to very dense sands and gravels.

The main 5m diameter sewer tunnels are being constructed using bentonite face support within a shield. Short lengths of smaller side tunnels are constructed by hand excavation in shields with faces supported by compressed air. Where tunnelling shields enter and exit from shafts there is the greatest risk of soil instability as the shaft wall is penetrated while tunnelling equipment is being erected or removed. Chemical grouting of the sands was specified to lend protective stability to these zones at 16 shafts.

Treatments were thus nominally cylindrical, concentric with the tunnel axes and generally about 8-15 meters long. The diameter was 4 meters greater than that of the tunnel to give a 2 meter thick annular protection outside and ahead of the shield (Figure 22). Some hand dug connections between shafts were pretreated over their whole length.

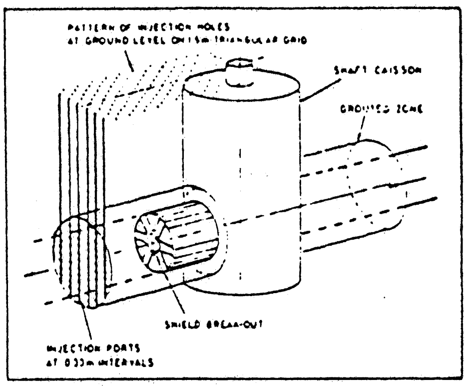


Fig. 22 - Typical Arrangement of Grout Zones and Injection Pipes, Contract 3, Cairo Wastewater Project (Greenwood et al., 1987).

Treatment was required only in the lower two units, the interbedded strata encountered intermittently at tunnel crown level, and the sands and gravels occurring through the core and below invert. The depth of treatment ranged from 10m-25m below surface depending on the size of the tunnel and location. The alkaline ground water was typically 2m-4m below the surface.

Clearly the ground treatment had to be extremely thorough to provide the desired effect in these saturated and variable but usually cohesionless sands. The success of the grouting philosophy of thorough permeation, in conditions indicated by the site investigation to be "borderline," demanded exceptionally stringent measures by the specialist contractor, a Joint Venture of Cementation Piling and Foundations, and STENT Foundations. This major achievement, as detailed by Greenwood et al., (1987), highlighted the following key factors which were evident throughout the year's operation in which approximately 5 million litres of chemical grout were injected:

• Ensuring Permeation: grout holes and injection points were installed at relatively close centres. No first phase cement based injections were conducted, (except as contact grouting around certain preformed shafts), as these could most likely cause hydrofracture planes, as opposed to notionally filling larger pore spaces.

Chemical injections were conducted over several successive phases (usually three), to target volumes equivalent to 35% porosity. Injection rates ranged from 15-20 litres/min. (Phase I) to 1-2 litres/min. at pump pressures of up to 0.5-0.6M_{pa}. The choice of the chemical grout "was restricted by economic and environmental restraints in addition to the engineering requirements." Most of the work featured a sodium silicate based grout reacted with citric acid and aluminum chloride, and giving a viscosity of 3cp for half its gel time (45-50 minutes). It had been estimated that grout particle sizes as small as 10μ would compromise penetrability and so the silicate as supplied had been filtered to 8μ. However, on mixing with the Cairo tap water, much larger flocculations occurred. Thus a centrifuge was employed to "clean" routinely the chemical components prior to mixing and injecting. In addition, particular care was taken (e.g., noncorrosive linings to tanks and pumps) to ensure that no extraneous particles would be otherwise incorporated in the mix.

• Control Over Batching and Injecting - Components were volume batched in small units to better than 0.2% accuracy in a fully automated station. Data from flow meters and pressure meters on each injection pump were displayed visually in the grouting module, and transmitted to a remote display in the control office. Thus the progress of the injection of each sleeve could be monitored (and varied if necessary, e.g., if hydrofracture was occurring), while a data logger/computer analysis system provided the basis for back analysis and hard copy issue.

The effectiveness of the grouting was proved directly during excavation by visual observation, and recording compressed air requirements. In addition, permeability testing was conducted before, during and after the treatment of each block (indicating final permeabilities consistently as low as 10^{-7}m/s). These supplemented the results of a precontract test block in which SPT testing was also conducted.

3.2.4 Singapore MRT Tunnels - The geology of the Island of Singapore is complex with several types of soils, such as beach, estuarine and fluvial deposits, marine clay and sedimentary soft rocks, occurring. In general, beach sand and fill, 3-5m deep, overlie very soft peaty clay, marine clay and fluvial soils to combined depths in excess of 15m. The base of this sequence is often marked by a layer of silty, fine sand overlying stiff to hard cohesive soils or weak rocks. The fine nature of these materials precludes the use of permeation grouting, and jet grouting is the preferred method of ground treatment wherever required.

Lot 106 of the Island's Mass Rapid Transit System runs between Dhoby Ghaut and City Hall stations. As the tunnels leave the former station (Figure 9) they pass in part through soft highly plastic formations in which the water level is 1-2m below surface. Without any soil improvement, even shield excavation was judged difficult, unsafe and unsuitable in the conditions. Treatment from the surface was feasible and economic.

In line with the design specifications, the grouting had to be extended to the full excavation area above soft rock or very stiff clay, and had to create an arch of strengthened soil 1.5-3.0m thick around the excavation. The thicker treatment was provided close to the station where the shield could not operate. To check the proposed solution and to set up the working programme, a large scale trial was carried out on site.

Two different layouts of jet grouted columns were tested--0.6 and 0.8m between centres of staggered elements. For each layout two different quantities of grout were injected (600 and 800 litres/m^3 of soil). The four combinations that resulted (including a total of 62 columns) were arranged to form the sides of a square area, excavated subsequently for visual inspection. The following general procedure was applied to each scheme: drilling to 10.5m depth, treatment from the bottom to 0.5m depth by the Rodinjet I technique, injecting a grout with w/c=1.6, and a grouting pressure of 40M_{pa}.

Instrumentation included inclinometers to check horizontal soil displacements, piezometers to record pore pressure variations and datum points to check vertical soil displacements.

The total volume of injected grout was 190m^3, which corresponded to 70% of the theoretically engaged volume of soil (270m^3). It is estimated that about 70m^3 of soil grout mixture was rejected during grouting and that the overall surface upheaval corresponded to about 60m^3 of upward displaced soil. Since no filling of natural voids can be expected in such a fine grained soil, it was inferred that the remaining 60m^3 (almost one-third) of the injected grout caused mostly radial displacement and compression effects.

Coring afterwards confirmed that even midway between column centers, the treated ground was over 50% above the specified minimum strength of 0.3M_{pa}. A test pit excavated inside the test area 15 days after treatment confirmed overlapping between adjacent columns, except for that group of tests with the higher spacing (0.8m) and lower grout volme (600 l/m^3). For the production work a 0.7m spacing was selected.

High surface movements were recorded--maximum horizontal displacement of 23cm, and vertical of 30cm--which necessitated variations to subsequent production parameters (always thereafter within safe limits--2-3cm).

Excess pore pressures in piezometers 6.5m deep and 3m and 6m from the perimeter of treatment were fairly low throughout (0.02-0.04M_{pa}).

The subsequent production works involved the successful treatment of about 9400m^3 of soil in four tunnel sections totalling 367m of running length. The interested reader is referred to the more detailed description by Cippo and Tornaghi (1985).

3.2.5 Milano Metro Tunnels Italy - The existing Milano Metro network of 54km of tunnels with 63 stations is being supplemented by the current construction of a further 28km (32 stations) of line. The work is being conducted in the centre of the city so that the former emphasis on cut and cover cannot be resumed. Instead ground treatment is being widely used (Mongilardi and Tornaghi, 1986), both from the surface and from underground, and solutions involving both permeation and jet grouting are being employed routinely. For example, for advancement by underground grouting, the following sequence is adopted:

(i) excavate adit shafts at 500m intervals, formed by diaphragm walls or jet grouting, as in Figure 10.

(ii) excavate a drift (a pilot tunnel) of cross section 9m^2 under cover of horizontal jet grouted columns (Figures 23 and 24).

(iii) treat the zone around this pilot by tube'a manchette permeation grouting.

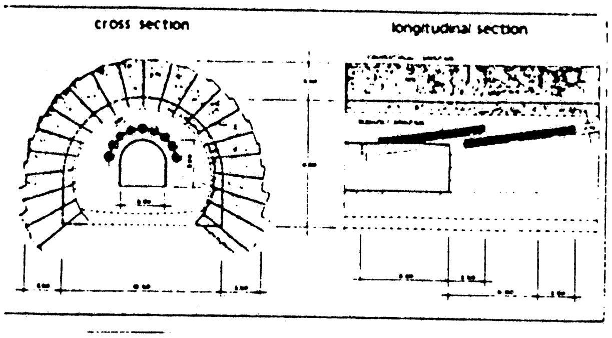


Fig. 23 - Principle of Tunnel Excavation by Forming Pilot (Protected by Horizontal Jet Grouting), to Allow Treatment (by Permeation Grouting) for Full Bore Excavation, Milano Metro, Line 3.

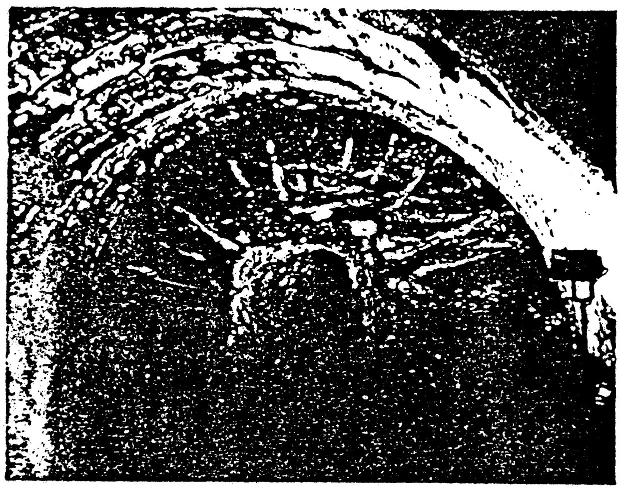


Fig. 24 - View of Pilot and Full Bore, Showing End of Jet Grout Columns, and Radial Tubes'a Manchette, Milano Metro Line 3.

(iv) excavate to full diameter, with the soil arch consolidated in step (iii) acting as protection.

Another novel factor, in relation to the previous Milano Metro work, was that strict restrictions were placed on the use of organic type reagents needed for higher strength silicate grouts necessary to provide ground treatment of adequate strength and durability. This led to the development of the Silacsol grout, as detailed above.

Equally, the scale of the project, the complexity of the design situations, the wide variability of the soil (gravel to silt) and the strict design specifications governing the use of chemical grouts, all combined to demand a new approach to site investigation and evaluation. Thus the PAPERO system of ground investigation and evaluation was developed.

As detailed by Mongilardi and Tornaghi (1986) the project involves a wide range of styles and combinations for ground treatment incorporating jet grouting (horizontal for adits, vertical from the surface for shafts, underpinning and earth retention) and tube'a manchette grouting. For the latter, cement based grouts are used throughout for routine consolidation, whereas chemicals are also injected for:

- underpinning of certain very sensitive buildings,
- impermeabilization where tunnel sections are below the water table,
- enhancing strength around the larger excavations.

The purpose of the grouting is one factor which dictates the geometry and sequencing of its execution. Other factors include:

- nature and possibility of surface access and presence and density of subsurface services,
- geometry and location of structure being excavated,
- programming and financial considerations.

Figures 25-29 illustrate various combinations which are being employed in response to these factors. It is significant to note that in Lot 2B a shield was initially used for the first 600m of pilot tunnel. However, it caused unacceptable settlements due to overbreak, at which time the construction reverted to protection by horizontal Rodinjet columns. The 9 columns in each pass average 9m long and 70cm in diameter, permitting excavation in 6m "runs." This system was used on the remaining 1300m of pilot on this section, and thereafter in the other lots.

Two other points are especially notable:
(a) chemical grouting: since the two components of the Silacsol system react so fact, they are introduced into the ground separately through twin tubes'a manchette; i.e., 2 small tubes are placed in each borehole for chemical grouting and the chemical reaction occurs in the saturated pores of the soil.
(b) strength testing of the jet grouted gravels confirms the mechanical properties of a fairly good concrete. Treated sands give a similar mean strength but with a significantly lower deformation modulus.

The overall value of the ground treatment works under contract to RODIO in this phase of the Project is well over \$100m spread over 5 years.

4. FINAL REMARKS

It is a time of rapid and significant advances in the field of ground treatment as applied to tunnelling schemes throughout the world. These advances are usually prompted by purely technical challenges, but are often dictated by stark economic reality. The direction of these advances is itself highly significant: more appropriate methods, improved materials, and far closer controls on the execution of the work. The trend is thus towards increased precision in the effectiveness of ground treatment as a routine construction process. In moving towards this goal, however, it is wise to recall the major potential influence of the natural variability of the ground--and the intensity and accuracy of its investigation--on a case by case basis.

With this caveat always in mind, the engineer may employ the methods described in this paper with a greater sense of confidence than ever before in their potentialities.

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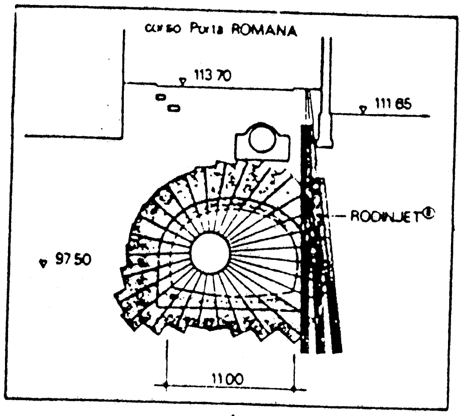

Fig. 25 - Radial Permeation Grouting From Pilot, and Jet Grouting From Surface to Protect Ancient Building, Milano Metro.

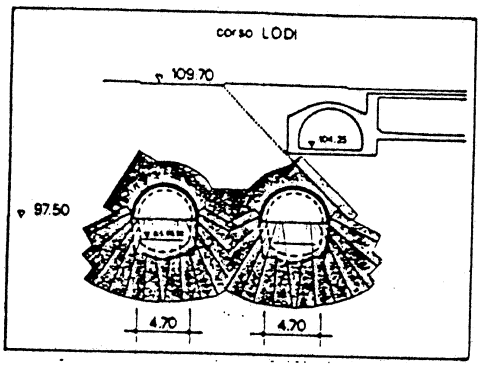

Fig. 26 - Permeation Grouting From Surface (to g.w.l.) Plus Steel Micropiles for Underpinning, Followed by Grouting from Pilot Under gwl, Milano Metro.

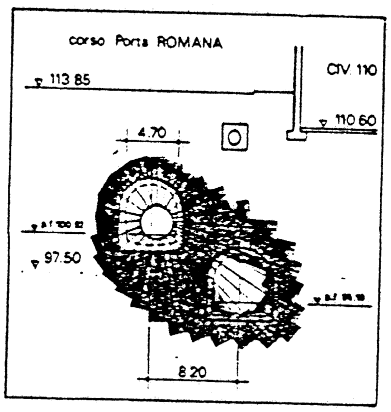

Fig. 27 - Permeation Grouting From Upper Pilot, to Permit Excavation of Two Parallel Tunnels at Different Levels, Milano Metro.

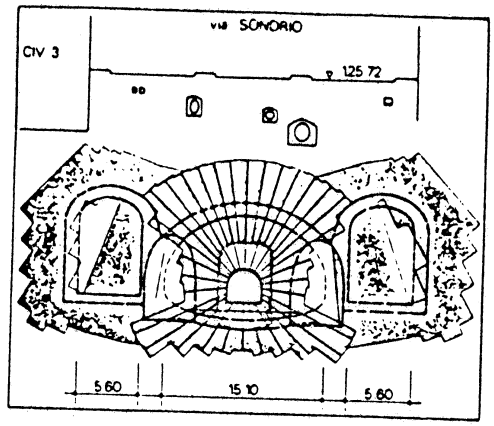

Fig. 28 - Treatment From Surface, to Protect Two Lateral Adits, Followed by Treatment From Central Pilot to Allow Excavation of Main Central Tunnel, Milano Metro.

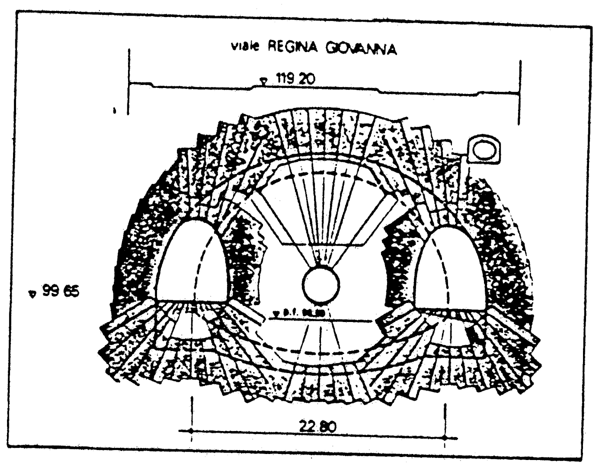

Fig. 29 - Treatment From Surface Followed by Treatment From Central Pilot and Two Lateral Drifts to Allow Excavation of Large Station, Milano Metro.

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