GROUND ENGINEERING

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Enhancing the performance of large diameter piles by grouting ...1

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Introduction

QUERULOUS EYEBROWS were raised in the 1975 Offshore Technology Conference in Dallas, Texas, when Gouvenot and Gabaix presented the conclusions of their test programme on the effects of postgrouting on large diameter pile performance:

- an increase in ultimate load of up to three times in sands and clays,
- similar increases in creep load,
- highly repeatable performance under cyclic loading with almost no permanent set, and
- direct relationship between ultimate load and volume of cement grout injected.

Since then, grouting techniques have been employed confidently throughout the world, as a routine construction process. By exploiting the higher values of skin friction and end-bearing resulting, engineers have been able to reduce pile dimensions safely. There are thus relative savings in plant and labour costs, as well as reductions in material requirements — a key logistical factor especially in developing countries.

Equally, where the satisfactory performance of conventional piling has been threatened, either due to faulty or inappropriate installation techniques (e.g. Logie, 1984), or due to interference by later construction activities (e.g. Karol, 1983), grouting as a remedial process has gained widespread application.

This review is a synthesis of data published primarily in the last decade on pile grouting. Case histories are presented in three basic categories:

(i) Enhancement by devices placed *within* the pile (i.e. attached to the reinforcement prior to installation).
(ii) Enhancement by toe/ground grouting (i.e. after concreting), and
(iii) Enhancement by ground consolidation (either before or after construction).

Throughout, the term "grouted pile" refers to a pile which has been treated in one of these categories. It is not used in distinction to the term "concrete pile" for example.

Enhancement by devices placed within the pile

Such devices have been developed to improve both shaft friction and end-bearing. The use for the former has been encouraged by the success of such as Ostermeyer (1974) and Jones & Turner (1980) with respect to albeit smaller diameter anchorage and minipile systems. On the other hand, the need to improve end-bearing performance, primarily to minimise the settlement required to fully mobilise it, has triggered intensive practical research.

Bearing in mind that shaft friction is fully mobilised at displacements of 0.5-1.0% *D*, but that end-bearing is maximised only at displacements of 10-15% *D*, it is clear that the service settlement criteria will usually govern the design basis, and that the frictional component is likely to be fully mobilised.

Indeed, Lizzi (1981) noted that there is even a tendency to over-design pile lengths to thereby ensure adequate capacity in shaft friction alone, reflecting fears that disturbance of the pile toe zone by pressure relief or by upflow of ground water during construction may seriously reduce the end-

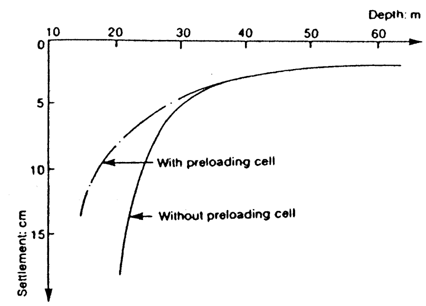
TABLE I: TYPICAL BASE GROUTING VOLUMES
(Bolognesi & Maretto, 1973)

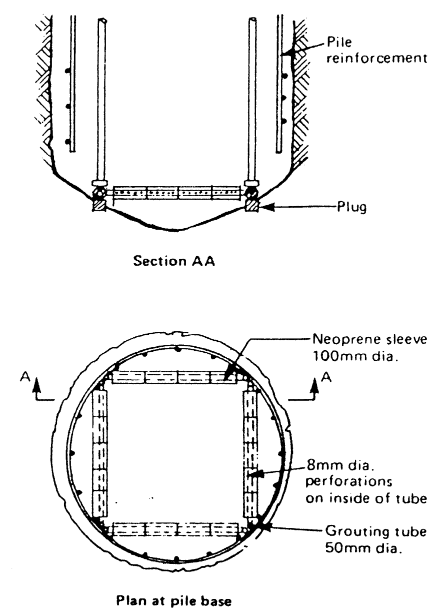
Typical grout takes

Pile diameter, m	Average number of stages	Grout take kg of portland cement
1.00 (SS.RC)	2	500
1.20 (SS.RC)	3	700
1.80 (RCS.RC.LG)*	3	3500
2.00 (WS.BA)	5	6000
2.00 (SS.BA)*	Still experimental	

SS: Steel Shell.
RCS: Reinforced concrete shell.
WS: Without Shell.
RC: Reverse circulation.
BA: Bucket augering with drilling mud.
LG: Lateral grouting to increase skin friction of pile shaft.

*Piles in the river course





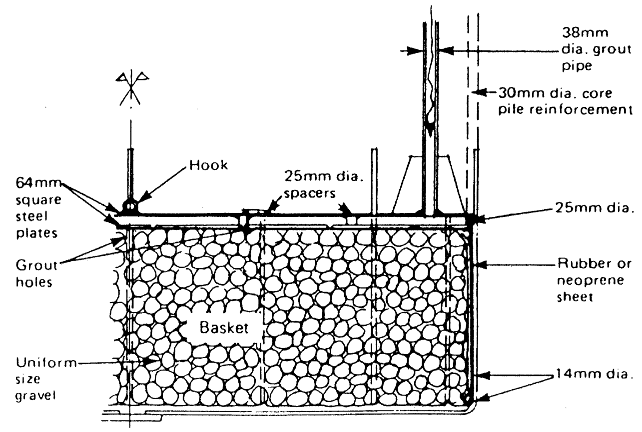


Fig. 1 (right). "Preloading cell" grouting device, Parana River (Bolognesi & Moretto 1973)

Fig. 2 (top, right). Estimated settlement of hotel tower as a function of pile toe depth, Cairo (Piccione *et al*, 1984)

Fig. 3 (centre, right). Details of base grouting device, Thailand

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bearing potential.

Such disturbance to the underbase material is almost impossible to eliminate completely. However, the effects may range from not being of "practical importance" in soft rocks or clays (Sliwinski & Philpot, 1980), to the other extreme: Sliwinski & Fleming (1984) concluded that "if a pile is founded in cohesionless soils which are likely to be disturbed, with a loss of the original compaction, consideration should be given to reducing or *ignoring* the end-bearing, unless pressure grouting is used to restore it."

In addition to problems caused by base instability, end-bearing can be further compromised by accumulations of sediment preventing the "clean butt joint" of concrete and bearing material referred to by Sliwinski & Philpot (1980), and by poor concreting procedures. Endo (1977) notes that such accumulations may be up to 400mm thick at the pile centre, and twice that at the perimeter.

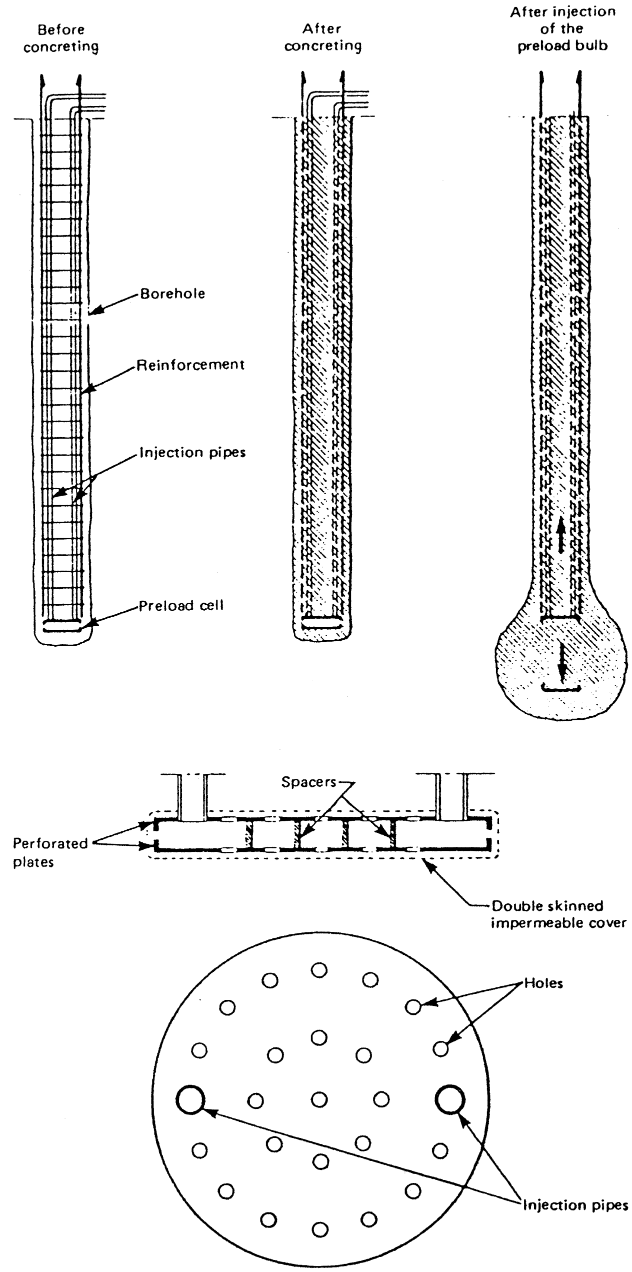
According to Sliwinski & Fleming (1984) pregrouting of pile bases via grouting cells was first reported in 1961 at the Maracaibo Bridge site, whilst the similar system later used for bridge foundations on the Parana River has been described in detail by Bolognesi & Moretto (1973). This project involved several hundred piles up to 75m long and 2m in diameter, each of which was "preloaded" through a basal-cell (Fig. 1). Each pressure cell had 40 holes, protected by a rubber membrane with an equal number of offset holes, thereby acting as a one-way injection valve. One grout line from the surface extended to each quadrant of the cell. Suspended below, and fixed to the pile reinforcement, was a basket of coarse gravel serving as a "grout and pressure distribution chamber". A neat grout mix of w/c ratio 0.66 (by weight) was used in several phases of injection typically 12 hours apart. The target volume for each phase was 500-1 500kg, depending on pile diameter, and grouting was repeated until either the target pressure of 100 bars was held for 5 minutes or a pile uplift of 20mm was recorded. Table I summarises the typical grout takes. The authors noted that the technique not only improved the characteristics of the soil below the toe, but it also served to appraise the development of skin friction along the shaft.

More recently, Piccione *et al* (1984) described a similar application at the El Gazira Sheraton Hotel in Cairo. 76 piles 1 800mm dia. for the critical lower structure were cast into very dense fluviatile sands ($N \geq 50$). Individual working loads of 850 tonnes were required for the 27m long piles, cased to 10m and drilled with bentonite flush therebelow. Fig. 2 compares the total (calculated) settlement with and without the cell. (The calculated ultimate end-bearing resistance was 1 710 tonnes (Berezantzev's formula)) and side friction was 226 tonnes (Kerisel's formula). Given the shorter pile length and different soil conditions from the Parana contract, maximum grouting pressures were limited to 12 bars, in theory sufficient to fully mobilise the shaft friction. Typically grout takes were 500-1 500kg, with the lower value yielding the volume necessary to fill the gravel pack voids.

Individual cell injections took between 2 and 5 hours to complete. All piles proved completely satisfactory in service.

Most recently, the engineers of a major

Fig. 4 (right). The "preload" cell (Lizzi, 1981)



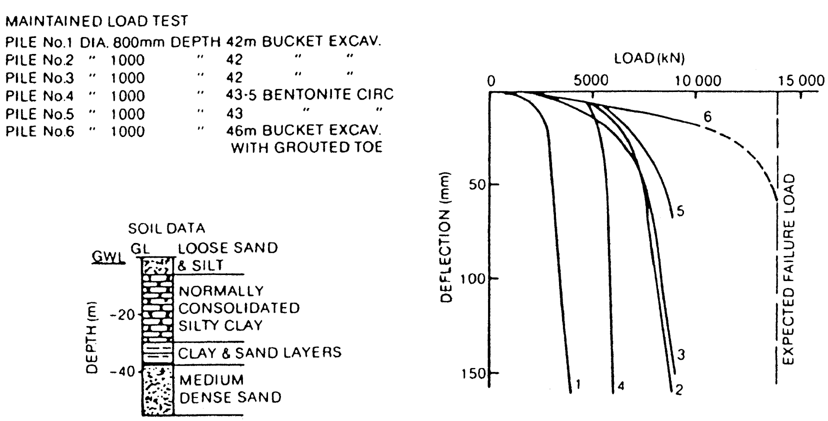


Fig. 5. Results of experimental pile tests, Italy (Sliwinski & Fleming, 1984)

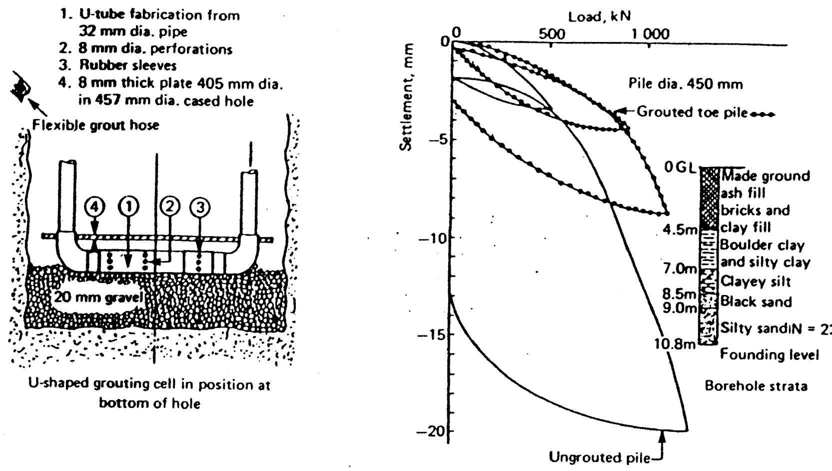


Fig. 6 (right). Details and results from Cementation experimental grouting cell (Sliwinski & Fleming, 1984)

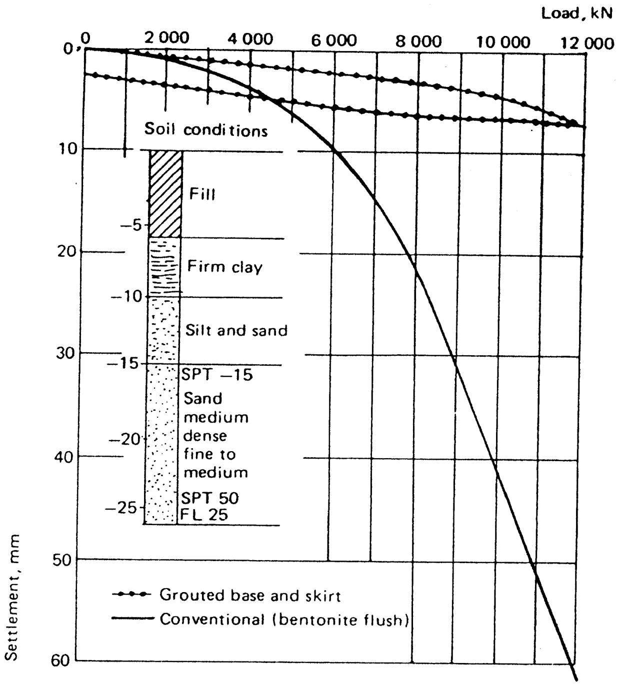


Fig. 7 (above). Comparative performance of two 1 500mm diameter bored piles, on the same site, one grouted, one conventional
(Sliwinski & Fleming, 1984)

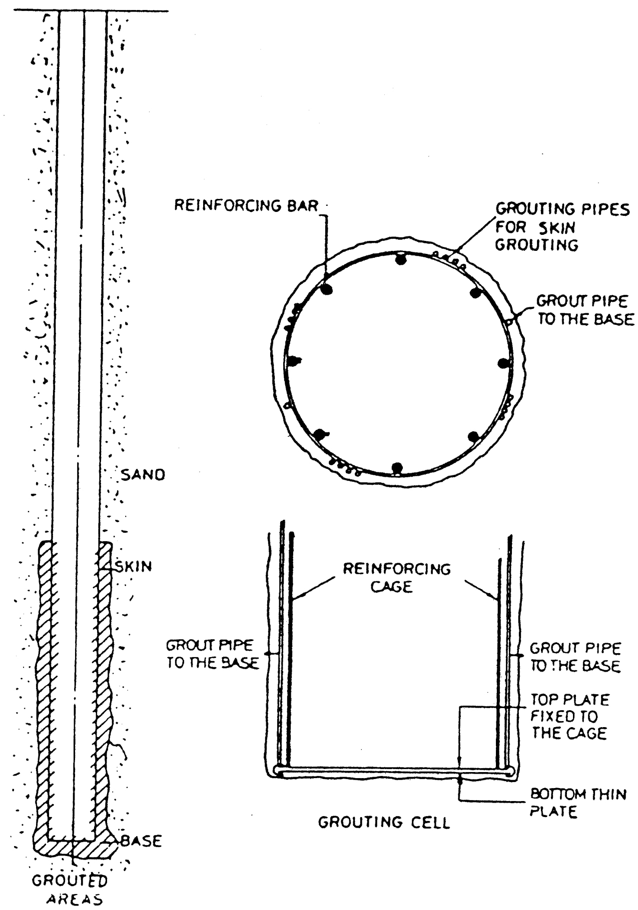
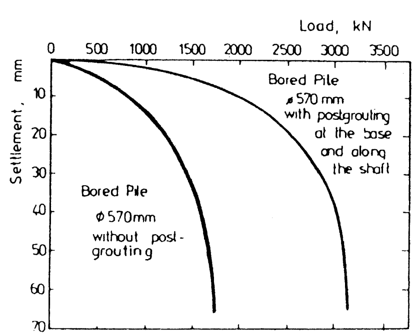


Fig. 8 (right). Arrangement for toe and shaft grouting (Stocker, 1983)

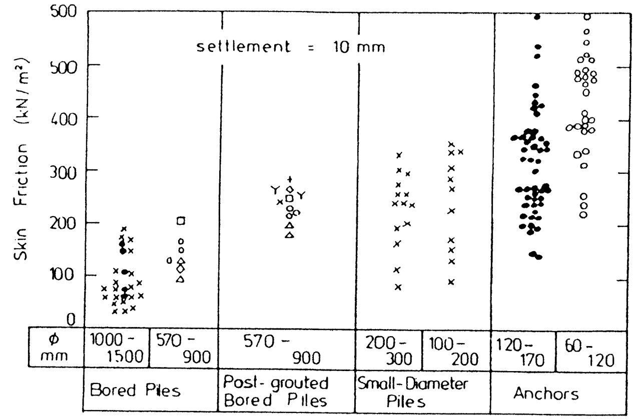
Fig. 9 (below, right). Performance of bored piles of 570mm diameter with and without postgrouting in sand with medium to high relative density (Stocker, 1983)

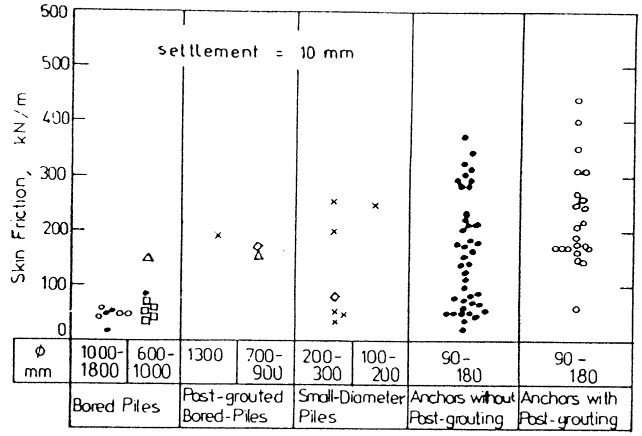
cable-stayed bridge in Thailand have reverted to a system similar to that of Fig. 1 but without the suspended cage, (Fig. 3). Here 172 bored piles of 2 000mm dia. up to 35m long are being installed under the piers for the bridge, and are founded in very compact fine to medium silty sand. Test piles of smaller diameter have proved that displacements of over 120mm are necessary to mobilise adequate end-bearing resistance, as a consequence of the hydrogeological and constructional circumstances. (No shaft friction is considered through the overlying soft clays). Grouting of the pile toes has therefore been specified to improve service performance.



Grouting is conducted in three separate stages (to 20, 40 and 60 bars, verified by grouting trials), and in such a way as to avoid hydrofracture of the ground and so damage to that or adjacent piles. Injections are made when the pile concrete is "relatively weak", (i.e. less than 2 days). Strict controls have been placed on the timing (more than one stage) and the spacing (> 4*D*) of adjacent treatments, whilst the maximum pile uplift is limited to 5mm.

Similar devices have also been developed by Lizzi (1981) (Fig. 4), whilst Sliwinski & Fleming (1984) cite test data from "experienced specialist contractors in Italy" (Fig. 5) and from their own company's





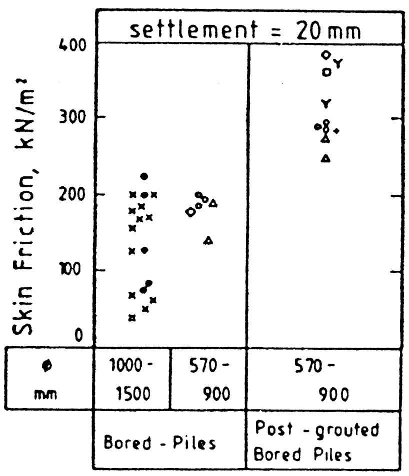


Fig. 10 (above, left). Skin friction of cast-in-situ bored piles (at a settlement of 10mm) and anchors in non-cohesive soils (Stocker, 1983)

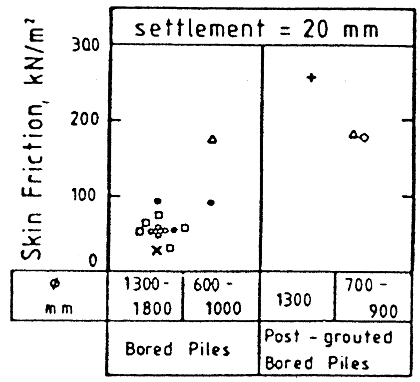
Fig. 11 (left). Skin friction of cast-in-situ bored piles with and without post-grouting at a settlement of 20mm in non-cohesive soils (Stocker, 1983)

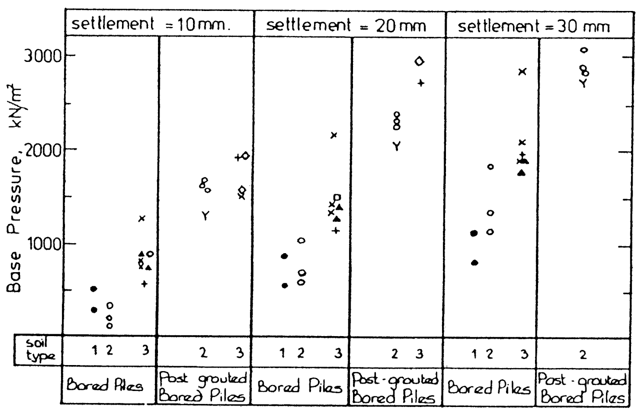
Fig. 12 (top, right). Skin friction of cast-in-situ bored piles (at a settlement of 10mm) and anchors in cohesive soils (Stocker, 1983)

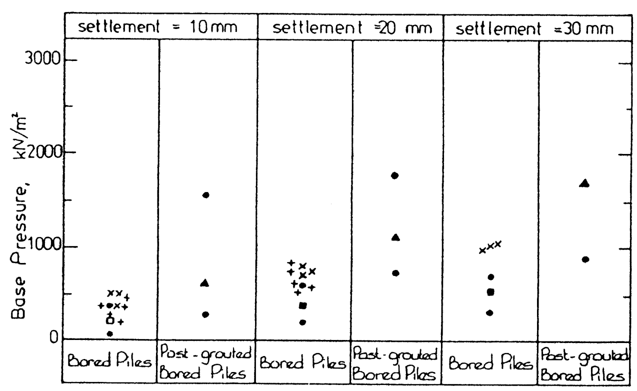
Fig. 13 (right). Skin friction of cast-in-situ bored piles with and without post-grouting at a settlement of 20mm in cohesive soils (Stocker, 1983)

Fig. 14 (below, left). Base pressure of bored piles with and without post-grouting in non-cohesive soil at settlements of 10, 20 and 30mm (Soil type: 1 = sand, medium density, 2 = sand, high density, 3 = gravel, medium to high density) (Stocker, 1983)

Fig. 15 (below, right). Base pressure of bored piles with and without post-grouting in cohesive soils at settlements of 10, 20 and 30mm (Stocker, 1983)







experimentation "some years ago" (Fig. 6). The same authors also quoted results from "a company working in the Middle East" (Fig. 7) featuring systems designed to incorporate improvement in shaft friction as well. However, on this particular topic, the most comprehensive and quantitative contribution appears to have been made by Stocker (1983).

He described the tube à manchette/flat jack combination of Fig. 8, and recommended that to improve *shaft friction*, injection after 1 or 2 days, at pressures up to 50 bars, be conducted, repeatedly if necessary. For improvement in *end-bearing*, grouting at the same interval to 60 bars was executed. He emphasised that the head displacement during toe grouting operations must be carefully monitored and limited to 3mm: higher displacements could decrease the frictional resistance subsequently mobilised. He also reminded that the amount by which the bearing capacity can be increased depends on the type of soil and its virgin density. Typical data are shown in Fig. 9, whilst the results of Stocker's experimental data may be summarised as follows:

Effect on shaft friction

Non cohesive soil

Figs. 10 and 11 show results at settlements of 10 and 20mm respectively. (The elastic deformations were not considered). These results show increasing friction with decreasing diameter, ascribed to the effects of dilation and construction method. Stocker did emphasise, however, that the post-grouting of the large diameter piles was by high pressure at different levels, whilst minipiles and anchors were grouted at low pressure (5-6 bars) over their whole length, during installation.

Cohesive soils

Figs. 12 and 13 show results at settlements of 10 and 20mm respectively. A distinct relation between diameter and frictional resistance is apparent, with a major improvement made by post grouting (supported by data of Ostermeyer (1974) and Whitaker & Cooke (1966)).

Effect on end-bearing

Non cohesive soil

Fig. 14. Although theoretical studies (Meissner, 1982) have shown a clear relationship between end-bearing capacity

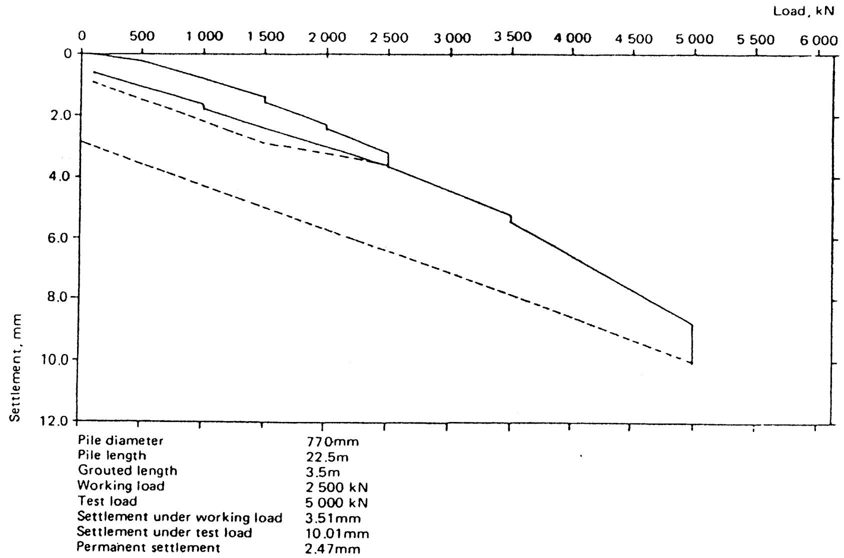


Fig. 16 (left). Pile load test results, Pile Test 1, Pier E-55, Jeddah (Bauer AG)

and pile diameter, this was not conclusively confirmed by Stocker's tests. A "very essential" increase of the base pressure was, however, attainable.

Cohesive soil

Fig. 15. These data are for piles from 620 to 1 800mm in diameter. From limited results it would appear that post-grouting increases the bearing capacity significantly (but less than in non cohesive soils) within the range of working loads considered.

To illustrate the practical application of the method, Stocker's company have reported on a project executed in Jeddah between 1981 and 1983 wherein 1 072 piles of 770mm dia. (working load 250tonnes) and 1 213 of 900mm dia. (working load 375tonnes) were installed. These piles support 300 piers carrying an elevated section of the Jeddah-Mecca Expressway. The soil consisted of loose silty sand and broken coral (–5 to –28m) overlying hard coral limestone. The piles ranged from 8 to 30m in length. Grouting was executed at the base, and along the shafts (in the dense sand). Fig. 16 shows the excellent test loading results of a 22.5m long pile, 770mm in diameter, tested to twice working load (i.e. 500tonnes).

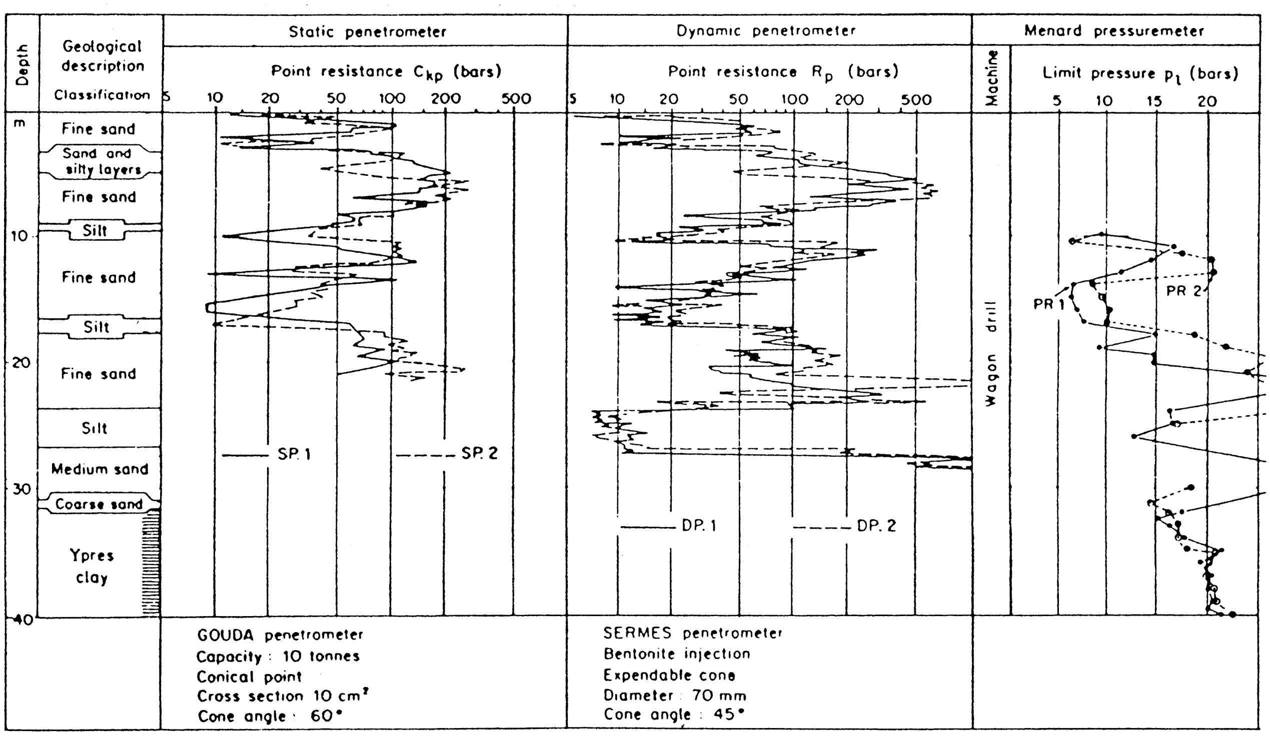


Fig. 17 (above). Summary of site investigation data, N. France (Gouvenot & Gabaix, 1975)

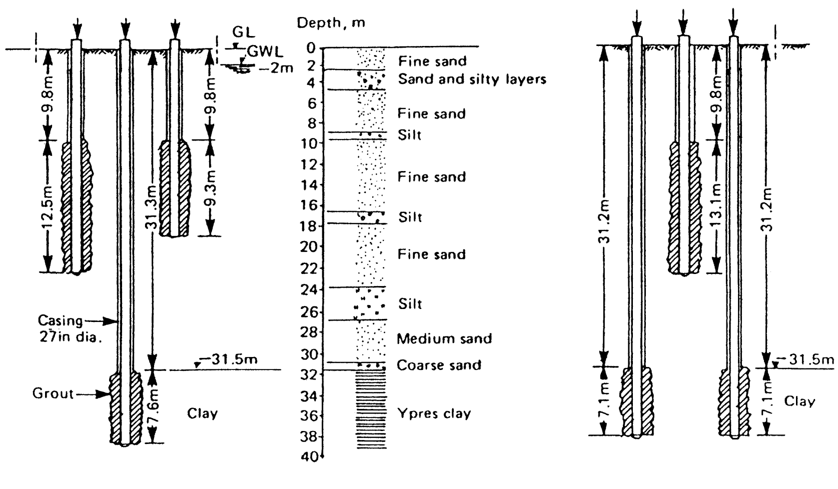


Fig. 18 (left). Location of test piles, N. France (Gouvenot & Gabaix, 1975)

TABLE II: SUMMARY OF GROUTING OPERATIONS (Gouvenot & Gabaix, 1975)

			Injection I_1			Injection I_2			Total	
Pile number	grouted soil	Grouted length in metres	Quantity injected in tonnes	Mean grout pressure at the end of grouting in bars	Quantity injected per metre of grouted length in t	Quantity injected in tonnes	Mean grout pressure at the end of grouting in bars	Quantity injected per metre of grouted length in t	Quantity injected in tonnes	Quantity injected per metre of grouted length in t
A_1	Sand	12.50	17.7	25-30	1.4	32.1	30-40	2.55	49.8	3.95
A_2	Clay	7.80	–	–	–	26.45	20-50	3.4	26.45	3.4
A_3	Sand	9.50	31.5	12-30	3.3	7.6	50-80	1.25	39.1	4.1
B_1	Clay	7.40	19.7	20-30	2.65	5.4	50-80	0.73	25.1	3.38
B_2	Sand	13.40	–	–	–	25.6	15-30	1.9	25.6	1.9
B_3	Clay	7.40	18.8	20-50	2.55	11.25	50-100	1.7	30.05	4.23

TABLE III: SUMMARY OF LOAD TEST RESULTS (Gouvenot & Gabaix, 1975)

Pile Nos.	E_0			E_1			E_2			Compression			Pull-out		
	Q_L	Q_F	$\frac{Q_L}{Q_F}$	Q_L	Q_F	$\frac{Q_L}{Q_F}$	Q_L	Q_F	$\frac{Q_L}{Q_F}$	Q_L	Q_F	$\frac{Q_L}{Q_F}$	Q_L	Q_F	$\frac{Q_L}{Q_F}$
A_1	200 (ex)	120	1.6	350 (ex)	220	1.6	520 (ex)	330	1.6						
A_2										Poorly defined > 300	260		Poorly defined > 300	> 300	
A_3	70 (r)	55	1.3	400 (ex)	> 300		420 (ex)	> 400							
B_1	180 (r)	120	1.5	370 (r)	240	1.5	440 (ex)	270	1.5				Grouting destroyed by loads on B_2		
B_2										450	300	1.5	160	140	1.14
B_3	180 (r)	120	1.5	440 (ex)	290	1.5	490 (ex)	> 400							

ex – Load found by extrapolation
r – Effective load without extrapolation

Q_L – Ultimate bearing capacity
Q_F – Creep load

All loads in tonnes

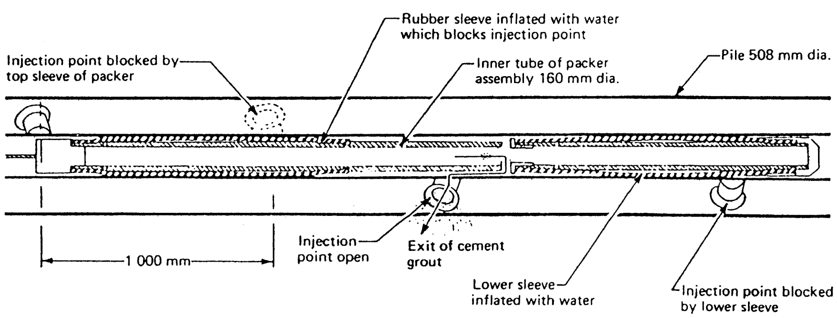


Fig. 19 (left). Post grouting device for pipe piles, N. France (Gouvenot & Gabaix, 1975)

The work of Gouvenot & Gabaix (1975) concentrated solely on the improvement of skin friction as a means of reducing offshore pile geometries. These tests followed Gouvenot's (1973) experiments with soil anchors where a 2-5 times enhancement of frictional resistance was attained. A six pile field test programme (Figs. 17 and 18) was conducted in fine-medium silty sands and Flanders clay. Over the uncased pile lengths, subsequently pressure-grouted, the diameter was 660mm, formed by rotary drilling with water and/or bentonite in the sands, and by auger and bucket in the clay.

Pile reinforcement consisted of a 508mm o.d. steel pipe of 11mm wall thickness, into which was attached an inner tube assembly of 160mm i.d. (Fig. 19), permitting subsequent pressure grouting at discrete 1m intervals, through non-return valves.

The initial phase of pile grouting (i.e. at gravity pressure, while installing) was referred to as I_0, whilst later pressure grouting operations were referenced I_1 and I_2. After each of these three phases, load testing was conducted (i.e. E_0, E_1 and E_2 data). Grouting results are summarised in Table II.

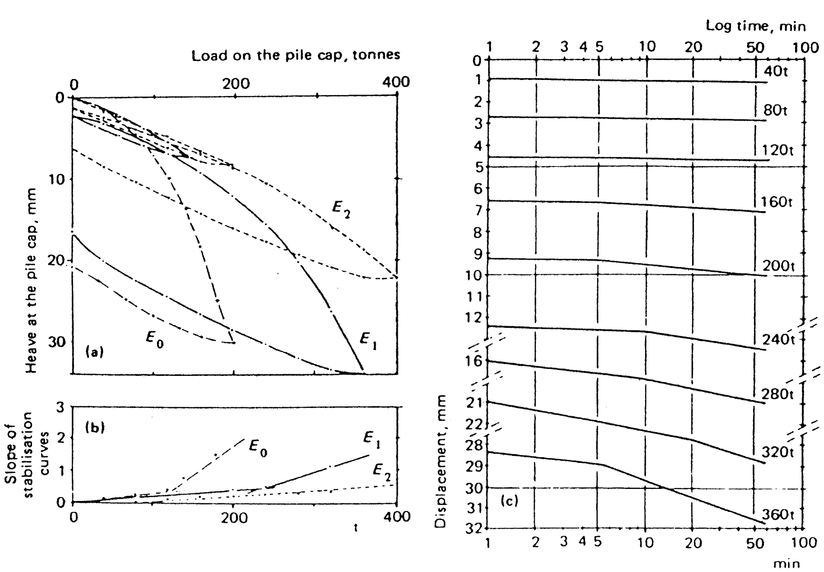


Fig. 20 (left). Performance of Test Pile A1 in sand, N. France (Gouvenot & Gabaix, 1975)

The substantial improvements in performance after each of the phases of pressure grouting are shown clearly in Figs. 20 (sand) and 21 (clay), whilst the results of the whole programme are summarised in Table III. As indicated in the introduction to this review, the authors were able to make several clear and important conclusions, principally:

(1) Q_L, the ultimate bearing capacity, is very sensitive to the effects of grouting — by a factor of 2 to 3.

(2) Q_F, the creep load, varies similarly, with the Q_L:Q_F ratio (1.3 to 1.6) independent of the grouting influence.

(3) The piles remained very stable under cyclic loading: after several cycles no permanent displacement remained.

(4) The increase in Q_L is directly proportional to

$$\sqrt{\frac{(V+V_p)}{(V_o+V_p)}}$$

for both clay and sand (Fig. 22) assuming controlled pressure and volume (i.e. no claquage),

where V = volume of grout injected

V_p = volume of pile

V_o = volume of grout used for initial gravity grouting.

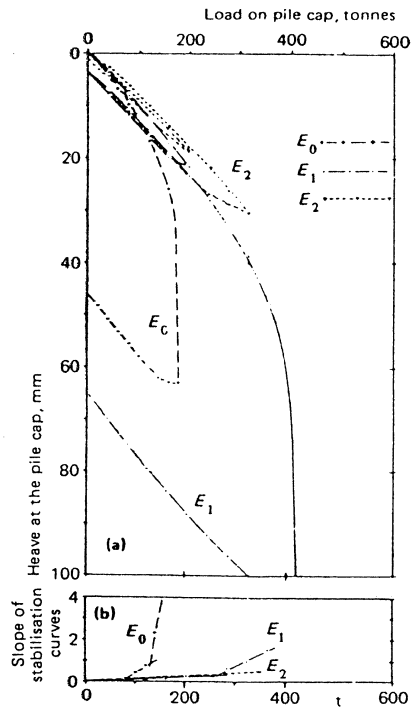
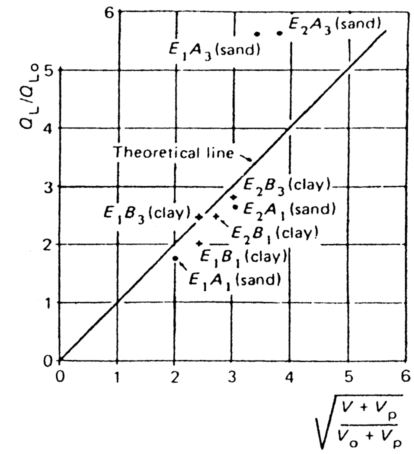


Fig. 21. Performance of Test Pile B1 in clay, N. France (Gouvenot & Gabaix, 1975)



Q_L = Ultimate bearing capacity of the pile

Q_S = Effective grouted diameter

A = Coefficient characteristic of the pile's geometry (grouted length)

$Q_L/Q_{Lo} = \frac{\text{Ultimate bearing capacity after pressure grouting}}{\text{Bearing capacity before grouting}}$

V_p = Pile volume

V = Volume injected under pressure

V_o = Volume filled without pressure

Fig. 22. Inferred relationship between grout volumes and enhancement of pile performance (Gouvenot & Gabaix, 1975)

Enhancing the performance of large diameter piles by grouting . . . 2

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Enhancement by toe/ground contact grouting

When long piles are constructed through very difficult and variable soft ground conditions to bear on hard fresh rock, it is understandable that problems are occasionally encountered with soft sediment inclusions at the bases. Despite due skill and attention on the part of the contractor, circumstances often conspire to defeat the intention to provide absolutely clean bases and intimate, continuous pile-rock contact. Equally, in cohesionless soils under high hydraulic gradients, the natural resultant base instability will confound the best efforts to remove disturbed sediment. Occasionally, other constructional problems may occur, such as delays prior to concreting allowing settling of suspended sediment from the flushing medium.

The presence of the resulting soft and highly compressible inclusions is detected by coring, sonic testing, load testing and/or analysis of construction records and inevitably the first reaction is to condemn the pile. However, the author has been involved in a number of recent cases where successful remedial works have been put in hand, the costs of which have been minimal in relation to the costs involved in pile replacement (which has no guarantee of more satisfactory behaviour, and which may simply not be feasible given physical or logistical restraints).

Understandably, the publicising of such schemes is not generally encouraged and so Logie's Paper (1984) has particular relevance. Although the details vary from site to site, the basic philosophy and approach are both typical and well proved.

1 000mm dia. piles were designed for the structures comprising the Jakarta Mandarin Hotel, Indonesia. They were founded in the Lahar Formation (cemented sandy silt and silty sand associated with volcanic terrain). This formation is 8-18m thick and is the favoured local foundation horizon, isolated by soft to stiff cohesive fills and alluvium above, and silts, sands and clays below. Although the designed working load was 300tonnes, results from a test pile reduced this to 270tonnes to ensure total and differential settlements within "tolerable limits". On this basis, the 284 production piles were installed. However, load tests on selected piles were either marginally acceptable (sf 1.93) or failed entirely (Fig. 23). Investigation ensued.

Review of these test data, plus analysis of the construction records led to the conclusion that unconsolidated debris existed between the base of the failed piles and the competent Lahar. The frictional resistance of the piles had been exceeded at about 175tonnes. No end-bearing contribution had by then been mobilised due to the high compressibility and low strength of this unconsolidated material (110mm and 590mm thick in the two failed piles respectively). Possible explanations for the presence of these zones included:
- inefficient concreting procedures,
- inadequate cleaning,
- deterioration and spalling of borehole walls in the period prior to concreting,
- dislodging of material during cage homing, and
- delays between cleaning and concreting (thus allowing settling out of sediment).

Coring in 48 other piles showed average

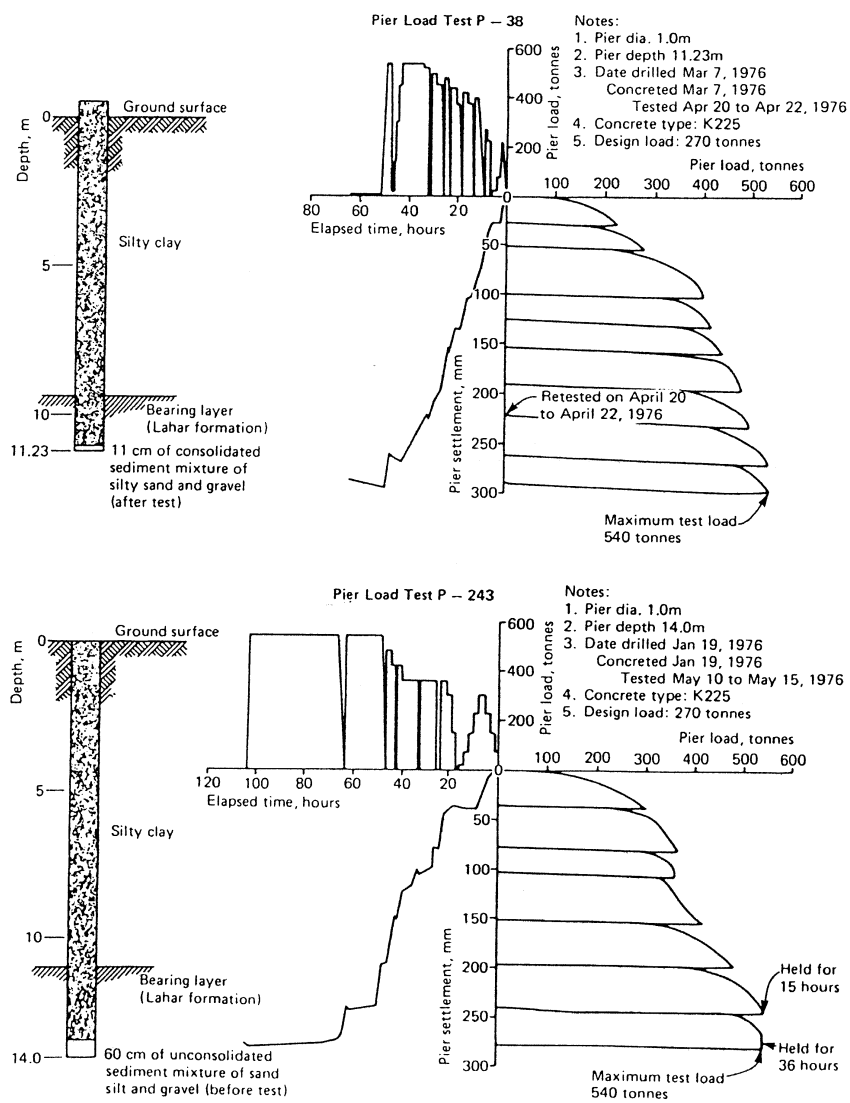

Fig. 23. Unsuccessful test loadings, Jakarta (Logie, 1984)

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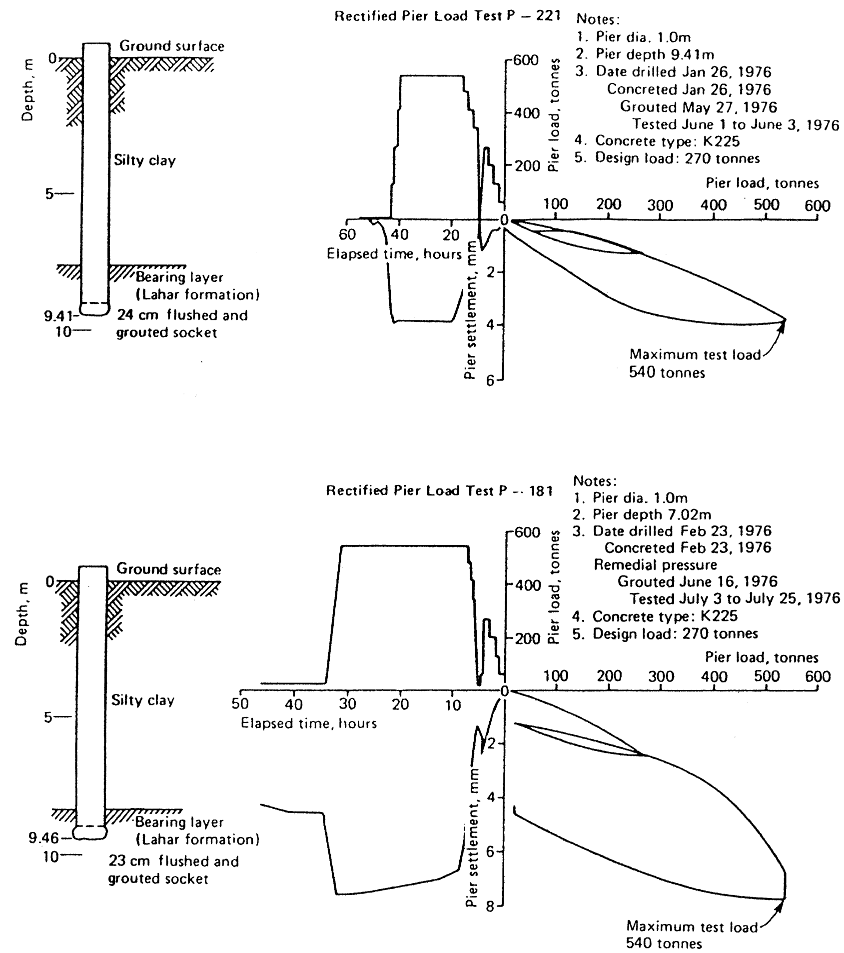


Fig. 24. Successful test loadings, following grouting, Jakarta (Logie, 1984)

sediment thicknesses of 160mm (max. 640mm). The other piles were explored by percussive drilling.

The two favoured options for rehabilitation were:

(a) Preloading each pile to a load in excess of the design capacity, and

(b) Removal of unconsolidated sediment below the piles by flushing with air and/or water followed by the filling of the resultant void by pressure grouting with cement.

The second method was determined to be the most expedient and cost-effective.

After field tests, a system employing a 25mm dia. closed-end steel flushing pipe with four horizontally directed nozzles was adopted, operating down a 73mm central hole drilled in each pile. A constant pressure (5-7 bars) water pump was used, and this system proved far more effective than a combined low pressure air/water intermittent jetting alternative. In addition, the use of high pressure air injection only had led to "bubbling" around adjacent piles and so the possible decrease in their frictional resistance. The volumes of excavated sediment were measured by a down-the-hole calliper. Records showed that in over 70% of the cases the estimated sediment volume and the subsequent grout takes were within about 100litres of each other.

TABLE IV: EARLY PILE TEST RESULTS (from Logie, 1984)

Pile	Net (plastic) displacement (mm)	Net total displacement (mm)
P221	2	4
P181	5	8

Sulphate-resisting cement was used with a non-shrink additive, producing a very fluid, workable, and stable grout of w/c ratio = 0.5. Tremie grouting procedures, through the central cored hole, were used to ensure displacement of any remnant sediment and ground water. Pressure grouting was then conducted from the top of the pile, to a maximum pressure of 30 bars. Migration of grout to adjacent piles was occasionally recorded and combatted by usual grouting practices, including 30 minutes "standby" periods.

Results (Table IV) from two early piles (Fig. 24) showed excellent performance at 540tonnes (i.e. 2 *WL*).

However, a third pile (P-62) which had shown "suspicious" grouting characteristics (i.e. anomalously low grout take) proved unsatisfactory at 400tonnes (22mm net, 25mm total settlement) and this prompted a re-evaluation of the programme. This isolated 74 "suspect" category piles (on basis of grout take less than sediment volume) out of 137 completed to that point. 49 were cored and 22 showed no sediment, 17 suffered core loss and were reflushed and regrouted as necessary, whilst 6 showed a thin layer of granular sediment 10-20cm thick and were preloaded to 400tonnes (all satisfactorily). The grouting had densified or partially intruded these materials.

The other suspect piles were preloaded to 400tonnes and by reviewing their characteristics, 25 were "cleared". The others were subjected to continuous pre-load pressure until the settlement stabilised (i.e. any sediment had been "appropriately" consolidated).

By the end of the project 209 out of 284 piles had required rehabilitation before being accepted as capable of supporting the design load of 270tonnes with an acceptable factor of safety. The subsequent behaviour of the structure was generally linear and uniform during construction with the total settlement being 16mm at a total structural dead load of about 33 000tonnes.

Enhancement by ground treatment

The complex task of describing the principles of ground treatment has been addressed comprehensively in recent years by FHA (1976), Cambefort (1977), Littlejohn (1983) and Karol (1983). In addition a wealth of relevant case histories and research findings is provided in the proceedings of specialist conferences, notably at New Orleans (1982) and Helsinki (1983).

Nevertheless, it merits repetition to state that the characteristics of the ground dictate fundamentally the basic grouting parameters, particularly the choice of grouting material. For example, if permeation of the surrounding or underlying soil is intended, then it is essential to quantify those parameters which can be interpreted to provide information on likely pore sizes, viz. particle size distribution, density and permeability. Ranges of grout types have thus been developed (e.g. Fig. 25) for different ground characteristics and purposes.

Where grouting must be accomplished by chemical (solution) type materials, it must also be ascertained that such materials are not environmentally hazardous, and that their effect may be regarded as permanent under the ambient conditions. Karol (1983) lists that in terms of "usable strength" (including a safety factor of 2), a range of values for preliminary design use (to be verified by tests) is:

Lignosulphonates	0.035-0.07N/mm^2
Low concentration silicates	0.035-0.105N/mm^2
Acrylamides	0.035-0.14N/mm^2
Phenolic resins	0.035-0.21N/mm^2
Amino resins	0.07 -0.35N/mm^2
High concentration silicates	0.14 -0.35N/mm^2

In general, the strength of grouted sands increases with increasing density and decreasing effective grain size (D_{10}). Well graded soils have higher strengths than uniform soils of the same effective grain size.

Ground treatment can aid pile performance in three applications:

(a) before piling, to also resist base instability during the subsequent piling operations,

(b) immediately after the piling process, to improve the base resistance, and

(c) during service, where the working environment of the piles is likely to be detrimentally affected by new adjacent construction.

(a) Ground treatment before piling

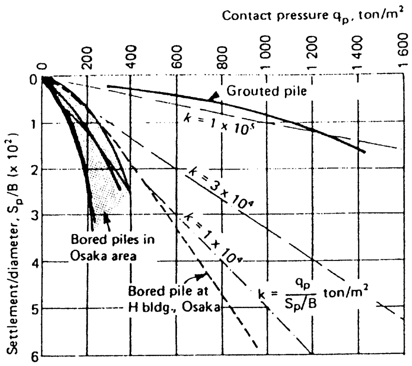
Endo (1977) described two examples in Japan. The larger project involved 340 piles up to 2 000mm in diameter for the Umeda H building in Osaka. A pregrouting method was adopted, featuring TACSS chemical grout to

TYPE OF GROUT			CRUSHING STRENGTH	RELATIVE COST (1 m³)	USAGE	GROUTING BEHAVIOUR
SUSPENSIONS	Unstable grouts	Suspension of cement in water (+ sand) W/C 1.5/1 or 1/1 to 10/1	Comparable with concrete	4.2	Fissured rock or masonary	Unlimited quantities but reaches "refusal" pressure
	Stable grouts (separation of a few percent)	Activated cements and mortars { Prepakt, Thermocol, Colcrete }	Comparable with concrete		Filling of large voids	Limited quantities
		Cement-clay (+ sand)	0.1 to 5 MPa	1	Wide fissures + sands and gravels $k > 5.10^{-4}$ m/s	
		Treated clay	< 0.1 kPa	1.1		
		Actisol	0.1 to 80 MPa	1		
LIQUIDS	Chemical based grouts	Silacsol	0.1 to 30 MPa		$10^{-7} < k < 10^{-4}$ m/s	Limited quantities
		Hard gels { Sodium Silicate { + $CaCl_2$ / + Ethyl acetate }	1 to 2 MPa (Mortar 4 MPa)	10.7 / 11	$k > 10^{-4}$ m/s	
		Hard gels { Lignosulphite + bichromate	0.03 MPa (Mortar 0.4-0.5 MPa)	6.5 to 8	$k > 5.10^{-5}$ m/s	
		Plastic gels { Sodium silicate + reagent / deflocculated bentonite	5 kPa / 1-2 kPa	2 to 4 / 1.8	$k > 10^{-5}$ m/s / $k > 10^{-4}$ m/s	
		Organic resins { AM 9 / Resorcin formol / Urea formol (acid grout)	< 0.1 MPa / 1 to 10 MPa / 2 to 10 MPa	50 to 130 / 10 to 40	For usual groutings $k > 10^{-6}$ m/s	
		Organic resins { Precondensed (epoxy) polymers	Comp. 100 MPa Tens. 30 MPa	150 to 500	Bonding cracks in concrete	
		Hydrocarbon binders { Bituminous emulsions { + silicate / + resorcin }	0.01 MPa / (Mortar 1 MPa)	6 / 12	$k > 10^{-5}$ m/s	
		Hydrocarbon binders { Hot bitumen	Highly viscous liquid		Major water flows	

Choice of grout according to the permeability of soil.

GROUT			S : Strengthening or W : Waterproofing	Initial permeability of soil k m/s: > 10^{-1}	10^{-1}–10^{-2}	10^{-2}–10^{-3}	10^{-3}–10^{-4}	10^{-4}–10^{-5}	10^{-5}–10^{-6}	< 10^{-6}
CEMENT			S	■	◧					
CLAY - CEMENT			W-S	■	■	■	◧			
CLAY GEL deflocculated and solidified BENTONITE			W				■			
ACTISOL			W-S	■	■	■	◧			
SILICA GEL	strengthening	concentrated	S				■	◧		
		low viscosity	S					■	◧	
	waterproofing	concentrated	W				◨	■		
		very dilute	W					■	◧	
SILACSOL			W-S				◨	■	◧	
RESINS	ACRYLAMIDE		W						■	■
	PHENOLIC		S						■	■
SILACSOL (solution)			W-S						◨	■

Fig. 25. Characteristics, applications and suitability of principal grouts (Soletanche technical brochure, 1984)



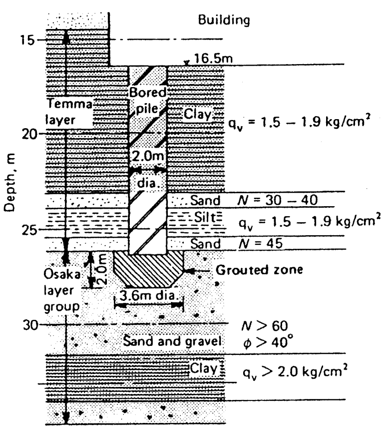


Fig. 26 (left). Details and performance of piles, Osaka (Endo, 1977)

"completely eliminate" loosening at the pile tip, and so ensure superior pile performance (Fig. 26). For the given soil parameters, the UCS of 17 samples extracted from treated ground ranged from 3.7 to 4.7N/mm² (av. 4.2), and a safe end-bearing pressure of 3.5N/mm² was used as the basis for the subsequent pile design and construction.

Normally of course, piles bearing on materially competent bedrock require no special pretreatments. However, where such piles are founded on strata suspected to contain significant voids e.g. in karstic limestone or in gypsiferous caprocks, then pregrouting is a common solution. Thus, many projects in certain areas of the Middle East have featured methodical programmes of probing and infilling at and between pile locations to restore the mass integrity of the ground. In this way also the possibility of sudden, massive losses of flushing materials or concrete, into unfilled cavities is markedly reduced, and so more regular piling progress results. It is also clear that the scepticism which formerly greeted the prospect of conducting such pretreatments (the costs of which are seldom allowed for at tender stage) has been replaced by a ready acceptance, arising from a sad and expensive catalogue of delayed completion dates and unusable superstructures.

A different approach to combatting the effects of major voids has been described by Satiropoulos & Cavounidis (1979), based on successful applications at the Aherou/Vovopotamos bridges, and for a building at Kastoria, both in Northern Greece. Concerned at the real prospect of these vital large diameter end-bearing piles punching down through voids in otherwise strong limestone, each location was first probe-drilled to 3 or 4m into the rock. Where voids were identified, the following procedure was adopted:

– excavate and case pile, and toe in 600-800mm,
– place reinforcement cage, into which is fixed two 300mm dia. tubes (Fig. 27),
– concrete pile and extract casing,
– drill through each tube and a minimum of 1m below the cavity,
– place I beam (with canvas formwork), from bottom of drill hole to at least 1m above pile toe, and
– concrete/grout each tube, simultaneously inflating the fabric formwork within the cavity, to about 600mm diameter.

The method proved practical and satisfactory.

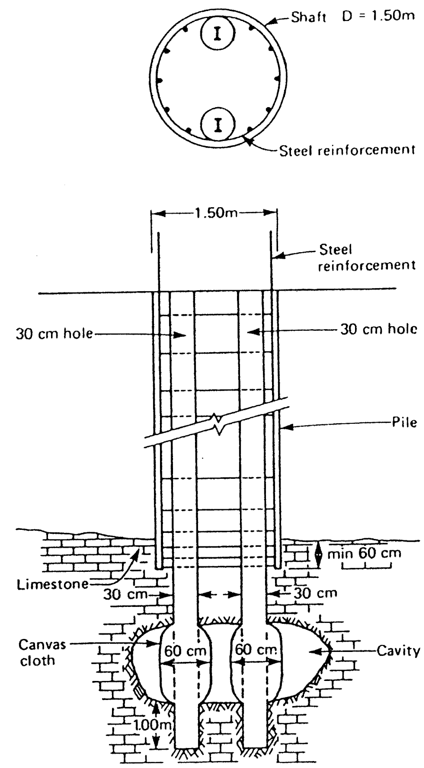
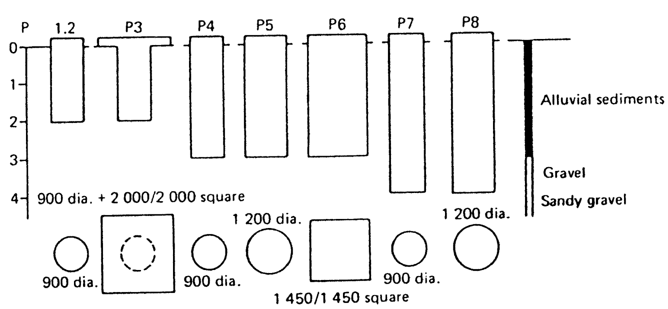


Fig. 27. Details of piling solution to overcome Karstic voids, N. Greece (Satiropoulos & Cavounidis, 1979)



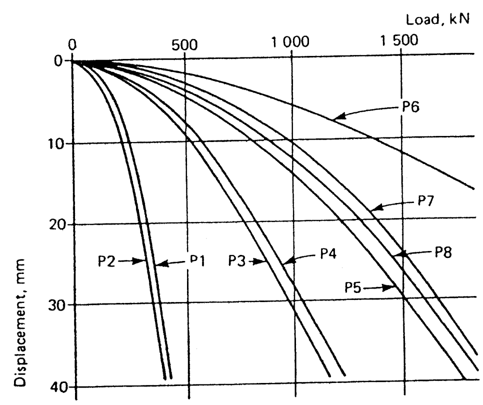


Fig. 28 (above). Performance of test piles, Bratislava (Cernak *et al*, 1983)

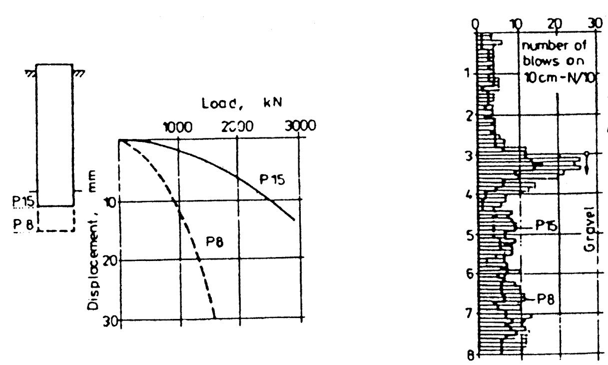


Fig. 29 (left). Performance of conventional pile P8, and grouted pile P15, Bratislava (Cernak *et al*, 1983)

(b) Ground treatment immediately after piling

Two case histories, of contrasting sophistication, are cited.

Cernak *et al* (1983) reported on routine measures taken to improve the end-bearing properties of simple, shallow, 900mm dia. piles at Bratislava, Czechoslovakia. The founding strata consisted of sandy gravels of highly variable density and permeability, vertically and laterally.

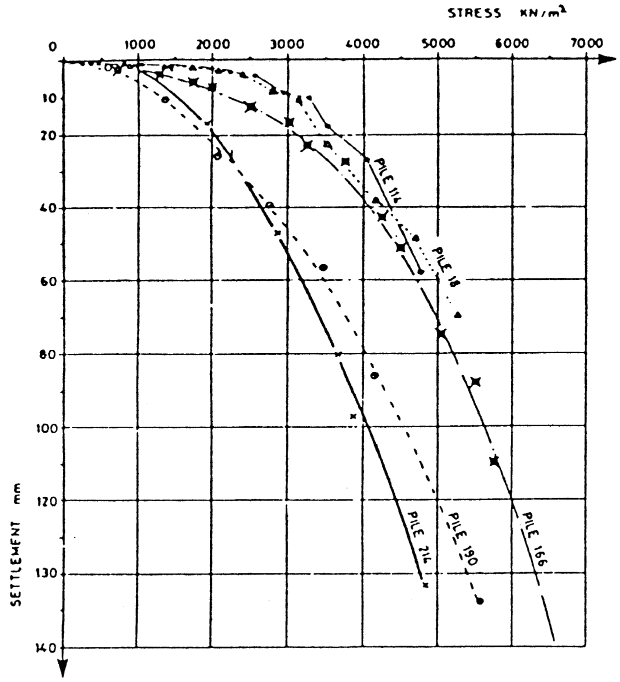
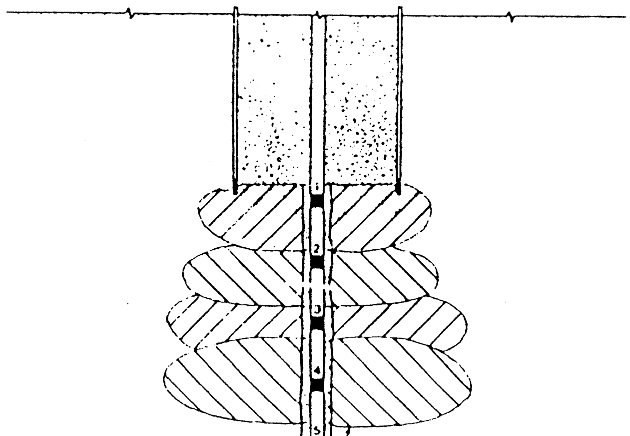


Fig. 30. Pile test results, Jeddah (Littlejohn *et al*, 1983)



Production Grouting Procedure	
INJECTION SEQUENCE	GROUT VOLUME (litres)
4	600
1	200
2	200
3	200

Fig. 31. Tube à manchette installation and production grouting procedure, Jeddah (Littlejohn *et al*, 1983)

Test loadings of eight piles (Fig. 28) indicated that settlements were both excessive (partly due to the construction process) and irregular. A very simple cementitious grouting operation was conducted 1m under the piles, resulting (Fig. 29) in highly acceptable and regular results. Grouting proved highly cost effective: in addition to permitting the safe minimisation of pile dimensions, it allowed excavation to terminate above the ground water level and so markedly simplified construction techniques ("concreting of unbraced boreholes", excavated by grab).

On the other hand, Littlejohn *et al* (1983) described the successful application of complex grouting techniques for 960-1 200mm piles, of working loads up to 580tonnes, at the Corniche Centre, Jeddah, KSA.

All piles were designed as end-bearing in a stratum consisting of dense coarse sand ($\phi = 35°$), interbedded with hard sandy silt ($c_u = 60kN/m^2$, $C' = 0$; $\phi = 30°$), encountered at a depth of 11m. Bearing in mind the high working loads, and the specified limiting settlement of 80mm at 150% working load, these piles seemed unusually shallow for the local conditions. The method of pile construction included the use of a Benoto grab to minimise soil disturbance, and involved a final stage of limited (cement grout) injection beneath the toe.

Preliminary tests of Piles 190 and 214 (Fig. 30) indicated excessive settlement under load due primarily to ground disturbance beneath the base: penetrometer tests confirmed the presence there of some 300mm of soft, silty sand sediment. Tests also indicated failure of limited ground treatment beneath the toe using a neat cement grout.

As a result of these preliminary findings, it was decided to treat the low permeability silty sand, but with a low viscosity (1.5cp) resorcinol formaldehyde grout, to form an enlarged base of strengthened soil. To obtain the required degree of control over the location and quantity of chemical injected, a tube-à-manchette system extending 2m beneath the base of each pile was proposed.

Laboratory tests of grouted sand showed that the grout developed 75% of its strength within 48 hours and full strength 5-7 days after injection ($0.4\text{-}1.3N/mm^2$).

Grout trials were carried out, both to investigate the effect of the resorcinol formaldehyde injection on load settlement behaviour of the pile, and also to optimise injection pressures, flow rates and grout

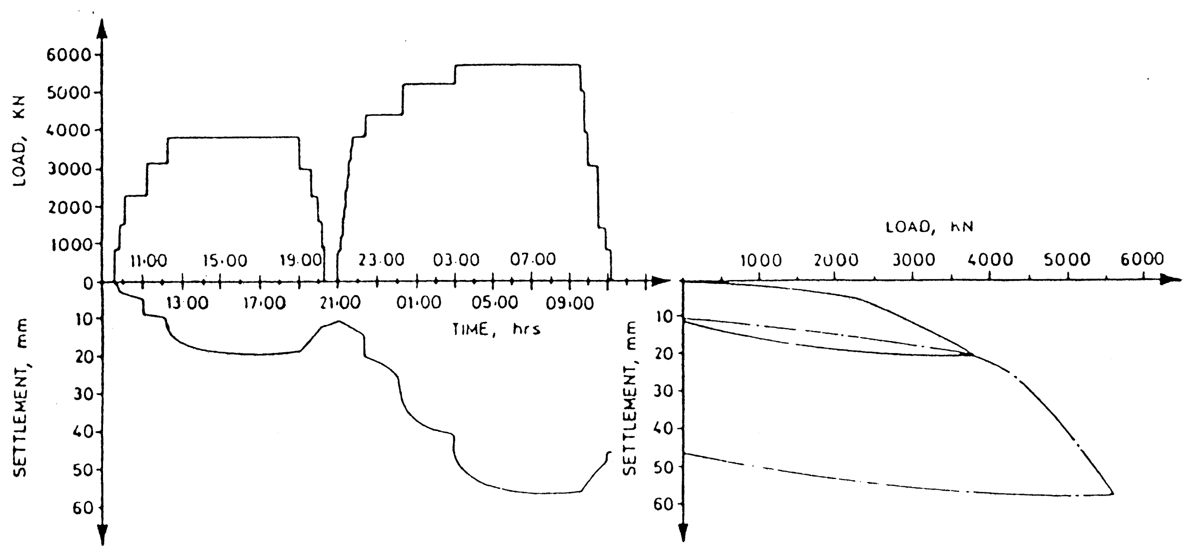


Fig. 32. Typical pile test results, Jeddah (Littlejohn *et al*, 1983)

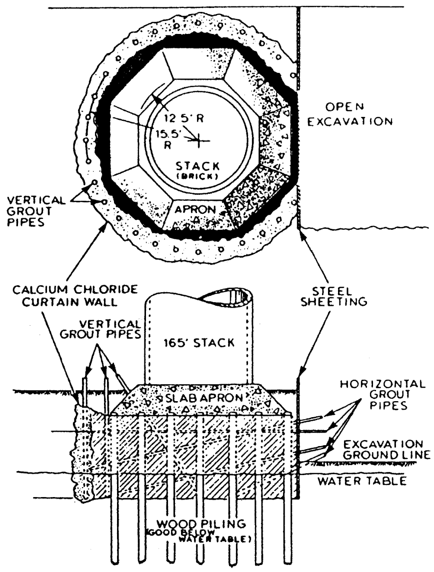


Fig. 33. Grouting of sand to maintain pile performance, Minneapolis (Karol, 1983)

quantities for the production grouting.

Trial injections followed, featuring a single tube-à-manchette installed from ground surface in order to monitor water absorptions and judge the overall groutability of the soil prior to pile grouting. Water flow rates through the sleeves indicated in-situ permeabilities ranging from 1 x 10^{-5} to 5 x 10^{-7}m/sec. Initial grout injection rates through sleeves generally ranged from 2.5 to 30 litres/min at an injection pressure of 5 bars and grout consumptions in excess of 1 500litres/m depth confirmed that adequate grout treatment by permeation could be achieved from a single grout tube installation. Due to the finely laminated nature of the sand and silt beds, a sleeve spacing of 0.33m was adopted.

The next stage was to exploit the system under a production pile (No. 166) incorporating a full length central 100mm dia. duct through which the drill string could be homed. After drilling 2m into the underlying soil a single tube was then installed through the duct and encased in bentonite/cement sleeve grout (Fig. 31). When the sleeve grout (w/c = 2, plus 5% bentonite by weight of water) had been allowed to cure for three days, water tests were carried out followed by sequential primary chemical injection below the pile. Sleeves 5 and 6 were incorporated simply as a contingency in the event of a nil take in sleeve 4. Secondary grouting was attempted through each sleeve after 24 hours to check and ensure the tightness of the ground.

For ground porosities of 25-33%, the quantities of chemical injected indicated a 1.8-2.1m spread of grout for a depth of at least 1.0m below the toe of the pile.

After one month, Pile 166 was test-loaded (Fig. 32) to compare the load settlement behaviour with those of the ungrouted piles and thereby judge the effectiveness of the grouting. The recorded improvement in base resistance was accepted by the engineer for the production piles. Approximately 339m^3 of chemical grout was used for the production pile grouting. Test loadings, made at random, proved the successful performance thereafter.

(c) Ground treatment during service

Although the three examples presented below are of relatively small scale, and apply to wooden piles in the United States, similar works have been executed in many parts of the world (Bruce & Shirlaw, 1985) where new tunnelling works are underway in close proximity to existing structures, supported on piled foundations. Such treatments are particularly significant in certain locations, e.g. Cairo, where ground conditions are potentially difficult and little is accurately known of the status of the existing piles.

Firstly, in Minneapolis, Minnesota, existing wood piling under a 50m high brick chimney had deteriorated above the water table, and so the support to the foundation slab was reducing. The sand (St. Peter formation) had been loosened due to the pile decay and was not capable of carrying the load without excessive settlements. It was therefore necessary to strengthen the foundation soil to the extent that load could be transferred to the sound portion of the piles, beneath the water table.

Fig. 33 shows the grouting principle used to solidify and strengthen the soil. The nature of the soil precluded the use of a cement based grout and so a high concentration silicate based grout (target UCS: 0.5N/mm^2) was used for the underpinning. The contractor first opted for a more economic calcium chloride grout curtain to constrain the lateral travel of the more expensive silicate grout. About 15m^3 of soil was treated and tests gave in-situ values comfortably in excess of the design expectation.

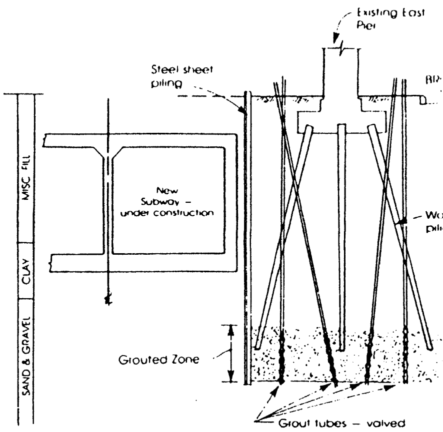


Fig. 34. Grouting of sand and gravel to safeguard piles subject to external effects, Philadelphia (Karol, 1983)

Secondly, in designing the Philadelphia Broad Street subway extension, the reinforced concrete box of the subway was to pass very close to the three rows of piles supporting the east pier of the Walt Whitman Bridge approach (Fig. 34). Although excavation for the subway would stop well above the load-bearing zone of the piles, it was feared that vibration due to subway operation might densify the granular soils, and so cause settlement of the pier. To prevent this, it was decided to grout the

DEPTH FEET 0	GENERAL DESCRIPTION	STANDARD PENETRATION BLOW COUNT – LOCATION #1 – BEFORE GROUTING	LOCATION #1 – AFTER GROUTING	LOCATION #2 – BEFORE GROUTING	LOCATION #2 – AFTER GROUTING
5	MISCELLANEOUS FILL	23/1"	16	17	
		3	4	13	
10		15	16	2	
15		3	9	9	
20	Firm Silty Clay with decayed vegetation	10	7	14	21
25	Dense medium to fine sand with trace fine gravel and occasional silt layers	30	33	27	58 59 37 139 55 100/3" (GROUTED ZONE)
		36			
			100/2" (GROUTED ZONE)		
30				30	
		38	67	40	
35	Dense gravelly sand	40	97		
		64	79	28	
			69		
40				48	
		46		38	
45		52 Fig 2		20	

Fig. 35. Effect of grouting on SPT values Philadelphia (Karol, 1983)

granular soils in this zone. A relatively high strength, low viscosity chemical grout was used, employing 115 holes of the Stabilator valve tube system (Bruce, 1984) on a 1.5m grid. The grout consisted of a proprietary polyphenolic polymer base, reacted with formaldehyde and accelerated by sodium dichromate. Grouting extended from 1.2m below the deepest pile to 0.9m above the shallowest. Grouting of the outermost holes first, to predetermined volumes gave a grout curtain to contain the rest of the grout. Tests of the ground after grouting gave the very acceptable results of Fig. 35, indicating high and increased SPT values relative to the virgin soil. In addition, running sand was eliminated, and a marked increase in the "cohesion" of the ground with a decrease in its permeability were also noted.

In the third example, two tubes of the Washington metro were to pass under the bridge piers of the 7th Street Underpass of I-95. In order to prevent loss of support to the piles, grouting around their bases was executed. The zone treated extended 6m beyond the extreme piers and over the total width of the Interstate plus 4m. Holes were drilled to 5m depth and grouting executed through 38mm dia. slotted polyethylene pipes on a 2m grid. The grouted section was approximately 1m each side of the tunnel crest line i.e. the material 1m above the top of the tunnel and 1m into the tunnel. Grouting was executed with a sodium silicate base with ethyl acetate and formamide reagents modified with peroxide oxidisers. After grouting, a 930mm dia. hole was excavated through the treated zone to permit visual inspection of the consolidated material and an entirely satisfactory result was concluded.

Summary and conclusions

This review of well documented case histories illustrates clearly the benefits of grouting to improve the performance of large diameter piles. Grouting is used both as a routine construction method, and as a recognised, reliable remedial measure.

In normal construction, grouting permits increased working loads to be utilised, or alternatively allows pile dimensions to be reduced whilst providing the same load as non-grouted piles. This is particularly advantageous in the case of deep, high capacity piles through difficult ground conditions in remote locations. Most often, grouting is executed through devices placed within the pile.

As a remedial measure, grouting ranges from the simple underbase grouting of pile/ground contact zones, to sophisticated ground treatment programmes conducted to improve the bearing properties of the ground itself. Such programmes have been conducted at the time of construction when test loadings have highlighted unsatisfactory performance, and at later stages, when adjacent new constructions have threatened the service behaviour of existing piles.

Continuing developments in grouting technologies on the one hand, and the increasing demands placed upon piling specialists on the other suggest strongly that the role of grouting in enhancing pile performance will continue to grow in the years to come.

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