A review of minipiling, with particular regard to Hong Kong applications

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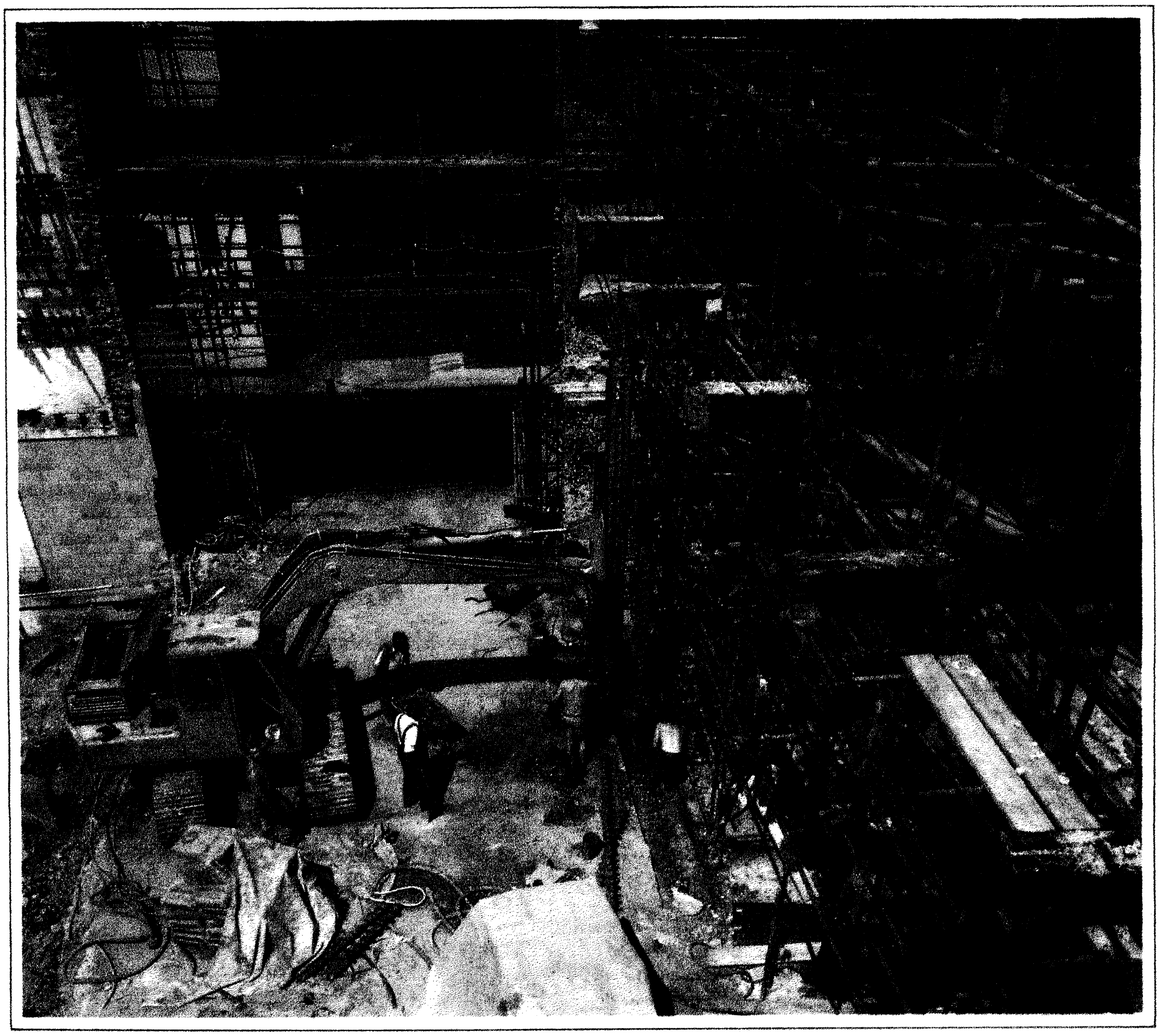

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Definition

Minipiling in its many variants has been conducted for over 30 years following its inception in Italy. The flexibility of the technique throughout the world is reflected in the range of particular descriptions: minipiles, micropiles, root piles, pali radice, needle piles, pieu racine, Wurzelpfahle, and Estaca Raiz. All, however, refer to a 'special type of small diameter bored pile' (Koreck, 1978).

Distinction is often drawn on pile diameter. Some regard 'minipiles' as being in the range 150 to 250 mm, with 'micropiles' smaller. Many consider that a diameter of 100 mm defines the lower limit, recalling that 'conventional' bored piles commence at over 300 mm. However, since the actual in-situ diameter is invariably larger than the nominal size of the drilling system, subdivisions based on hole size seem irrelevant.

A major definitive aspect is their mode of construction. Standard drilling and grouting equipment and techniques, as exercised in ground treatment or anchoring, are used. In the context of this paper, this proviso excludes displacement piles of the type driven by hammers, jacks or vibrators, although such piles can provide attractive solutions under certain conditions.

The subject of the review therefore, is a bored pile, from 100 to 300 mm in diameter, and installed by conventional drilling and grouting methods.

Major characteristics

Minipiles can be constructed to considerable depths (over 30 m) through all types of soil, rock and obstructions, and in virtually any direction. They have a high slenderness ratio, and so transfer load almost wholly by shaft friction, eliminating any consideration of underreaming. Most feature substantial steel reinforcing elements and so can sustain axial loading in both senses. This reinforcement can be designed to resist bending stresses safely and at small displacement.

Bearing in mind the early, sensitive applications, outlined below, the general construction dictum is *primum, non nocere* (firstly, do not harm – referring to certain medicines for the sick). The construction is therefore characterised by equipment ensuring minimum vibration, ground disturbance and noise, and capable of operating in awkward and restricted access conditions. Thus although their nature always ensures that they are lineally more expensive than conventional piles, (e.g. driven sheet – or H-piles) they may be the only guaranteed solution in a particular set of ground, site and performance conditions.

Regarding the aspect of service behaviour, minipiles exhibit relatively high carrying capacity (given their diameter) and very small settlements. As illustration, Koreck (1978) notes that minipile capacities of up to 1000 kN are common, whereas the allowable load on the 400 mm minimum diameter conventional bored pile allowed in the relevant German DIN is 300 to 370 kN. Similarly, Fenoux (1976) quotes a test on a 150 mm diameter minipile which reached 1700 kN without apparent distress. A survey of the major European specialists confirms that ranges of up to 600 kN working load are standard. Their outstanding performance is attributable largely to the method of construction, and especially the grouting techniques which often feature repeated phases of high pressure injections.

Applications

In 1952, at a time when contemporary construction codes were stipulating minimum diameters of 400 mm for cast-in-place piles, Fondedile began the commercial exploitation of root piles (pali radice) in the restoration and strengthening of historic buildings (Figure 1). The immediate reaction from traditional piling engineers was predictable, but within a short time they were convinced by the technical and economic attractions, underlined by the evergrowing pool of successful field tests and case histories.

From 1965, similar systems became popular in Germany, mainly in the underground construction of roads, subways and metros, as in Munich, Hannover, and Berlin. In the early 1970s the original Italian patents began to expire and so the growth of new proprietory systems was further stimulated as progressively more ingenious schemes had to be devised to satisfy foundation problems in urban environments (Figure 2). Hence the Gewi pile (Dywidag), the Tubfix Micropile, (Rodio), Pieux I.M. (Soletanche), the Rohrphfahl (Stump), and the Menard minipile made their appearances, and proved excellent construction techniques. The next logical step was then to develop preloading facilities in certain types of minipile, especially those used for underpinning old, sensitive structures. Preloading eliminates any structural settlement due to the action of the structure, distributes the pile reactions according to a pre-established pattern, and can reset the original configuration of a structure distorted by differential settlements.

Fondedile meanwhile had further exploited minipile potential with the Reticulated Root Pile concept (RRP). Designed to resist lateral displacement of a large soil mass, the first application is illustrated in Figure 3 (Lizzi, 1982). The working principle is akin to inserting reinforcing bars into mass concrete: a large earth mass is engaged by installing a diverging root pile network at close spacing and in a

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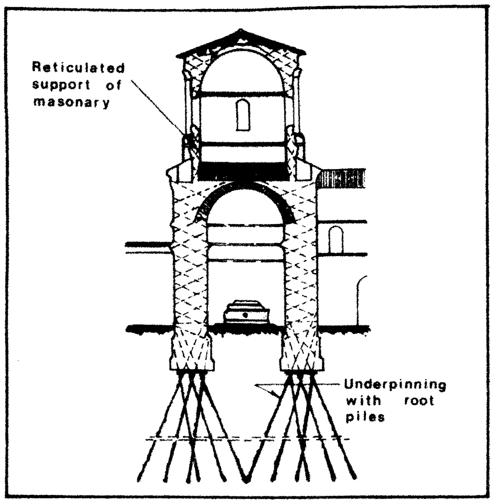


Fig. 1a S. Andrea delle Fratte Church in Rome. Underpinning and masonry reinforcement.

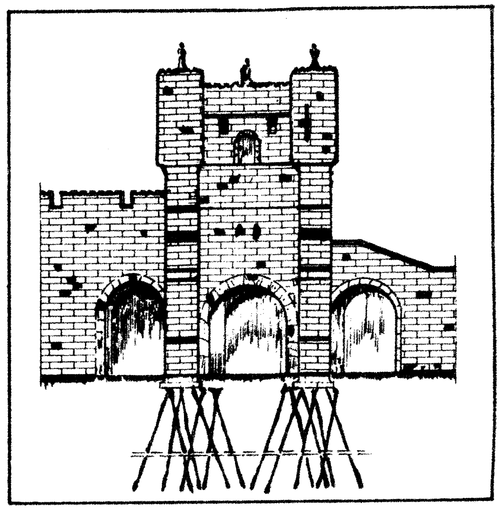


Fig. 1b Bootham Bar in York. Underpinning (after Lizzi, 1982)

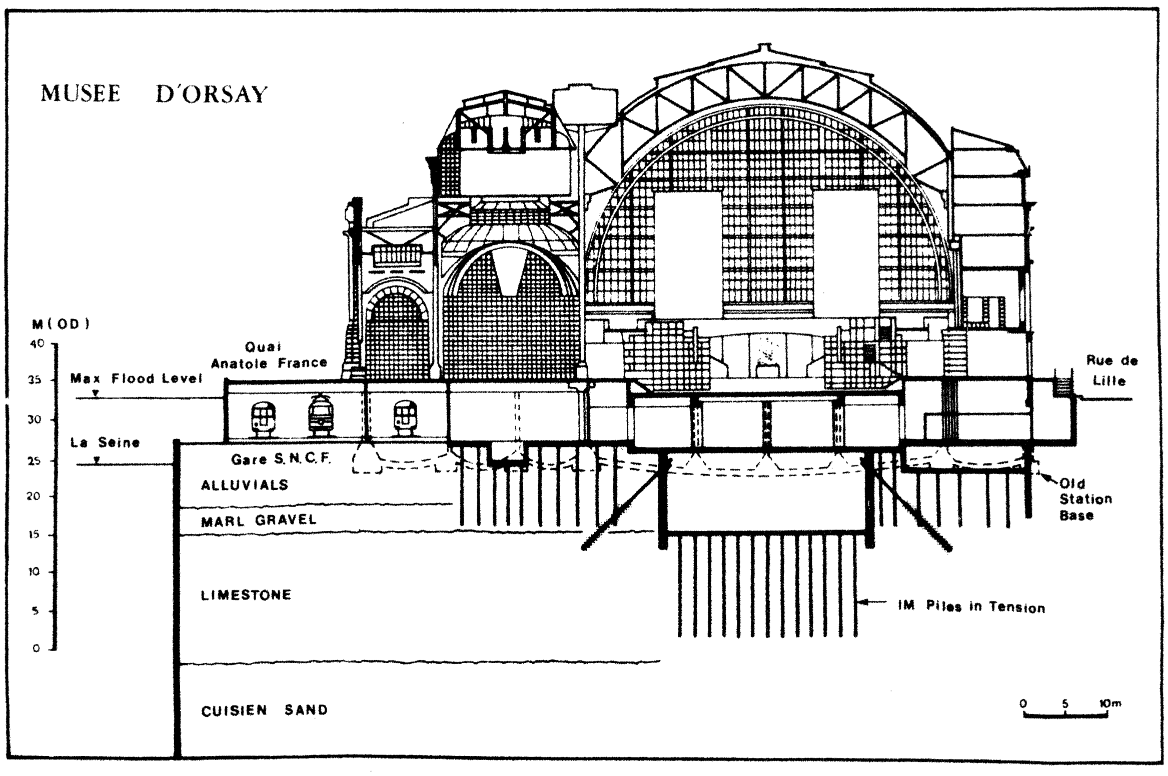


Fig. 2a Musee D'Orsay, Paris. Arrangement of minipiles (Soletanche Technical Brochure)

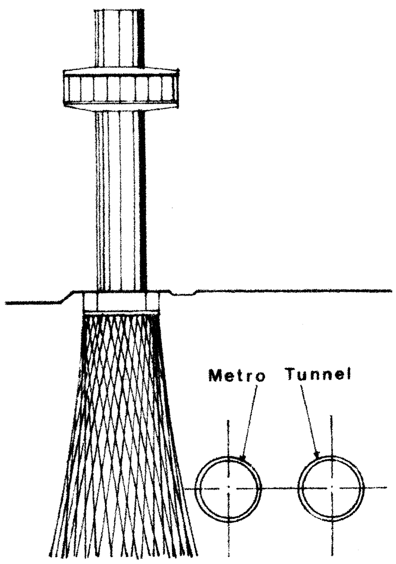


Fig. 2b Panorama Tower, Tokyo, Arrangement of RRP type minipiles (Lizzi, 1982)

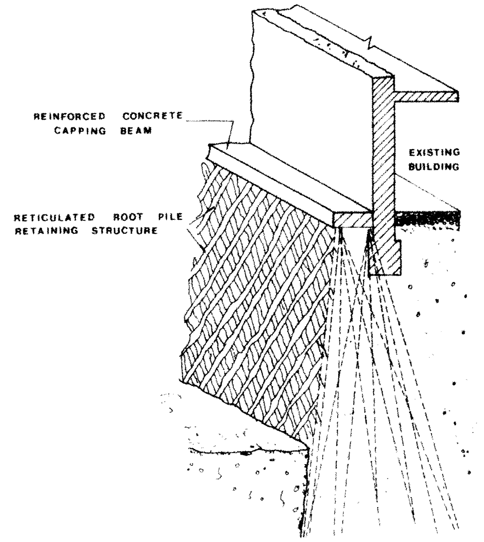


Fig. 3a Principle of RRP system (FHA Report, 1976)

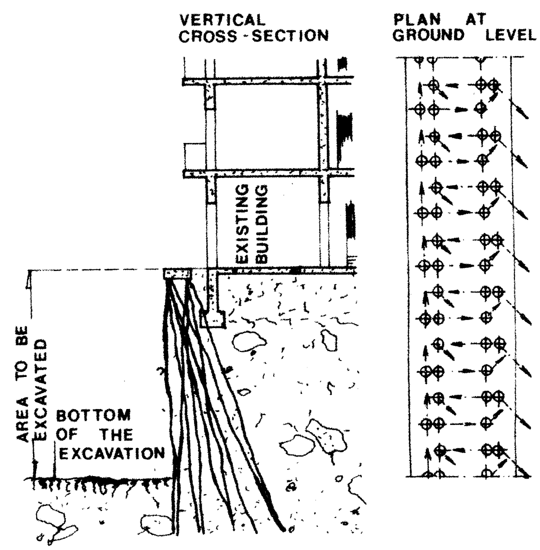


Fig. 3b Typical scheme of RRP (Lizzi, 1982)

particular geometric pattern. A lattice structure is thus constructed to encompass the soil which may then be regarded as behaving monolithically. Design procedures involve analyses similar to those used for gravity walls, namely, evaluating the overturning moment, determining the position of the vertical reaction on the base, and checking for horizontal shear through and below the monolith. The degree of the ground reinforcement is based on the 'density' of the piles, as opposed to the capacity of the individual elements. As shown in Figure 4, the array can also provide direct underpinning.

It is emphasised that the system does not rely upon any intergranular improvement of the soil, by penetration of grout. If this does happen, for example in coarser soils, then it is a bonus, permitting some reduction in pile numbers: the system functions primarily through pile-soil interaction, in which the ground is reinforced in a general way, regardless of the characteristics of individual horizons.

Particularly interesting minipile case histories are described by FHA, (1976), Lizzi (1982), Weltman (1981), Koreck (1978), Jones and Turner (1980), and Mascardi (1982), and these applications can be broadly classified as follows:

Pure bearing capacity

- As piles in the normal way for light industrial or domestic dwellings in (a) weak soils, (b) swelling soils, (c) shrinkable soils, and to reduce differential settlement between old and new construction,
- For underpinning,
- For support of isolated machine bases by design or later necessity,
- For support of floor slabs in industrial buildings,
- To reduce or arrest total settlements,

Bearing capacity and soil reinforcement

- Light foundations on sloping sites,
- Beneath retaining walls,

Soil reinforcement

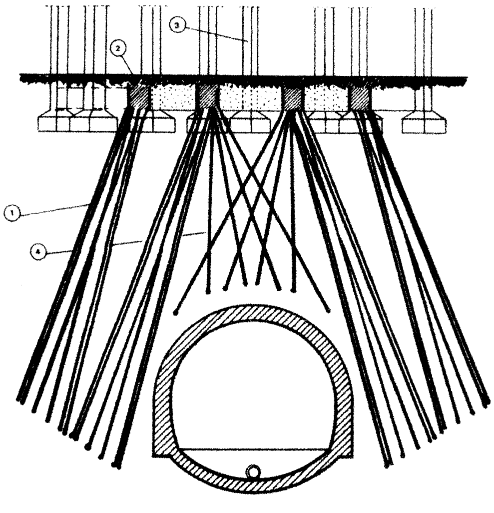
- To form a reinforced soil 'wall',
- Stabilisation of slopes,

In tension

- Below buoyant structures,
- Below towers and masts.

It is proposed that locally, the major applications would be

(i) as load bearing piles,
(ii) in retaining wall construction, and
(iii) in slope stability/landslide prevention



Legend: **1 RRP type minipiles as underpinning**
2 Network of reinforced concrete beams capping the RRP minipiles, and encasing the footings of the building
3 Existing footings
4 RRP type minipiles for further ground reinforcement

Fig. 4 RRP type minipiles applications (after FHA, 1976)

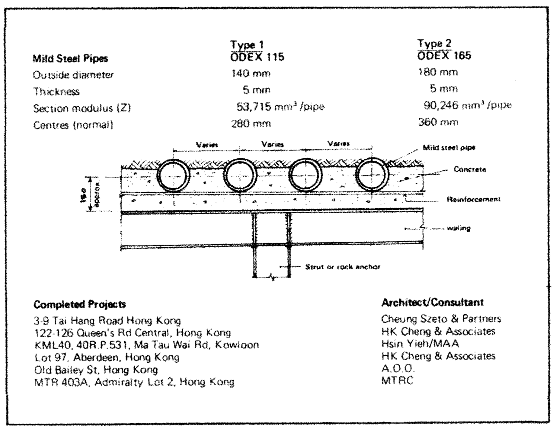


Fig. 5 ODEX pipe pile retaining wall system (Franki PSC Ltd. Technical Publication)

Application (i) is clear. With respect to application (ii), there are occasions when sheet piles may not be practical, diaphragm walls may not be economic or feasible, and consolidation grouting alone may not be completely suitable. Then retaining walls can be constructed either by the RRP system described above, or by closely spaced vertical minipiles, know locally as 'pipe piles' (Figure 5). Walls so formed can then be reinforced by concrete or shotcrete, rendered watertight with grouting or supported by anchors or propping.

Application (iii) has clear potential in Hong Kong, and would provide obvious opportunities for the RRP system, particularly as no detrimental change to perhaps delicate slope equilbria would be made. For slopes in loose, clayey materials, with boulders, the only prerequisite would be a firm stratum at a reasonable depth into which the piles could be toed (Figure 6a). Slopes in very shattered or weathered material (Figures 6b and 6c) require only that the piles extend sufficiently (say a few metres) beyond the slip plane. This is in contrast to, say, prestressed anchor solutions wherein the stress conditions of the existing slope are altered, and often considerable depths must be drilled to ensure good fixed anchor rock conditions.

Design

General

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

However, whereas the design of a conventional system is normally controlled by the external (i.e. ground related) carrying capacity, their small cross sectional area dictates that minipile design is most often limited by the internal carrying capacity. In addition, the external carrying capacity is dictated by skin friction, as opposed to end bearing: a pile of 200 mm diameter, 5 m long, has a peripheral shaft area 100 times greater than the cross sectional area. Further-

Table I
Details of Dywidag bars

Steel grade yield/ultimate	Number of bars	Nominal diameter	Ultimate load	Working load				Yield load	Working load		
N/mm²		mm	$F_z = B_z \cdot A$ kN	Ult. load 1.6 kN	Ult. load 1.7 kN	Ult. load 1.8 kN	Ult. load 2.0 kN	$F_s = B_s \cdot A$ kN	0.75 Yield load (Yield load/ 1.33) kN	0.6 Yield load (Yield load/ 1.67) kN	Yield load/ 1.75 kN
835/1030	1	26,5	568	355	334	316	284	460	345	276	263
835/1030	1	32,0	828	518	487	460	414	671	503	403	384
835/1030	1	36,0	1048	655	617	583	524	850	637	510	486
1080/1230	1	26,5	678	424	399	377	339	595	446	357	340
1080/1230	1	32,0	989	618	582	549	495	868	651	521	496
1080/1230	1	36,0	1252	783	736	696	626	1099	824	659	628

Designation according to S1-standard: F = force, A = cross sectional area of steel

Table II
Details of Gewi bars

Load characteristics of the GEWI-Steel			
GEWI-threadbar	dia. 50 mm	51mm	57mm
	kN	kN	kN
Ultimate load	98o	1133	1443
Yield load	824	795	1o13
Workingload at yield load / 1,75	47o	454	579
Area (sq. cm)	19.63	2o.27	25.81

*manufactured in USA

more, settlements of ten to 20 per cent are necessary to mobilise full and bearing capacity, compared to 0.5 to 1.0 per cent for friction piles to mobilise maximum shaft resistance.

Internal load carrying capacity

STEEL

As discussed below, calculations based on composite action (i.e. load distributed between steel and grout) may not be acceptable under local Building Regulations. The nature of the pile is then limited by the characteristics of the steel member, which may be a bar (or group of bars), or a pipe.

In both cases, the element is fully embedded in high-strength microconcrete or grout. Bearing in mind that steel pipes of circular section have a high radius of gyration and a constant section of modulus in all directions, they have intrinsically good column properties. For cases of pure axial loading in well-restrained conditions, bars are often preferable on logistic and economic grounds.

Steel sections, plain or fabricated should comply with the requirements of BS 4360, whilst pipes should also comply to BS 3601 or 1775 (although compliance with requirements of hydraulic pressure testing is unnecessary).

Typical of the kind of high strength bar used, and of the results possible, is the Dywidag bar (Table I). It will be noted that bars of diameter 26.5 to 36.0 mm will give working load capacities of 263 to 783 kN, depending on steel grade, factor of safety, and method of calculation (i.e. on yield, or ultimate, load).

The load characteristics of the larger Gewi type thread bar are shown in Table II. Loads available with the Soletanche Pieux 1M system, which is very flexible in the type and number of reinforcing elements, are shown in Table III.

For comparison, the maximum permissible reinforcement stresses in reinforced concrete allowed under local Building Regulations is shown in Table IV. In contrast, characteristic strengths quoted in CP 110 are shown in Table V.

A lower value on the characteristic strength may be taken for design if necessary to reduce deflection or control cracking.

For bored piles and for other pile types where no driving stresses are inflicted, CP 2004 (1972) allows working stresses of up to 50 per cent yield stress. Assuming that

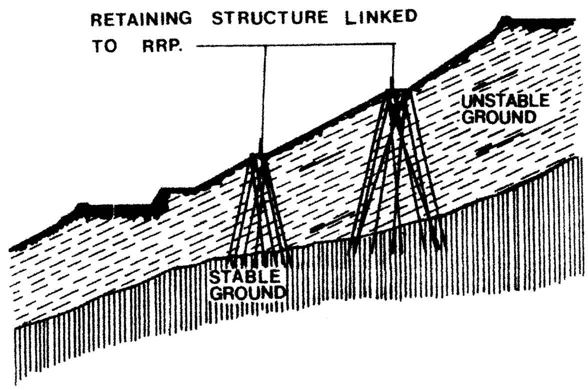


Fig. 6a E.U.R. Rome. RRP retaining wall in loose soil

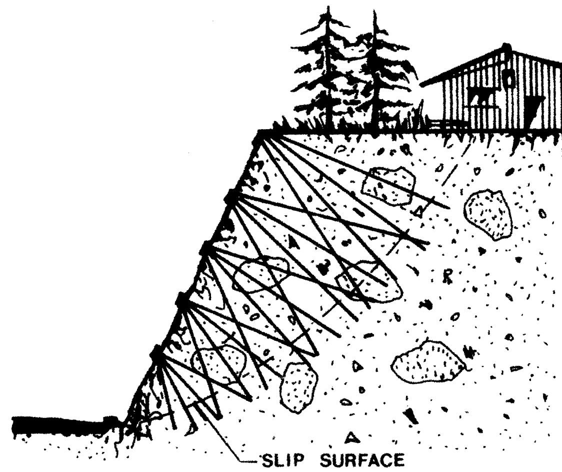


Fig. 6b Southern Highway, Italy. RRP reinforcement in weathered formations

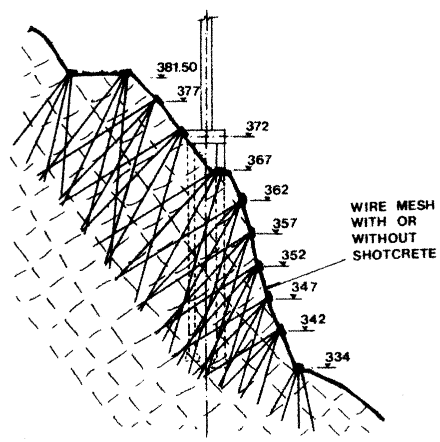


Fig. 6c Rodovia dos Immigrants, Sao Paulo. RRP reinforcement in fissured rock (after Lizzi, 1982)

the yield stress is approximately 85 per cent of the ultimate stress, this means that if piles are to be tested to 2WL, and all the load is considered to be borne by the steel, then the maximum permissible steel stress, at WL is 42.5 per cent of ultimate.

These stresses relate to piles confined in the ground. When they project above ground level, the allowable

Table III
Main types of Pieux 1M (after Soletanche technical brochure)

	Type of reinforcement		Characteristics		Nominal Capacity (to DTu 13.2, Foundations Profondes June 78)	
Type	Dimensions (mm)	Elastic Limit (N/mm^2) σ_{ea}.	Minimum hole diameter (mm)	Steel Section (mm^2) S_a	2/3 σ_{ea} S_a (kN)	1/2 σ_{ea} S_a (kN)
Profile	IPE 100 X 55 X 4	240	150	1000	160	120
Tubes	46/60	390 530	100	1200	310 420	230 310
	70/89	390 530	120	2300	600 820	450 620
	97/114	390 530	150	2800	730 1000	550 750
	109/127	390 640	170	3400	880 1200	660 900
	157/178	390 530	200	500	1300 1760	980 1320
Bars and Cages	20T 32T 40T 26DY 33DY 36DY	400 400 400 800 800 800	60 to 250 depending on the reinforcement	300 800 1300 500 800 1000	80 210 340 280 450 530	60 160 260 210 330 400
	6 Nr. 32T 4 Nr. 36T	400 800	150 150	4800 4000	1290 2120	970 1600

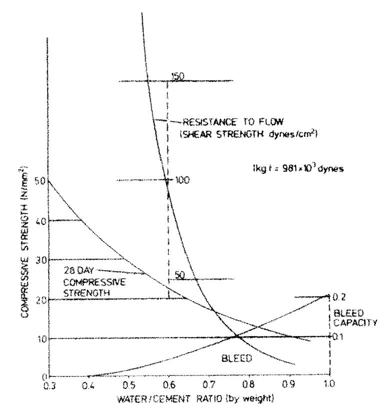


Fig. 7 Effect of water content on cement grout properties (Littlejohn, 1982)

stresses are those indicated, for different slenderness ratios, in Table VI.

A grout cover of at least 20 mm is typical for permanent protection in normal soils. For tension piles in very aggressive conditions, a further protection in the form of corrugated PVC, for example, is provided.

GROUT

Typical material are as follows:

- Ordinary Portland cement and RHC (to BS 12)
- Sulphate resisting cement (to BS 4027)
- Sand (to BS 882)
- Water (to BS 3148, or suitable for drinking)

Cement grouts should be sufficiently fluid to allow efficient pumping and injection, and sufficiently stable to resist displacement and erosion after injection.

The principal variable affecting the properties of cement grouts is the water/cement ratio (w). The amount of water determines rate of bleeding, subsequent plasticity and ultimate strength of the grout. The extent to which these, and also fluidity, are related to w is shown in Figure 7 (neat Type 1 cement). Excess water causes bleeding, low strength, increased shrinkage and poor durability. Typically, w values of 0.45 to 0.55 are used.

Fine sands can be added to neat cement/water suspensions to form an economical grout particularly where a high solids, low water grout with relatively high frictional shear strength is required. Sand is chosen as for concrete in relation to durability, shrinkage, and alkali reaction, and in general, hard bulky crushed rock is preferred to flat, angular or flaky material which gives poor fluid handling properties. Evenly graded sands are preferred (5 mm down to 75 μm) and for long pumping distances the maximum size should ideally be reduced to 0.5 mm and the maximum sand/cement ratio limited to 3 to maintain the particles in suspension and avoid segregation. Rarely however does the ratio exceed 1.5, and 1.0 is most common, giving characteristic strengths of the order of 30-35 N/mm^2. In confirmation, Koreck (1978) advocates a minimum cement content of 600 kg/m^3 (= sand/cement ratio of 2.3 at w ratio of 0.50), and Lizzi (1982) advocates 600-800 kg of cement per cubic metre of sieved sand: "Therefore a high strength concrete".

Table IV
Maximum permissible stresses in reinforcement in reinforced concrete (Hong Kong Building Regulations)

Kind of stress	Maximum permissible stresses in reinforcement in MPa		
	Mild steel complying with BS 4449: 1969		Steel with minimum yield point f_y MPa
	Not exceeding 40 mm diameter	Exceeding 40 mm diameter	
Tension other than tension in shear reinforcement	140	125	0.55 f_y or 210 whichever is the lesser
Tension in shear reinforcement	140	125	0.55 f_y or 155 whichever is the lesser
Compression in longitudinal reinforcement in axially loaded columns and in main reinforcement in beams and slabs where the compressive resistance of the concrete is not taken into account	125	110	0.55 f_y or 155 whichever is the lesser.
Compression in longitudinal reinforcement in columns other than axially loaded columns and in main reinforcement in beams and slabs where the compressive resistance of the concrete is taken into account	The calculated compressive stress in the surrounding concrete multiplied by the modular ratio but not exceeding, as appropriate.		
	125	110	0.55 f_y or 155 whichever is the lesser.

Admixtures can be added in relatively small quantities to modify grout properties (Table VII) especially those to prevent shrinkage, allow reduction in *w* (while maintaining fluidity/pumpability) to accelerate or retard setting and to prevent bleeding (thereby discouraging corrosion). Most commercial admixtures are compatible with Type I and III Portland cements, but many are incompatible with high alumina and super sulphated cements. Admixtures should not be regarded as a replacement for good grouting practice and must not be used indiscriminately.

Bearing in mind the various kinds of batching plant and mixing techniques, trial mixes must be carried out and the following is recorded and reviewed if necessary:

- water/cement ratio (by Baroid Mud Balance),
- admixture concentration,
- flow reading (through flowmeter, flow cone or viscometer),
- crushing strengths (minimum two cubes at 3, 7, 14 and 28 days) (to BS 1881),
- notes on amount of free expansion, shrinkage, bleed and final setting time.

GROUT/STEEL BOND

This parameter has significance in that it is the mechanism of load transfer from steel to ground. A great body of research has been conducted into the nature, magnitude, distribution and controls over grout/steel bond characteristics. Furthermore, CP 110 (1972) relates ultimate anchorage bond stresses (in tension) to the characteristic concrete strength as in Table VIII.

However, it is clear that in the majority of cases this parameter is not critical in determining pile capacity or geometry: rather the grout/ground bond, or the carrying capacity of the steel will be the critical limit.

Grout/steel bond values for bars used or recommended in practice (tension case) are illustrated in Table IX.

COMPOSITE ACTION

Local Regulations do not permit pile capacity to be assessed on composite action between grout and steel.

Table V
Strength of Reinforcement (CP 110: 1972)

Designation	Nominal sizes mm	Specified Characteristic strength f_y N/mm²
Hot rolled mild steel (BS 4449)	All sizes	250
Hot rolled high yield (BS 4449)	All sizes	410
Cold worked high yield (BS 4461)	Up to and including 16	460
	Over 16	425
Hard drawn steel wire (BS 4482)	Up to and including 12	485

Table VI
Allowable stress in axial compression for piles projecting above soil level (Cornfield, 1974)

Slenderness ratio l/r	Grade 43 (mild steel)		Grade 50 (high yield stress steel)	
	N/mm²	ton/in²	N/mm²	ton/in²
90	–	–	–	–
95	–	–	100	6.50
100	–	–	92	5.97
105	74	4.78	85	5.50
110	69	4.45	78	5.07
120	60	3.86	67	4.34
130	52	3.37	58	3.75
140	46	2.96	50	3.26
150	40	2.61	44	2.86
160	36	2.32	39	2.53
170	32	2.07	35	2.25
180	29	1.86	31	2.01

Notes: l = effective length; r = appropriate radius of gyration
Where no stress is given, maximum working stress of 0.3 x minimum yield stress controls.
It is emphasised that the stresses given are for axial loading only

Table VII
Common cement admixtures (Littlejohn, 1982)

Admixture	Chemical	Optimum Dosage (% cement wt)	Remarks
Accelerator	Calcium Chloride	1–2	Accelerates set and hardening
	Sodium silicate	0.5–3	Accelerates set
	Sodium Aluminate		
Retarder	Calcium Lignosulphonate	0.2–0.5	Also increases fluidity
	Tartaric Acid	0.1–0.5	
	Sugar	0.1–0.5	
Fluidifier	Calcium Lignosulphonate	0.2–0.3	
	Detergent	0.05	Entrains air
Air Entrainer	Vinsol Resin	0.1–0.2	Up to 10% of air entrained
Expander	Aluminium Powder	0.005–0.02	Up to 15% pre-set expansion
	Saturated Brine	30–60	Up to 1% post-set expansion
Anti-Bleed	Cellulose Ether	0.2–0.3 (for $w < 0.7$)	Equivalent to 0.5% of mixing water
	Aluminium Sulphate	Up to 20% (for $w < 5$)	Entrains air

Table VIII
Maximum permissable bond stresses (CP 110: 1972)

Type of bar	Concrete Grade			
	20	25	30	40+
	Maximum bond stress, N/mm²			
Plain	1.2	1.4	1.5	1.9
Deformed	1.7	1.9	2.2	2.6

In many other countries this is acceptable and has been demonstrably successful. For example, Dywidag (Pers. comm) state "when subjected to compressive stress, the cement grout cylinder also participates in the carrying of the load. This results in lower settlement values for the compression pile". The contribution of the grout in piles consisting of a cage of reinforcing bars is clearly substantial. However, by discounting composite action, larger and more expensive reinforcement must be used, in order to provide sufficient steel capacity to, in theory, sustain the total load.

External load bearing capacity

It is common practice in European countries to design minipiles such that all the load is transferred through side shear to the surrounding ground. Where the ground is very loose, cohesive, or of variable quality, piles are socketted into rock. Often the contribution of the pile/ground interface is ignored, and all the load reckoned as distributed solely over the pile/rock interface.

ROCK/GROUT BOND

Principles applied to rock anchor design (albeit in tension) may be used with equal facility (and greater safety) in the compression case of piles. For convenience, the bond is assumed to be uniformly distributed over the grout/rock interface, and so the working load, LW, may be calculated from

$$\text{LW} = \frac{\pi D L \,\tau_{\text{ult}}}{\text{sf}}$$

where D = diameter
L = length
τ_{ult} = ultimate skin friction
sf = safety factor

The choice of a suitable value for τ_{ult} is often related to local knowledge and the back analysis of successful applications. Equally, trusted relationships such as the following can be employed:

$$\tau_{\text{ult}} = \frac{\text{UCS}_{\text{rock}}}{10} \text{ to a maximum value of } 4.2\ \text{N/mm}^2$$

Thus, even if a very strong granite of UCS > 42 N/mm² is proved, a value of 4.2 N/mm² is selected as τ_{ult}. Applying a safety factor of 3, a value of τ_w of 1.4 N/mm² may be chosen. Even if the permissible local value of 1 N/mm² is taken, for a 220 mm dia. hole drilled 2000 mm* into such rock, the allowable working load could be

$$\text{WL} = \pi D L\, \tau_w = \frac{\pi \times 220 \times 2000 \times 1}{1000} = 1382\ \text{kN},$$

underlining again that it is internal load carrying capacity which limits individual pile rating. Conversly, a typical minipile of 550 kN working load, with similar geometry would mobilise an average τ_w value of less than 0.4 N/mm², a figure to be regarded as conservative when compared with values of rock/grout bond (Table X) used in practice in tension anchors. It will be noted that these calculations automatically assume that the grout transferring load from steel to ground can readily sustain the required shear stresses.

GROUND/GROUT BOND

Typical values may be quoted from work on ground anchors and bored cast-in-situ piles, viz:

Coarse sands and gravels: τ_{ult} = 0.3 to 0.4 N/mm²
Fine medium sands: τ_{ult} = 0.2 to 0.25 N/mm²
(Littlejohn, 1970)

Locally, Holt *et al* (1982) reported on skin friction values for large diameter bored piles of 0.15 N/mm² and 0.03 N/mm² for dense completely decomposed granite

* It is noteworthy that such a minimum embedment length is often specified in order to avoid terminating the pile in a boulder: few boulders are found over 2000 mm in diameter and consisting of very fresh bedrock material.

Table IX
Grout/bar bond values which have been employed or recommended in practice (Littlejohn and Bruce, 1977)

Bar tendon	Embedment (m)	Load (kN)	Working load (N/mm²)	Test bond (N/mm²)	Ultimate bond (N/mm²)	Remarks	Source
Plain			1.2.1.9			Design criteria: bond dependent on concrete	Britain—CP110(1972)
Deformed			1.7.2.8				
Square twist			5.25			Design criteria	Britain—Roberts (1970)
Ribbed			7.0				
Plain					1.38	Short embedment test	Canada—Brown (1970)
Plain and threaded end					2.62	Bond dependent on embedment and grout tensile stress	
Deformed bar	30d					Design criteria: "solid" rock	Canada—Ontario Hydro (1972)
Deformed bar	40d+					Design criteria: "seemy" rock	
20 No. 20mm dia plain	2.5	1760		0.56		Test anchor	Italy—Berardi (1960)
20mm dia ribbed and threaded	2.2			1.1		Test anchor	Italy—Beomonte (1961)
with end nut	3.9			0.6		Test anchor	
	2.2			1.2		Test anchor	-
	2.2			0.9		Test anchor	
26mm dia deformed bar	0.2				2.7	Test anchor	Canada—Brown (1970)
	0.4				5.0	Test anchor, Bond for deformed bar = 5x bond for plain bar	
	0.6						
25.4mm dia square	1.83	289		1.96		Test anchor	USA—Salisman & Schaefer (1968)
25.4mm diaplain	0.06	52			10.1	Test: for each pair the first test conducted at 28 days and the second at 90 days	Australia—Pender et al (1963)
	0.06	53.5			11.2		
	00.12	52			5.5		
	0.12	67			7.0		
	0.18	63			4.4		
	0.18	117			8.1		
	0.36	139			4.9		
	0.36	148			5.1		
28mm dia plain	5.9	160		0.31		Test anchor: bond known to be much higher locally	Italy—Berardi (1967)
	11	160		0.16			
28mm dia plain		400	0.76			Design criteria	Switzerland—Comte (1971)
28mm dia plain	0.91	220		2.72		Test anchor	USA—Drossel (1970)
30mm dia plain		320	3.0	7.2		Test at bar UTS	Switzerland—Muller (1966)
21.8mm dia high tensile	1.83	605	3.3			Commercial anchor	USA—Wosser et al (1970)
31.8mm dia and thread	1.2	700		5.74		Test anchor	USA—Drossel (1970)
31.8mm dia Dywidag & locknut	8.5	545	0.64			Commercial anchor	USA—Oosterbann et al (1972)
35mm dia midl steel	6.1	380	0.54			Anchor pile	Canada—Jaspar et al (1969)
35mm dia plain	6.1	505		0.75		Test anchor	Canada—Barron et al (1971)
35mm dia plain	6	700	1.06			Commercial anchor	USA—Feld et al (1974)
43mm dia mild steel	12.2	610	0.37			Anchor pile	Canada—Jaspar et al (1969)
44mm dia plain	0.35				4.7	Test anchor	Canada—Brown (1979)
	0.6						

($N \geqslant 100$), and soft sediments and fill, respectively. They confirmed that "the skin friction in completely decomposed granite was in fact appreciable". Sweeney and Ho (1982) also recorded a value (not ultimate) of 0.17 N/mm² for local completely decomposed granite.

Test data (Figure 8) confirm that the magnitude of skin friction is strongly influenced by the grouting pressure, and many minipile systems exploit this. In addition, pressure grouting increases the nominal cross section, particularly in the weaker soil layers or near ground level, where natural in-situ horizontal stresses are small. Mascardi (1982) notes that in cases of repeated pressure grouting an effective pile diameter in the range 300 to 800 mm may be expected. The typical concept of the functioning of repeated pressure grouting is illustrated in Figure 9, referring to the Tubfix type pile. In general, pressure grouting is most effective in improving pile capacity in these conditions where deformations can be imparted relatively quickly: sands and gravels, residual soils, shales, and some weaker sedimentary and low grade metamorphic formations. Jones and Turner (1980) also note very favourable response in stiff clay. No experience of good behaviour in very soft non-consolidated clay or soft peat has been recorded.

For piles grouted under gravity head, the following two relationships may be used, particularly valid as they relate to Standard Penetration Test/N values:

(i) $\tau_{ult} = 0.007N + 0.12$ N/mm² (Suzuki et al, 1972)
(ii) $\tau_{ult} = 0.01N$ N/mm² (Littlejohn, 1970)

Thus, for N values consistently 200 or above, and for a 220 mm hole, loads of around 400 kN/metre can be mobilised, with a theoretical safety factor of 3.

Stability of minipiles

Mathematical models can be called upon to investigate the stability of minipiles with respect to buckling and bursting resistances. Regarding the former, early work by Bjerrum (1957) is supported by the detailed analyses of Mascardi (1970, 1982) and Gouvenot (1975). All authors conclude that only in soils of the very poorest mechanical properties, such as loose silts, peat and non-consolidated clays, is even a possibility of failure through insufficient lateral restraint feasible.

Similarly, bursting can be equally discounted after analyses to BE 21/5/010, UK Bridge Code for Reinforced Columns and Piers, and BS 5300 for example. Where the possibility does exist, additional lateral restraint can be provided by increasing the thickness of the grout annulus, modifying the grouting design and methods, or by maintaining a sacrificial casing through the dubious horizons.

Spacing of minipiles

Overall structural designs often dictate the need for pile groups. The compromise is then between selecting a close spacing, thus minimising the size and cost of the pile cap, and between increasing the spacing so as to avoid a group effect reducing the load carrying capacity of each member. CP 2004 (1972) states that for "friction piles, the spacing centre-to-centre should be not less than the perimeter of the pile; with piles deriving their resistance mainly from end bearing the spacing centre-to-centre should be not less than twice the least width of the pile".

Recalling that minipile construction tends to minimise ground disturbance, there would seem logic in reducing the minimum separation especially when they must be socketted into rock for appreciable distances. However,

Table X
Rock/grout bond values for rock anchors (after Littlejohn and Bruce, 1977)

Rock type	Working bond (N/mm^2)	Ultimate bond (N/mm^2)	Factor of safety	Source
Igneous				
Medium hard basalt		5.73	3.4	India–Rao (1964)
Weathered granite		1.50–2.50		Japan–Suzuki et al (1972)
Bawalt	1.21–1.38	3.86	2.8–3.2	Britain–Wycliffe-Jones (1974)
Granite	1.38–1.55	4.83	3.1–3.5	Britain–Wycliffe-Jones (1974)
Serpentine	0.45–0.59	1.55	2.6–3.5	Britain–Wycliffe-Jones (1974)
Granite & basalt		1.72–3.10	1.5–2.5	USA–PCI (1974)
General Competent rock (where UCS > 20 N/mm^2)	Uniaxial compressive strength–30 (up to a maximum value of 1.4 N/mm^2)	Uniaxial compressive strength–10 (up to a maximum value of (4.2 N/mm^2)	3	Britain–Littlejohn (1972)
Weak rock	0.35–0.70			Australia–Koch (1972)
Medium rock	0.70–1.05			
Strong rock	1.05–1.40			
Wide variety of igneous and metamorphic rocks	1.05		2	Australia–Standard CA35 (1973)
Wide variety of rocks	0.98			France-Fargeot (1972)
	0.50			Switzerland–Walther (1959)
	0.70			Switzerland–Comte (1965)
		1.20–2.50		Switzerland–Comte (1971)
	0.70		2.25 (Temporary) 3 (Permanent)	Italy–Mascardi (1973)
	0.69	2.76	4	Canada–Golder Brawner (1973)
	1.4	4.2	3	USA–White (1973)
		15–20 per cent of grout crushing strength	3	Australia–Longworth (1971)
Concrete		1.38–2.76	1.5–2.5	USA–PCI (1974)

Recommendations for design

Rock type	Working bond (N/mm^2)	Test bond (N/mm^2)	Ultimate bond (N/mm^2)	s_m (test)	s_f (ultimate)	Source
Igneous						
Basalt	1.93		6.37		3.3	Bratain–Parker (1958)
Basalt	1.10	3.60				USA–Eberhardt & Veltrop (1965)
Tuff	0.80					France–Cambefort (1966)
Basalt	0.63	0.72				Britain–Cementation (1962)
Granite	1.56	1.72				Britain–Cementation (1962)
Dolerite	1.56	1.72				Britain–Cementation (1962)
Very fissured felsite	1.56	1.72				Britain–Cementation (1962)
Very hard dolerite	1.56	1.72				Britain–Cementation (1962)
Hard granite	1.56	1.72				Britain–Cementation (1962)
Bsalt & tuff	1.56	1.72				Britain–Cementation (1962)
Granodiorite	1.09					Britain–Cementation (1962)
Shattered basalt		1.01				USA–Saliman & Schaefer (1968)
Decomposed granite		1.24				USA–Saliman & Schaefer (1968)
Flow breccia		0.93				USA–Saliman & Schaefer (1968)
Mylontised prophyrite	0.32–0.57					Switzerland–Descoeudres (1969)
Fractured diorite	0.95					Switzerland–Descoeudres (1969)
Granite	0.63	0.81				Canada–Barron et al (1971)

Values recorded in practice

in the absence to date of supporting test data, the existing stipulations are still observed.

Construction

A key factor in the categorisation of minipiles is their ability to be installed by the conventional drilling and grouting equipment and techniques, comprehensively described in geotechnical literature, and familiar to most ground engineers in Hong Kong. This section highlights only points regarded as particularly relevant.

In essence, the construction of minipiles consists of the following sequence of activities, as illustrated in Figure 10.

(i) Drilling, and flushing out the hole (cased temporarily through incompetent materials).

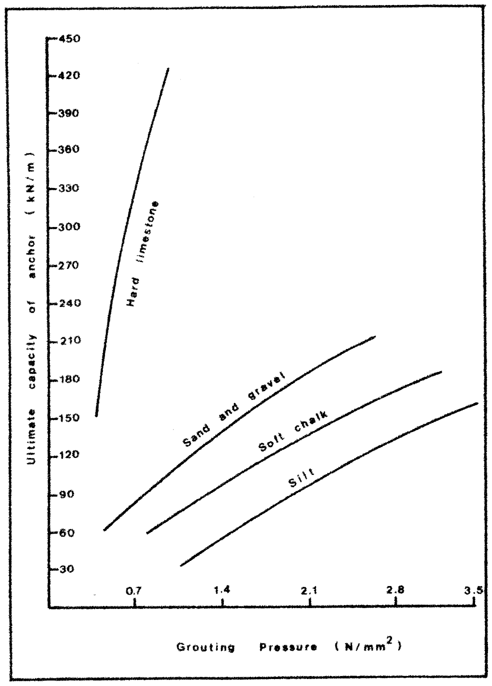


Fig. 8a Influence of grouting pressure on ultimate load holding capacity (Littlejohn and Bruce, 1977)

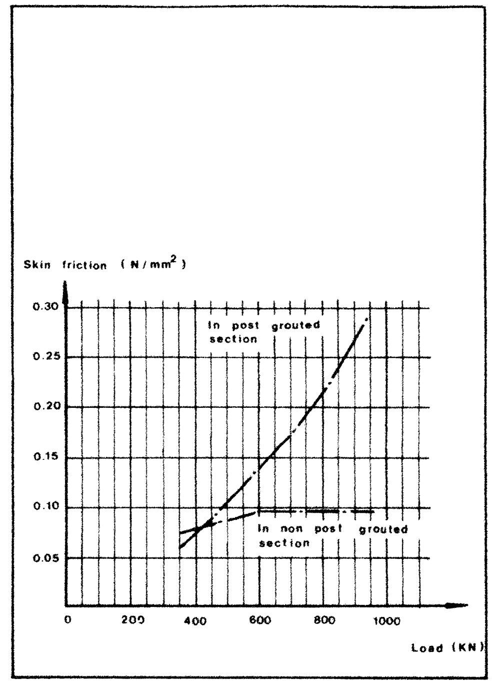


Fig. 8b Effect of post grouting on skin friction (Herbst, 1982)

(ii) Insertion of reinforcing element.
(iii) Filling temporary casing with grout.
(iv) Extraction of temporary casing, with or without excess grout pressure.

Phase (iv) can be followed by a separate pressure grouting operation, as in the cases of the Gewi and Tubfix piles for example.

Drilling

The selection of machine type clearly depends on the diameter and depth of drilling, but manoeuvrability and size are often equally important in minipiling works. In this connection, diesel hydraulic crawler-mounted machines have marked advantages. The drilling method is essentially rotary, with water flush, (occasionally with bentonite) although a duplex method (rods and casing together) is preferable, in that it is faster, more reliable and causes minimum disturbance to the surrounding ground. It also permits a (percussive) down-the-hole hammer to be temporarily introduced for punching through large boulders quickly. Very occasionally, a drilling system must be used which can provide a permanent hole liner in the upper portion of the hole, for purposes of stability and/or corrosion protection.

Insertion of reinforcement

The actual lengths may be dictated by the height restrictions, but often the main factors are the standard lengths supplied by manufacturers, and the safe handling length. Rarely are individual lengths more than 6 m used.

Lengths must be joined carefully – by screwed couplings in the case of thread bar elements, or by welding, in the case of pipe piles. In conjunction with the latter, the

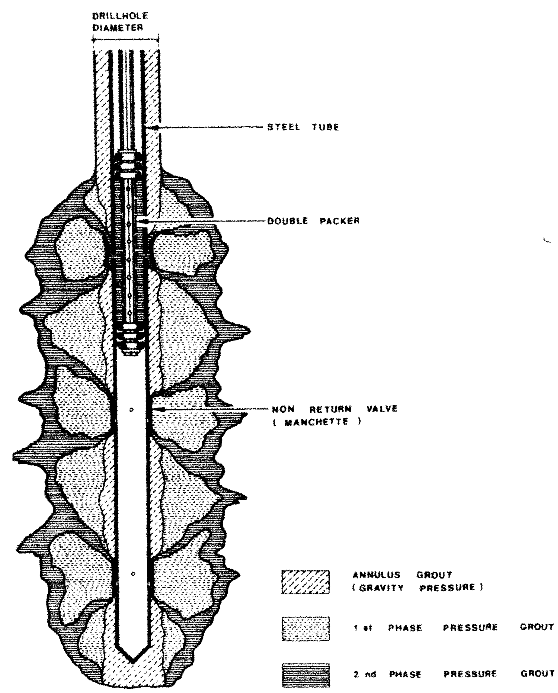


Fig. 9 Concept of repeated pressure grouting (Rodio SpA. Technical Publication)

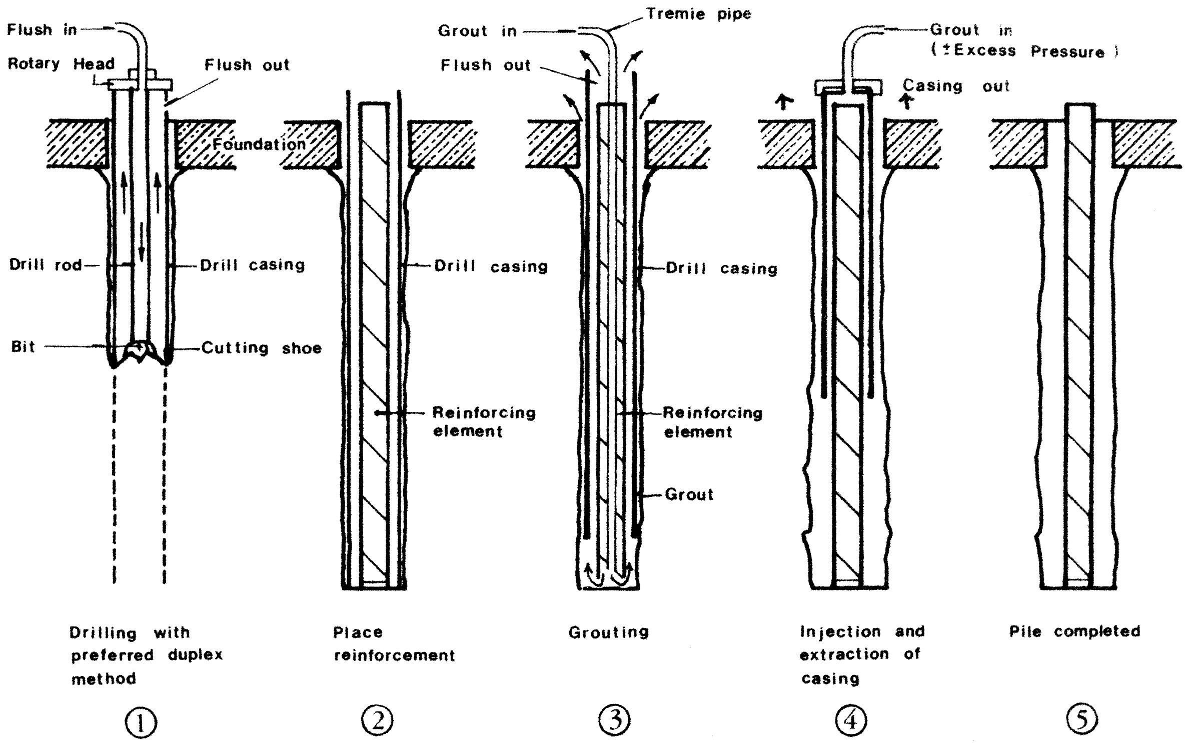


Fig. 10 Stages in the construction of a standard minipile (after Koreck, 1978)

quality of the welding materials and operations must be the highest, and appropriate standards must be observed (e.g. Manual Metallic Arc to BS 5135, electrodes to Swedish Code 150E-445.B.20).

Some proprietory systems prefer to fill the casing with grout prior to insertion of the reinforcement (e.g. Gewi pile).

The concentricity of the reinforcement in the hole is ensured by regularly spaced contralisers.

Grouting

Typically the equipment consists of a mixer, a weigh batcher and a pump, and a great variety of each exist. It is recommended, however, that a high speed mixer be specified, as this permits the production of a much more consistent high quality colloidal grout, at lower *w* ratios than a low speed, paddle type mixer.

Strict quality control of materials, and techniques is essential, particularly so since the effect of improper grouting will be accentuated at these small diameters. The simple site tests recommended earlier in this paper should be executed conscientiously.

Extraction of casing

When the borehole is filled with grout the extraction of the drilling casing begins, care being taken to prevent the grout level falling below the top of the casing. During this operation the possibility of 'necking' of the pile is prevented by the greater pressure exerted by the fresh grout against the wall of the bore hole.

During this phase, end of casing pressure grouting (Figure 11a) can be conducted to safe pressures related to ground type and the adjacency of sensitive structures.

Certain types of minipile system feature direct grout pressurisation by air to over 0.6 N/mm^2, but except under exceptional circumstances and with the tightest supervision, this technique is not to be recommended. In addition to acting on the grout, pressurisation is also found to ease casing extraction.

In the Menard minipile (Figure 11b), air is also used to inflate a central longitudinal packer, so compressing the grout column against the borehole wall. This technique is also not so widespread, reflecting practical problems.

Alternatively, pressure grouting can be executed after setting of the first phase grouting, through a tube à manchette system. Two or more phases of pressure reinjection are possible, with the final pressure maintained till initial set. For Tubfix piles, the reinforcing pipe also acts as the grouting pipe, whereas in the Gewi pile (Figure 11c) and similar piles, additional grouting pipes are placed in the hole, attached to the reinforcing bar.

Repeated pressure grouting is a costly operation, but the results are always beneficial, and often spectacularly so (Gouvenot, 1973, Ostermayer, 1975, Jones and Turner, 1980).

Testing

Testing methods and procedures for minipiles need not differ from those for conventional piles and as described in CP 2004 (1972) or the local Building Regulations, although clearly the acceptance criteria will be different. Often, however, the same practical constraints, such as very restricted headroom conditions, which dictate the selection of a minipile solution, will affect the conduct of the testing. Thus although kentledge is often the simplest and cheapest method of providing reaction during pile loading tests, it may be necessary to use ground anchors or adjacent piles in this role if space is particularly limited (Figure 12).

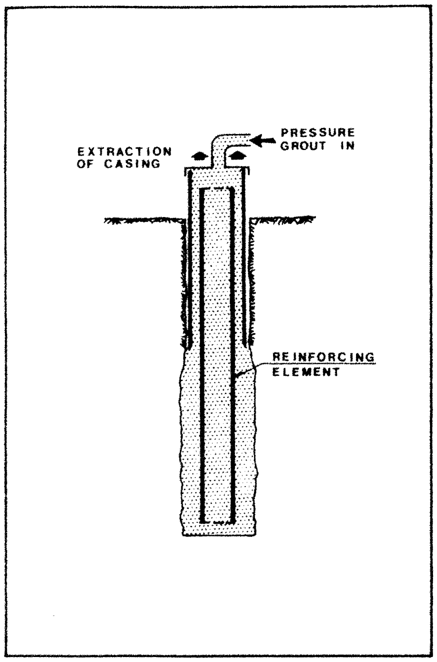


Fig. 11a Standard minipile

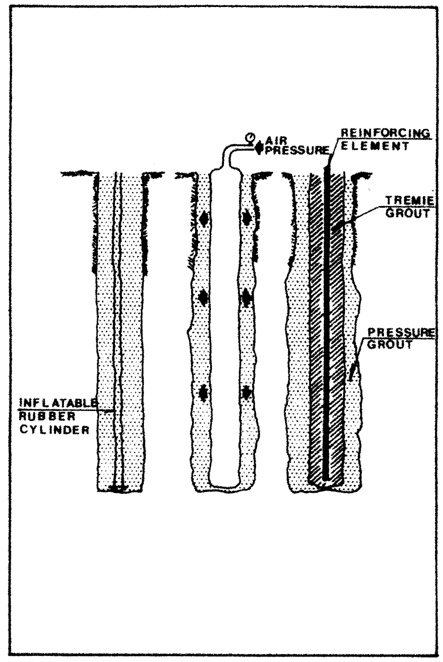


Fig. 11b Menard minipile

The key performance feature is clearly the ability to sustain loads with comparitively small settlements. As a general guide to what may be expected, Table XI summarises results obtained from various sites throughout the world.

Local case histories

The following case histories of contracts successfully executed in Hong Kong have been chosen to illustrate major points in minipile application, design, construction and performance. Of equal importance is the logic in each instance behind the selection of the minipile technique in preference to others.

Example A (Bachy Soletache Group)

Table XII represents a summary of four typical contracts executed by the Group. Of particular interest is the great variety of pile types and working loads (50 to 1230 kN), matched by the differences in the scale of the works (from 20 holes (120 m) to 654 holes (13 080 m).

Example B (Gammon Foundations Ltd.)

The 450 m^2 site at Tai Hang Road is situated on a steep slope and abuts existing buildings on one side. The site investigation also indicated a very large concentration of fresh granite boulders in the completely decomposed granite. Temporary support was required around three sides of the site, and the combination of topography and geology acted against conventional piling. A minipile wall was therefore designed, consisting of 193 piles, 140 mm in diameter and at 350 mm centres.

The piles were installed by the ODEX drilling method using welded casings. Each pipe was socketed a minimum of 1.5 m into bedrock and filled with cement grout (Figure 13). Excavation then proceeded in stages, allowing a a reinforced concrete wall to be cast against the minipiles, and inclined temporary anchors to be drilled into the underlying rock.

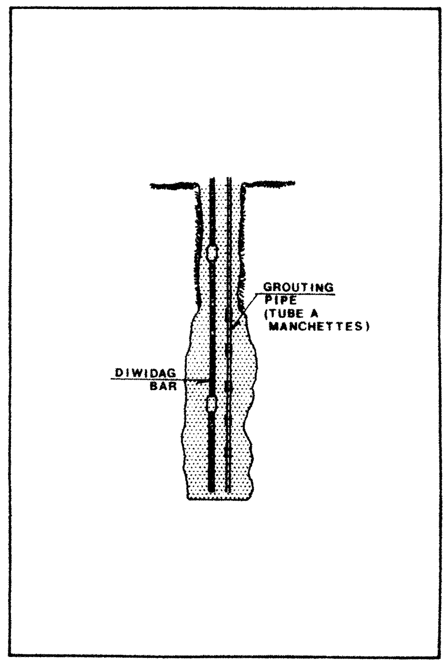


Fig. 11c Gewi minipile (after Mascardi, 1982)

Example C (Vibro (HK) Ltd)

To support a number of walkways in Shatin, piles were installed through varying thicknesses of marine deposits, to be founded in decomposed granite. Using the 50 mm Gewi bar (hole diameter 150 mm), working loads of 300

Table XI
Summary of load test data (FHA, 1976)

Soil Type	Nominal Diameter mm	Length m	Assumed Effective Length m	Max. Test Load kN	Settlement at Max. Load mm	Location
G	101.6	6.4	6.4	224.0	1.02	School Building, Milan, Italy
C	101.6	12.2	12.2	224.0	4.06	Olympic Swimming Pool, Rome
G	304.8	27.4	27.4	515.1	8.13	Bausan Pier, Naples
Si, G	101.6	14.9	6.1	201.6	2.03	Italian State Railroad, Rome
G	101.6	15.8	12.8	179.2	2.29	Bank of Naples
G	215.9	30.2	20.1	1099.4	5.59	Crops of Engineers, Naples
G	127.0	19.8	7.3	509.0	8.13	Washington, D.C., Subway
G	228.6	5.9	3.0	458.1	11.43	Queen Anne's Gate, London
G	177.8	8.5	5.5	509.0	7.62	Queen Anne's Gate, London
C-G	101.6	16.1	16.1	235.2	5.99	Salerno Mercatello Hospital,
G	203.2	25.1	13.1	1099.4	11.99	Marinella Wharf, Port of Naples,
G	203.2	14.5	14.5	604.7	0.89	Main Switching Plant, Genoa
G	203.2	22.3	22.3	636.3	1.65	Mobil Oil Italiana, Naples
G	203.2	20.1	20.1	597.6	0.94	Railway Terminal, Naples (Corso A. Lucci)
G	203.2	19.2	19.2	575.2	1.65	Plant (Brindisi)
G	203.2	18.4	18.4	575.2	0.71	Plant (Brindisi)
C	203.2	22.4	22.4	280.0	6.40	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
C	203.2	20.1	20.1	246.4	9.80	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
C	203.2	20.1	20.1	493.7	5.21	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
G	203.2	30.2	20.1	1121.8	5.41	Belt (Expressway) East-West, Naples
G	203.2	30.2	20.1	897.9	3.23	Belt (Expressway) East-West, Naples
G	203.2	18.1	18.1	695.3	1.55	Swimming Pool – Scandone Pool, Naples
G	101.6	10.1	10.1	218.9	2.21	Casa Albergo in Viace Piave
G	215.9	25.1	25.1	709.5	3.76	Port of Naples
G	215.9	25.1	25.1	709.5	3.81	Port of Naples

G = Granular
C = Clay
Si = Silt

NB Maximum test load does not mean failure load

kN in compression were obtained, for total pile lengths averaging 30 m. Several pile tests to 600 kN were conducted, with a typical example given in Figure 14. Other tests indicated non-recoverable (plastic) deformations of up to 1.48 mm at 600 kN. A total of 503 similar piles have been installed in City One.

Example D (Intrusion-Prepakt (Far East) Ltd)

A small but particularly interesting application is illustrated in the works executed recently at Causeway Bay East Concourse (MTRC 430). Temporary support was needed to support a concrete floor slab, around an existing caisson during top down construction. Once a column had been placed on the caisson, the floor loading would be transferred to the column, off the minipiles. The jobsite was very congested due to a large number of simultaneous activities in the very limited area: no large scale piling equipment could have been accommodated, and for the small number of piles to be installed, would not have been economical in any event.

Eight minipiles were installed by rotary methods to an average depth of 14 m. The principal elements of the design were as follows:

INTERNAL DESIGN

Pipe o.d. = 152.4 mm   i.d. = 133.3 mm
$F_y = 355 \text{ N/mm}^2$
∴ Maximum capacity = 1522 kN.
Compared with maximum working load of 456 kN.

EXTERNAL DESIGN

- Consider end bearing: $\frac{\pi \times (0.1524)^2}{4} \times 5000 = 91$ kN for grouted pipe

- Consider skin friction: Average N = 40.
  Max load $= N \times 4.788$
  $= 191.5 \text{ kN/m}^2$

i.e. per metre length $= \pi \times 0.1524 \times 191.5$
$= 91.7$ kN
for sf = 3, safe load = 30.6 kN/m.
∴ To resist maximum working load, friction contribution is
$\frac{456 - 91}{30.6} = 11.9$ m
say 12 m long pile

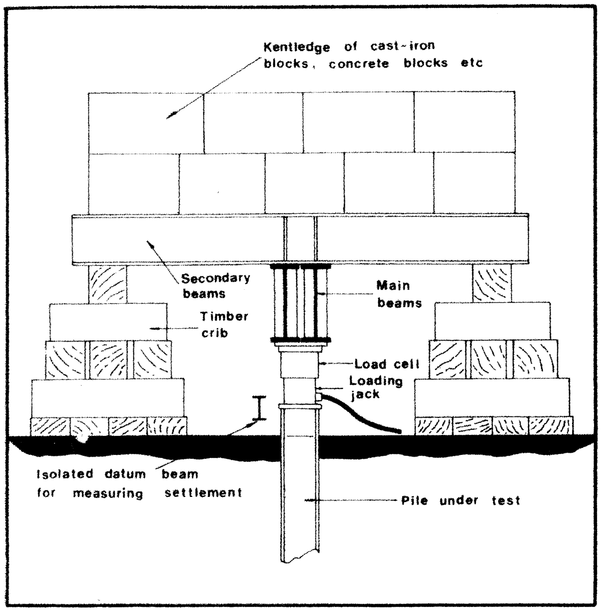


Fig. 12a Test loading – typical arrangement using Kentledge

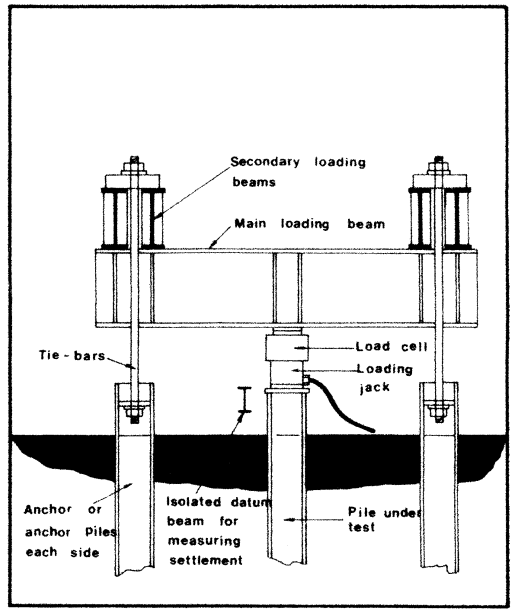


Fig. 12b Test loading – typical arrangement using anchors (Cornfield, 1974)

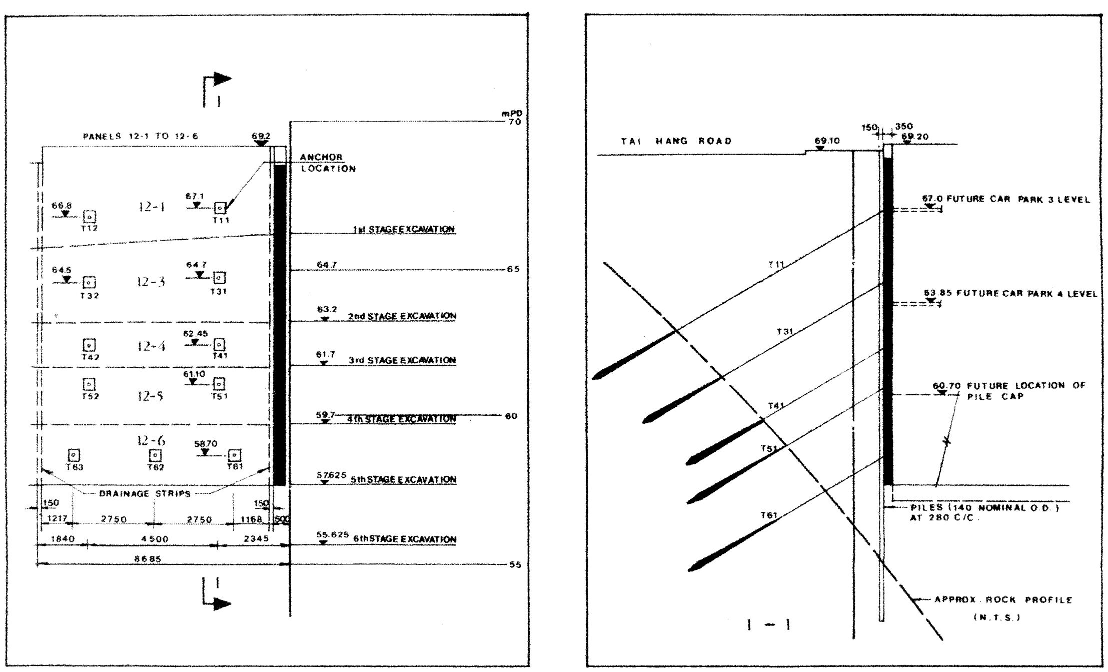


Fig. 13 Anchored minipile wall at Tai Hang Road (Courtesy of Gammon (HK) Limited)

Table XII
Typical minipile case histories, Hong Kong (By courtesy of the Bachy Soletanche Group)

Example	Contract Details	Purpose of Piles	Ground Conditions	Pile Characteristics	Working Load	Justification for selecting Minipiles
1	Ma Tau Kok Gas Works (Owner: Hong Kong and China Gas)	Transfer of light machine loads through compressible fill.	Fill overlying cdg	100 mm dia. hole, 6 m long. Single 32 mm HY bar. Gravity grouting. Qty 20	50 kN Compression	Very restricted working area between existing installations.
2	Hongkong & Shanghai Bank Annex (HSBC)	Underpinning of annex founded on raft to eliminate movements due to adjacent excavation.	Fill, marine deposits and cdg overlying granite	125 mm min. dia 25 m long. API tube (114/87 mm dia.) ±40 mm HT bar. Socketted 3 m into rock. Gravity grouting. Qty 154	660 kN or 945 kN Compression	Restricted headroom (less than 4 m).
3	Power Transmission Line (China Light & Power)	Foundations for Pylons	Silt (N:10-20)	168 mm dia. hole, 20 m long. Single 50 mm HY bar. Pressure grouting. Qty 654	410 kN Compression and Tension	Difficult access to large number of tower locations (Frequently only by helicopter)
4	MTRC 809: Cotton Tree Drive (MTRC)	Foundations for footbridge.	Fill, marine deposits and cdg overlying granite.	168 mm dia. hole, 26-37m long. One to three HY bars (40 to 50 mm). Socketted 5-6 m into rock. Gravity grouting. Qty 23	Up to 1230 (for three 50 mm bars) Compression	Small loading, bouldery ground Restrictions on ground water lowering and cost eliminated hand dug caissons

Example E (Colcrete Ltd)

The Hong Kong Country Club, at Deepwater Bay, is founded on conventional concrete piles through variable depths of fill and weathered volcanic rock. To minimise the total and differential settlements of the extension (Figure 15), a total of 76 560 kN minipiles were installed. Major factors favouring the minipile solution were the very restricted space and access conditions, minimum environmental disturbance and its financial and programming advantages.

The principal elements of the design were as follows:

INTERNAL DESIGN

Pipe o.d. = 139.7 mm i.d. = 121.3 mm
$F_y \geqslant 350$ N/mm^2
Working stress $\leqslant 42.5\% \, F_y$

$$\therefore \text{ Allowable working load } = 0.425 \times 350 \times \frac{\pi}{4}(D^2 - d^2) = 561 \text{ kN}$$

The exploitation of composite action was prohibited at the design stage by the Authority.

EXTERNAL DESIGN

Consider load transferred in 2 m rock socket only (maximum boulder diameter about 1 m).
∴ At working load, average rock/grout bond is

$$\frac{561 \times 1000}{\pi \times 220 \times 2000} = 0.41 \text{ N/mm}^2\text{, for 220 mm dia hole.}$$

From general information and local practice, this may be regarded as very conservative given the high quality of the bedrock.

For the same socket length, the average steel/grout bond developed at working load is 0.64 N/mm^2 which gives a factor of safety of 1.9 to 3.0 (depending on grout strength – from 20 to 40 N/mm^2) with respect to the allowances of CP 110 (1972)

A precontract test pile was installed and tested to verify the acceptability of the design.

Duplex drilling was conducted through the fill and weathered rock, with a down-the-hole hammer employed in the fresh rock. The use of diesel hydraulic self-tracking drill rigs greatly facilitated movements between piling locations. Grouting was executed with a 1:1 sand cement mix, w = 0.55, plus plasticiser/retarder additive, giving a typical 28 day strength of over 30 N/mm^2.

One further pile, of average length, was selected for testing during the production phase, and the load-settlement record is shown in Figure 16. Theoretically, an elastic deformation of 8.7 to 15.0 mm (at 1122 kN) was anticipated, with a permanent set of 4 mm regarded as acceptable. In fact, the recorded elastic deformation was under 8 mm, and the permanent set 0.74 mm.

Summary and conclusions

Minipiles were first installed in 1952 in support schemes for sensitive, historic buildings. Since then, the range of applications has widened greatly, principally into the areas of urban transportation developments and slope stabilisation. This growth has been facilitated by the progressive improvement in the performance and adaptability of the installation equipment available.

Their very nature makes them linearly more expensive than more conventional piling methods, featuring, for example, driven, or large diameter bored, systems. However, where particular combinations of ground conditions,

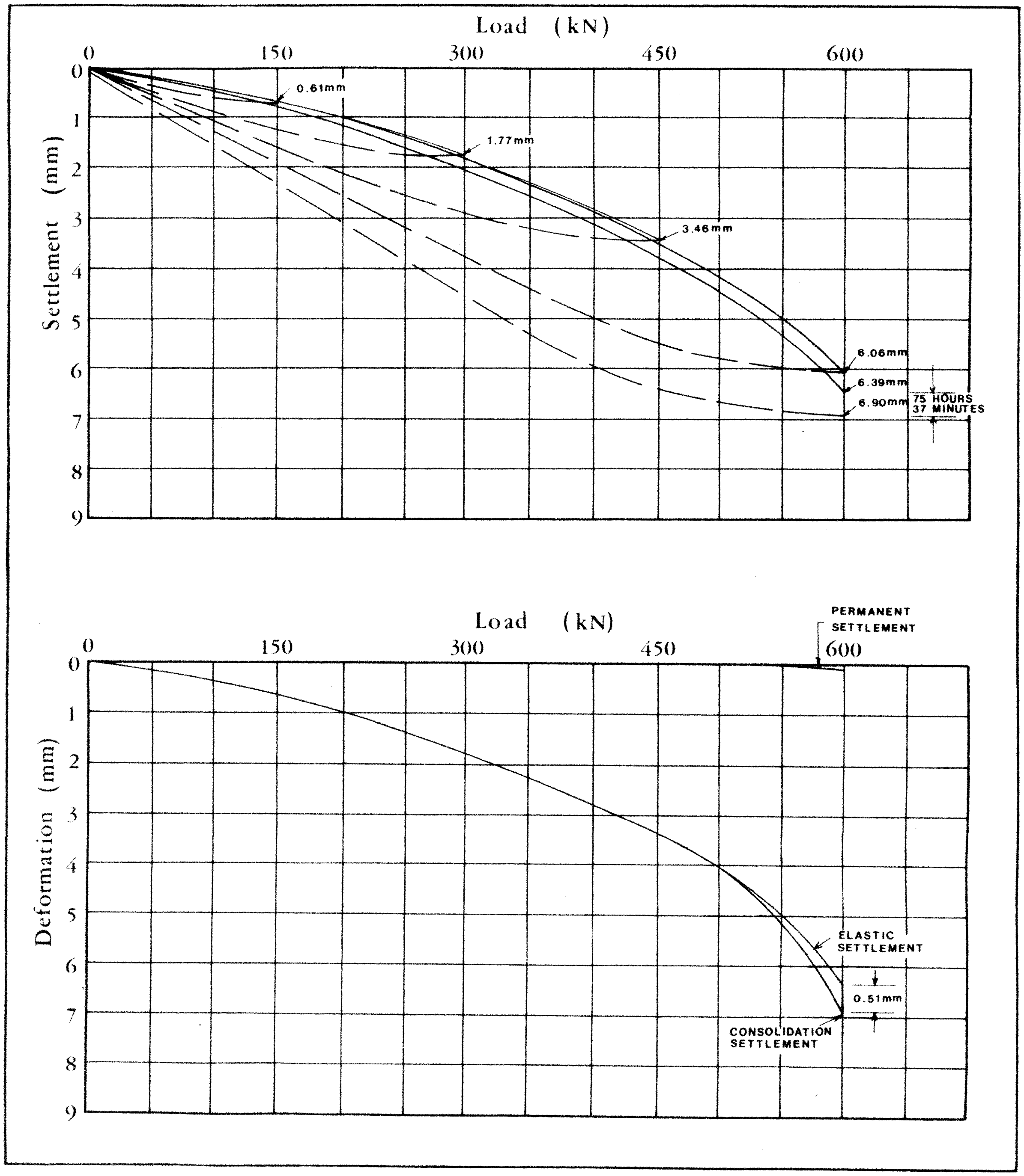


Fig. 14 Load/settlement, and load deformation curves, pile 32, Shatin Town Lot No. 1 (Courtesy of Vibro (HK) Limited)

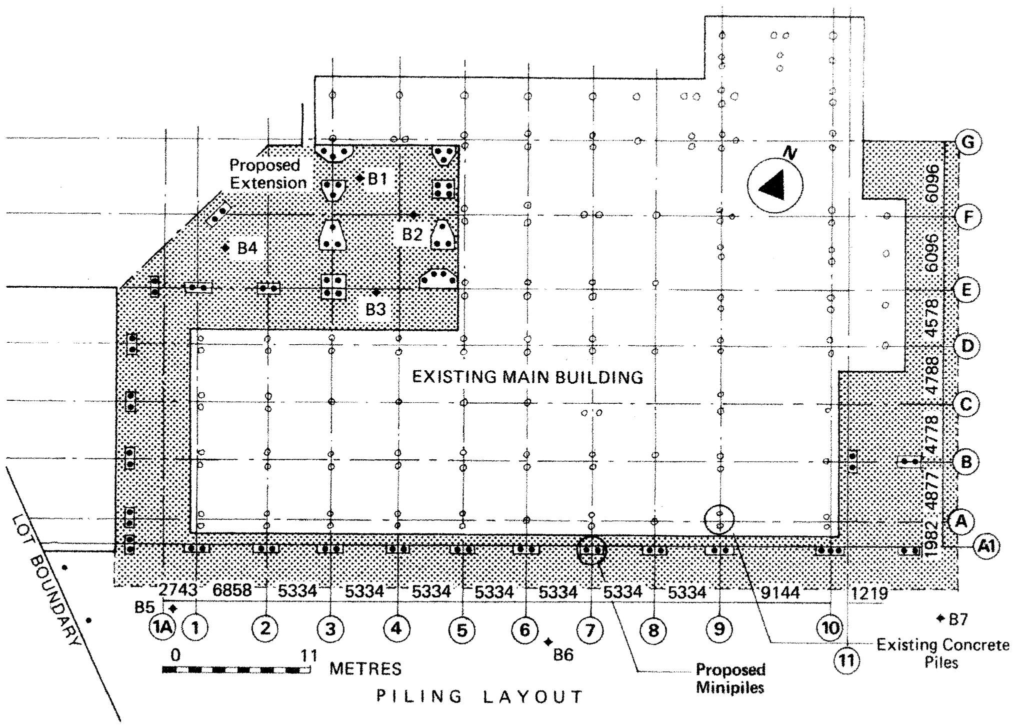


Fig. 15 Plan of Hong Kong Country Club showing proposed extension and minipile locations

space and access restrictions, environmental considerations, and performance criteria may exist, then minipiling offers a viable solution.

In Hong Kong, many successful applications can be cited, although it is doubtful if the full potential has been realised in what is undoubtedly a region very amenable to the options provided by minipiling.

It is hoped that this review will serve to underline the many advantages of the system, and thereby to encourage its exploitation, to the benefit of all parties in the contractual spectrum.

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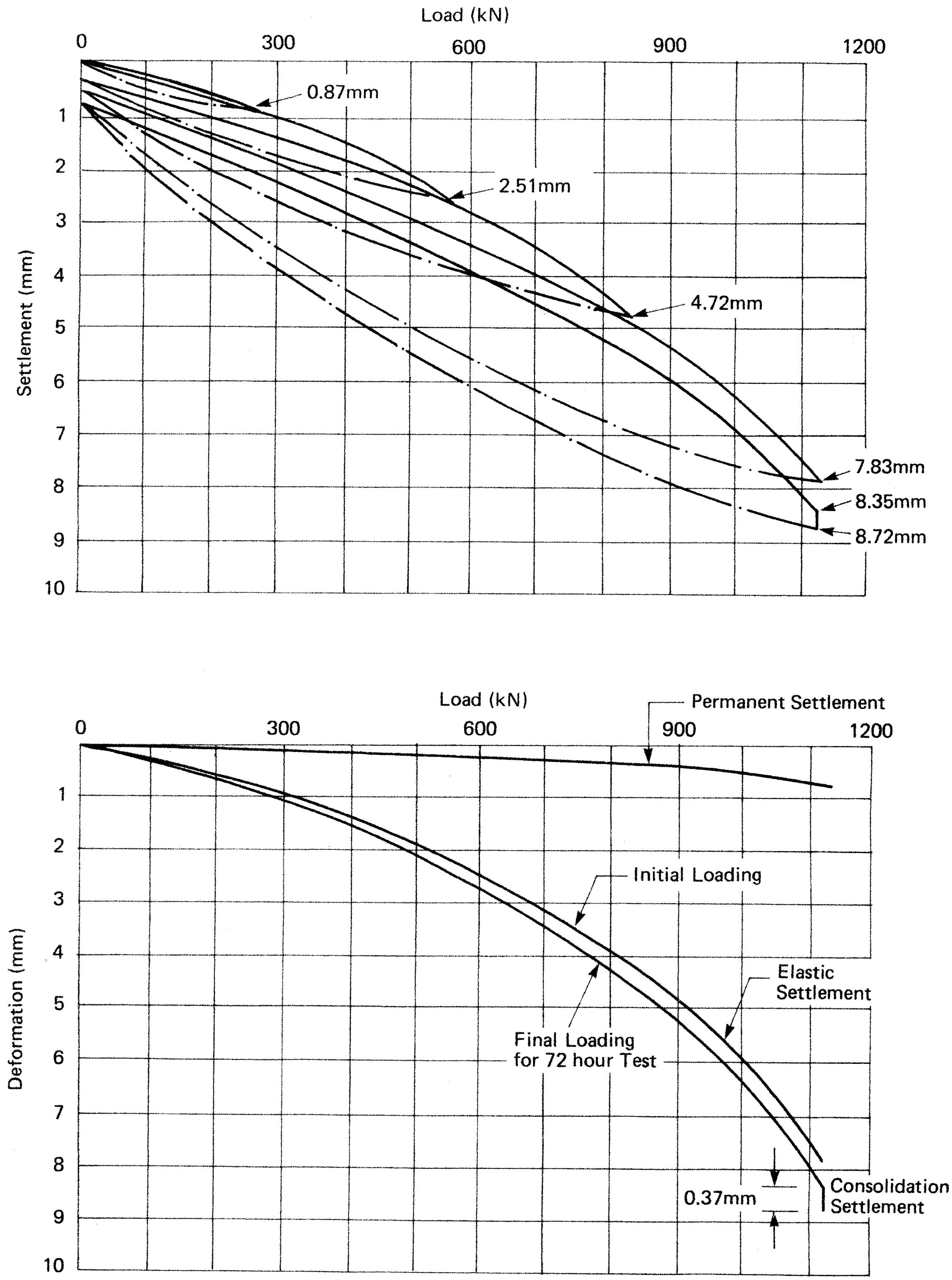

Fig. 16 Load/settlement, and load/deformation curves, pile NC7b, Hong Kong Country Club

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Relevant Codes of Practice and Standards

BS 12 : 1978	Specification for Ordinary and Rapid-hardening Portland Cement.
BS 882 : 1965	Aggregates from Natural Sources for Concrete (including granolithic).
BS 1881 : 1970	Methods of Testing Concrete – Part 4. Methods of Testing Concrete for Strength.
BS 3148 : 1980	Methods of Tests for Water for Making Concrete.
BS 4027 : 1980	Specification for Sulphate-resisting Portland Cement.
BS 5135 : 1974	Metal-arc Welding of Carbon and Carbon Manganese Steels.
BS 5400 : 1978	Steel, Concrete and Composite Bridges – Part 4. Code of Practice for the Design of Concrete Bridges.
CP 110 : 1972	The Structural Use of Concrete – Part 1. Design, Materials and Workmanship.
CP 2004 : 1972	Foundations.

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BE 21/5/010	UK Bridge Code for Reinforced Columns and Piers.

Swedish Code 150E-445.B.20

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