Minipiling at the Hong Kong Country Club

by D.A. BRUCE and C.K. YEUNG*

Occupying a superb location overlooking Deep Water Bay on the south of the Island (Fig. 1), the Hong Kong Country Club has for over 20 years been amongst the most prestigious venues in the Territory. To maintain its pre-eminence in terms of social and sporting facilities, considerable redevelopment is in hand, a major part of which is the extension of the main building.

The existing structure sits on variable fill, although it is founded through 139 No. 500mm diameter concrete piles to rock. Given the nature of the project (Fig. 2) and the existing rigid foundation design, it was clearly necessary to devise a solution to minimise both the differential and total settlements of the extension.

Early consideration was given to a cantilever extension from the existing structure but the magnitude of the extra loading potentially risked distress to the existing structure by overloading the foundation system. In addition, the upheaval to the Club's operations – which had to be maintained throughout – would have been particularly severe, and the cost of this scheme was high.

Attention was then given to a piling solution with the following requirements:

1) Construction technique had to cause least nuisance and noise to the Club membership, and to adjacent Ocean Park

2) Equipment had to operate in conditions of very restricted access, yet be capable of installing substantial piles to considerable depths

3) Minimum settlement was essential from the piling system

4) A short programme period was required (a limit of 74 calendar days was later set)

5) The piling system had to cause minimal disturbance to ground and existing structures

The authors are with Colcrete Ltd., Hong Kong.

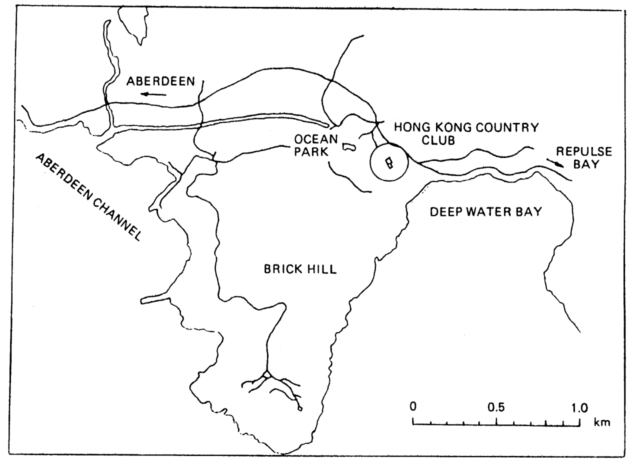

Fig. 1. Project location map.

As a final prerequisite, the piling solution had to be economically competitive.

Under less exacting criteria, a proposal featuring hand dug caissons would have been attractive. Hand dug caissons are widely used in the territory, and each is simply a hand excavated vertical shaft, from 800mm to 3000mm I.D., lined with concrete, and may reach to depths of 50m, depending on ground and ground water conditions. After completion, i.e. after founding into rock, the caisson is then filled with the material of the pile, e.g. concrete and steel reinforcement.

However, the particular requirements of this project pre-empted the

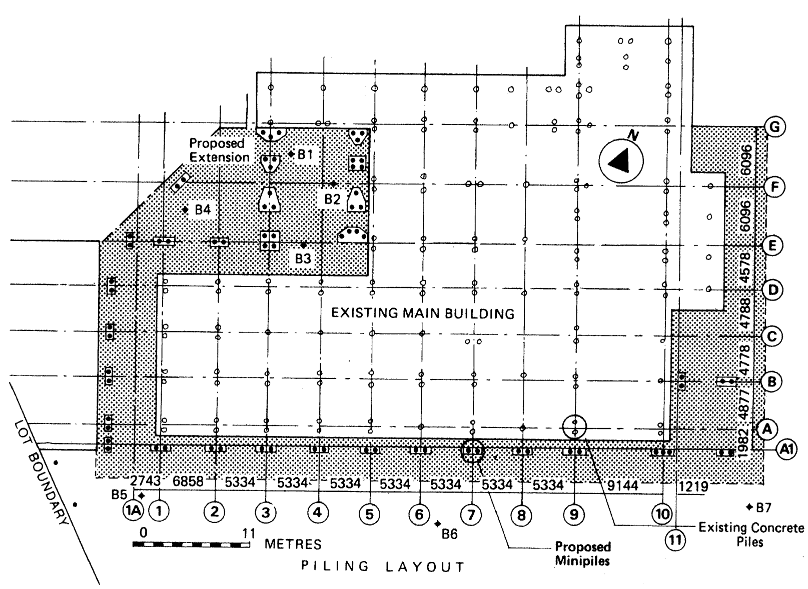

Fig. 2. Plan of Club showing proposed extension and minipile locations.

- Cast in-situ concrete piles (bored or driven)
- Steel piles
- Special piles including composite types

In the absence of a specific category for minipiles, discussions with the Building Ordinance Office revealed that the pile type in this instance had to be considered internally either as a concrete pile or as a steel pile; design of pile capacity featuring composite action would not be permitted.

Pile material

A simple calculation based on the maximum average compressive stress of $5N/mm^2$, allowed under the Building Regulations, showed that to design on the basis of a concrete pile would require a pile diameter well in excess of the standard minipile range (75-250mm). The alternative of designing on the basis of a steel pipe was therefore examined. For ease of handling, a single element as opposed to a large number of bars was considered. ERW steel pipe was located with the following properties: O.D. = 139.7mm, I.D. = 121.3mm, $fy \geqslant 350N/mm^2$ (N80 Grade, to BS1775). Agreement was given by the Authority to use a working stress of 42.5% fy, because although the pile would not be subject to driving stresses, a test load of 2 x WL would be imposed.

Thus, the theoretical permissible working load, assuming a steel pile action, is:

$$0.425 \times 350 \times \frac{\pi}{4}\left[D^2 - d^2\right] = 561kN$$

where D = 139.7mm
and d = 121.3mm

Given the ERW steel pipe, a borehole diameter of 220mm was selected as a standard drill size which would also ensure adequate concrete/grout cover to the steel.

It is interesting to note that with design criteria and principles accepted in the UK, this massive and strong steel pipe could have been safely and economically replaced by a group of four 18mm diameter high yield bars, with spiral reinforcement to prevent bursting.

Ground/grout interface

As shown in Table 1, the N values of the fill, and residual soil, where present, are low and variable. Furthermore, the decomposed volcanic material, although of high frictional characteristics, is of variable thickness and depth. As a consequence, it was determined to ignore the contribution to load transfer of any of the overburden materials and to socket the piles firmly into solid rock.

Due to a high slenderness ratio, minipiles transfer load by side shear, as opposed to end bearing. When considering bond values between rock and grout, the following relationship is often used as a guide:

$$\text{skin}_{\text{ult}} = \frac{\text{UCS rock}}{10}$$

for values of UCS up to $42N/mm^2$ and given grout of comparative strength.

Assuming the very hard volcanic bedrock to be at least of this strength, this relationship gives a working bond value of $1.4N/mm^2$, allowing a factor of safety of 3.

For a pile working load of 561kN, and diameter of 220mm, the required embedment length is:

$$L = \frac{561 \times 1000}{\pi \times 220 \times 1.4} = 580mm$$

Since it was recognised from the site investigation that boulders of up to 1m diameter could be expected, particularly just above rock head, it was determined to continue drilling in sound rock to a depth of 2000mm so as to ensure termination in bed rock, and not in a large boulder.

Thus, the actual average working bond (in compression) was further reduced to $0.41N/mm^2$, – a figure regarded as conservative, particularly with respect to values used in practice for rock anchors (Table 2).

Steel / grout interface

With regard to the average steel/

Table 2. 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)
Basalt	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 > $20N/mm^2$)	Uniaxial compressive strength—30 (up to a maximum value of $1.4N/mm^2$)	Uniaxial compressive strength—10 up to a maximum value of ($4.2N/mm^2$)	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		2-2.5 (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	Britain—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)
Basalt & 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 porphyrite	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

use of hand dug caissons, a major consideration being the possibility of ground water table draw-down and the associated removal of fines in suspension, potentially leading to surface settlements of the order of 3mm to 8mm per metre of draw-down.

After a careful review, a minipiling system was adopted having due regard to all the technical, logistical, programming and economic parameters: in essence, conventional drilling and grouting equipment could be quickly deployed, and the small pile diameters minimised the disturbance to the ground and the environment.

Site investigation

A pre-tender site investigation was conducted, and the major ground conditions are summarised in Table 1. Assuming rock head to be represented by slightly weathered volcanics of minimum core recovery 85%, rock head contours were predicted (Fig. 3a), as the basis for pile length estimation. Fig. 3b shows, for comparison, the rock head levels recorded during piling. In general, the rock head dipped to the southwest and so gave foreseen rock head depths of 11m to 23m in the region of piling (Figs. 3c and 3d).

Project requirements

Analysis of the superstructure loading schedule gave working loads for the 34 new columns ranging from 490kN to 2120kN. Each new column location was then examined with a view to determining a practical pile layout, and this process led to the conclusion that by adopting a standard individual pile working load of about 550kN, all new columns could be founded on pile caps spanning from one to four minipiles.

The overriding requirement of the piling system was that "any settlement and particularly differential settlement of the superstructure construction must be kept to an absolute minimum, and it is therefore required that the design should be based on founding on solid rock strata". The specification also stipulated that the permanent settlement should be less than 7mm after removal of the full proof load, i.e. 2 x working load (WL). Since the maximum number of piles per group did not exceed four, and bearing in mind that each pile was to be founded into rock, no "group reduction factor" was applied in the design of individual piles.

Statutory requirements

With respect to the requirements of the Building Ordinance Office, pile types used in Hong Kong may be classified as follows:

- Precast reinforced concrete

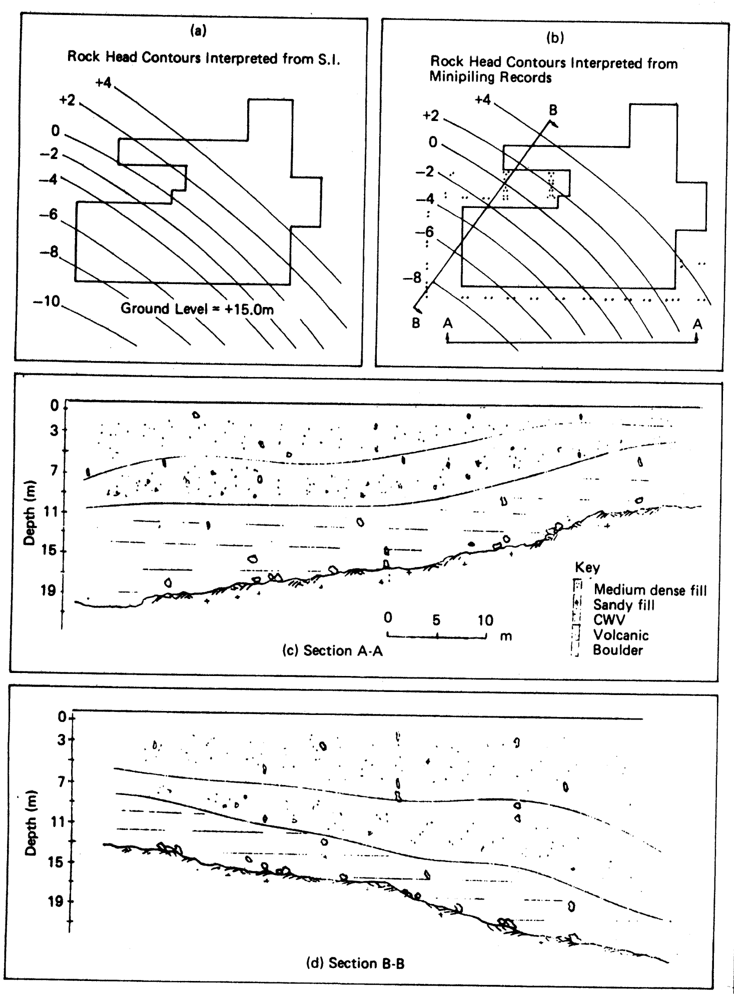

Fig. 3. Site investigation interpretation.

Table 1. Summary of site investigation borehole logs B1 to B 7.

Ground	Description	Notes
5 to 14m Overlying	Medium dense grey-yellow silty to coarse sand with bricks, gravel, and cobbles (FILL) Occasional fresh rock boulders	N = 6 to 70, variable, but often below 20
0 to 2.4m Overlying	Firm dense yellow brown clayey SILT/SAND (RESIDUAL SOIL)	N = 16 to 94 Present in B5 + 6
1.4 to 11.7m Overlying	HIGHLY WEATHERED dense grey yellow VOLCANICS with rock fragments: silty sand and gravel	N = 35 to 200 but usually 200+
Bedrock	Moderately strong – Very strong green grey fine grained Moderately – slightly weathered VOLCANIC ROCK with closely spaced stained joints	Core recovery 87–100% RQD typically 20–50% but often 0%

grout bond mobilised in the 2m socket, a figure of 0.64N/mm^2 may be calculated at the working load. CP110 (1972) relates ultimate anchorage bond stresses (in tension) to the characteristic concrete strength as shown in Table 3.

Hence the calculated working bond represents a factor of safety of:

$$\frac{1.2 \text{ to } 1.9}{0.64}$$

i.e., 1.9 to 3.0 depending on concrete strength and assuming the steel pipe to represent a plain bar.

Grout mix design

Based on prior experience, an economical 1:1 sand cement grout was chosen with a w/c ratio of 0.55 (Fig. 4). Ordinary Portland cement was used, together with Zone 2 building sand and fresh mixing water. Small quantities of a plasticiser/retarder were also added to improve the pumpability of the mix and to generally ease extraction of drill casing. A target 28-day strength of 20N/mm^2 was specified, but overall, the average recorded strength was in excess of 30N/mm^2.

Spacing of piles

The Building Regulation stipulates that for "piles bearing on rock, the spacings may be reduced (from either the pile perimeter, or 1m) to not less than 750mm or 2 times the least width of the pile, whichever is the greater". Given that the minipile system proposed would involve little ground disturbance the minimum interpile spacing of 800mm actually required by the details of the new column design was therefore wholly acceptable.

Table 3. Maximum permissible bond stresses (CP110, 1972).

Type of Bar	Concrete Grade 20	25	30	40+
	Maximum bond stress, N/mm^2			
Plain	1.2	1.4	1.5	1.9
Deformed	1.7	1.9	2.2	2.6

Construction

Drilling was conducted by a number of Casagrande SCM 5006 diesel hydraulic track rigs (Fig. 5), each equipped with a Hands England H2 FJ rotary head. Hutte system rotary duplex equipment with water flush was used to drill and case to rock head, whilst drilling in the hard rock was most quickly conducted by down-the-hole hammers. Upon reaching full depth the drill rods were extracted and the hole thoroughly flushed. Throughout, special care was exercised to observe the specified pile tolerances: ±50mm in plan and a verticality not exceeding 1 in 75.

The pipe lengths were then homed, the required high quality welding being manual metallic arc and carefully conducted in accordance to BS5135. Peripheral spacer bars were used to ensure pipe verticality and concentricity.

Grout was mixed in a Colcrete colloidal mixer, model SD10, and pumped via a Colcrete rotary screw Colmono 10 to the hole where a tremie tube permitted grouting from the base upwards. Throughout injection, regular control checks on the fluid grout were made viz. Baroid mud balance, bleed, and Colcrete flowmeter, in addition to standard 150mm cube sampling for strength tests.

Once good quality grout was observed to emerge at the top, the drill

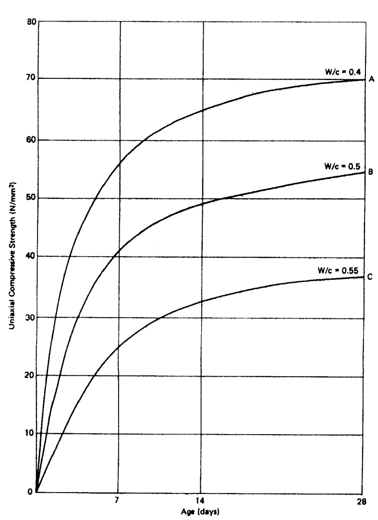

Fig. 4. Strength development curves for sand cement grouts (sand / cement ratio = 1).

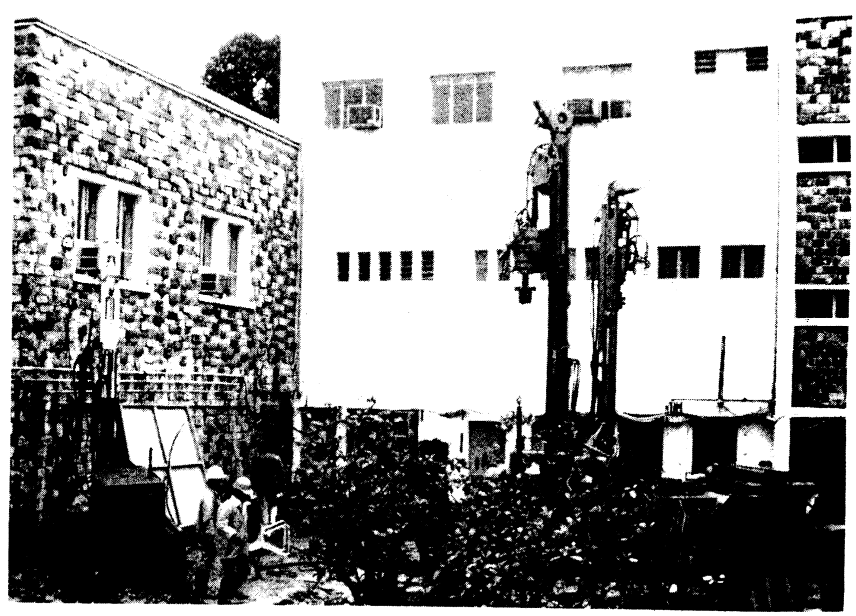

Fig. 5. Diesel hydraulic drill rigs on site.

casing was withdrawn, whilst the grout level was maintained at ground surface to avoid "necking" of the pile in the soft ground. The pile production programme was organised such that at least 24 hours elapsed between the installation of piles in the same group to avoid disturbance to the setting grout. Overall, the 76 piles required a total of 1337 metres of drilling (average 17.6m, range 11.6m to 23.8m) and 305 tonnes of grouting materials. Grout consumptions ranged from 2.1 to 16.4 times the calculated hole volume, and averaged over 3.6, highlighting the relatively high groutability of the permeable fill in particular.

Test pile

In advance of production piling a test pile was planned to demonstrate and test construction procedures, and to provide performance data as a field check on design parameters. In addition, one of the production piles was selected for proof testing to twice working load, at the conclusion of the piling works.

The test pile was installed, by the method described above, at a location to the southwest of the main building. The summary piling record is shown in Table 4. At the time of installation the special high yield steel of the production piles was not available and so the pile incorporated a pipe with properties: O.D. 141.3mm, I.D. 122.2mm, fy 241N/mm^2 (to API 5L Grade B).

Applying the same steel working stress limit of 42.5% as above, the pile was therefore theoretically rated as having a working load of 403kN.

Due to the restricted access at the time, reaction for the applied load was supplied by two inclined rock anchors. Load was applied in increments equivalent to 0.15WL (61kN) and held for 10 minutes to record any time related settlement. The sequence of loading was as follows:

- Load to WL and lock off for 24 hours
- Unload, reload to 1.5WL and lock off for 24 hours
- Unload, reload to 2WL and lock off for 24 hours
- Unload, reloading continued to 2.3WL when temporary problem with loading system occurred, requiring lock off at this load for 24 hours
- Unload, then reloading continued to 2.7WL at which point the limit of the loading system was reached. Lock off for 48 hours

The load-settlement curve indicated the performance to be satisfactory and linear in behaviour, with the final test load of 1090kN corresponding to 115% fy. Table 5 summarises the major features. A consolidation of 0.47mm was recorded in the 48 hours during which load was held at 2.7WL. Approximately 80% of this amount was recorded within the first 12 hours of testing.

Production test

Pile NC 7b (Table 6) was selected by the Authority for testing, and was of average length. Load was applied in increments of 0.5WL

Table 4. Pre-production test pile summary sheet.

Summary Drill Log

Depth (m)	Strata	Av. Pen. (min/m)	Notes
0–15.5	Fill	3.5	Soft sandy fill, occasional small boulder (< 1m). No flush return
15.5–18.9	Completely Weathered Volcanics	8.3	Firmer ground No flush return
18.9–21.9	Fresh Volcanics	13	Very hard rock No flush return

Installation

Pipe O. D. = 141.3mm, I. D. = 122.2mm, Length = 21.9m
fy = 241N/mm^2 (Test Pile only)

Grouting

OPC	: 2430kg	Sand 2430kg	
Colplus	: 7.49 1	w/c = 0.55	
Av UCS	: 40.3N/mm^2	(21 days)	
	45.3N/mm^2	(8 days)	

Table 5. Test pile performance summary.

Load		Total Settlement (mm)	Permanent Settlement (mm)
WL	(403kN)	2.75	0.25
2WL	(806kN)	6.00	0.75
2.7WL	(1090kN)	14.50	5.45

Table 6. Production pile NC 76 summary sheet.

Summary Drill Log

Depth (m)	Strata	Av. Pen (min/m)	Notes
0–3.0	Fill	9	Soft, sandy No flush return
3.0–3.5	Boulder	65	Very hard erratic
3.5–11.5	Fill	10	Soft and sandy with occasional cobbles. Little flush return
11.5–11.9	Completely Weathered Volcanics	10	Firmer. More flush return
11.9–13.9	Fresh Volcanics	70	Very hard rock

Installation

Pipe O. D. = 139.7, I. D. = 121.3mm, Length = 14.7m
fy > 350N/mm^2 (Typical of production piles)

Grouting

OPC	: 3420kg	Sand: 3420kg
Admixture	: 4.86 1	w/c : 0.55
Typical UCS	: 20N/mm^2 (7 days)	
	: 35N/mm^2 (28 days)	

(280kN), with 2 minute intervals, and the pile was loaded cyclically but continuously in this fashion before being held at 2WL for 72 hours. Load/settlement behaviour is shown in Fig. 6 with consolidation records in Fig. 7. It was possible to conduct this test using kentledge, as shown in Fig. 4.

Prior to this production test, the Authority deemed that the target performance was to be based on composite action within the pile. Calculations were made which gave a total theoretical elastic deformation of 8.7mm to 15.0mm (allowing for possible variations between modular ratios and E values for the grout) for this pile length. A permanent settlement target of 4mm was also set. This analysis was therefore consistent with the total settlement criterion of 15mm specified in the Building Regulations.

As shown in Fig. 7, the recorded figures were 7.98mm and 0.74mm, respectively (the corresponding figures were 2.15mm and 0.36mm, respectively, at working load).

These small settlements, together with the very low consolidation values recorded, permitted the engineering conclusion that the piles, as installed, would readily and safely fulfil their service requirements.

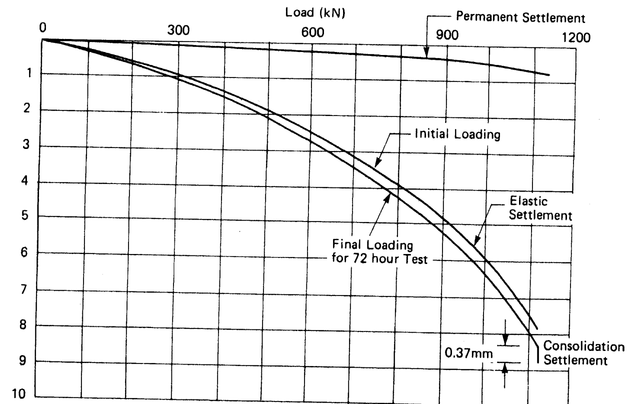

Fig. 6a. Load / settlement curve for production pile NC 76.

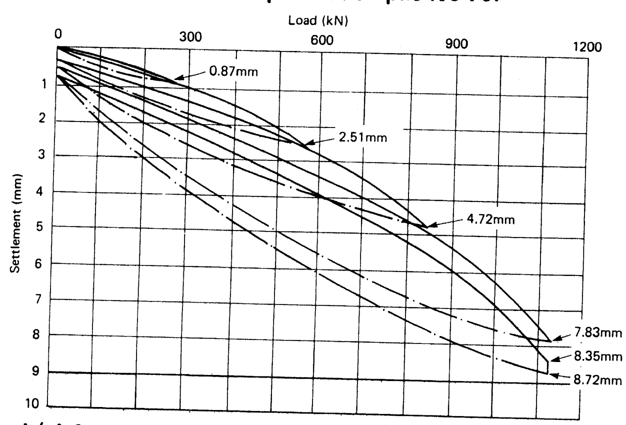

Fig. 6b. Load / deformation curve for production pile NC 76.

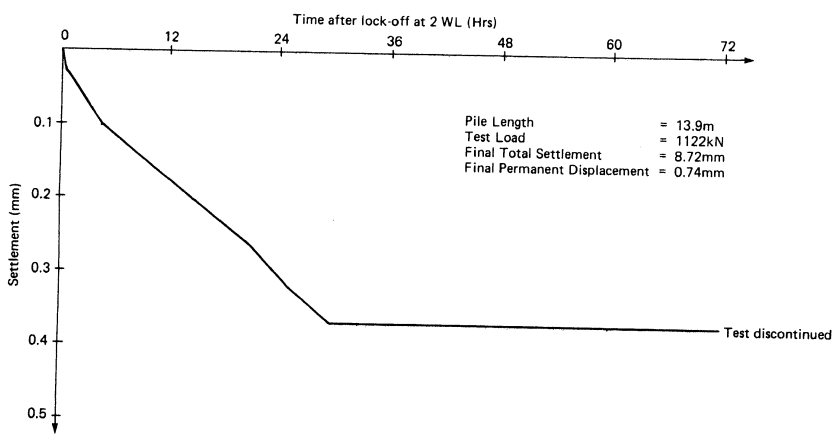

Fig. 7. Settlement / time data for production pile NC 76.

Conclusions

This case history illustrates that minipiles can provide an attractive practical solution where special problems exist, such as limited site access, rigorous performance criteria, and minimal disturbance during construction.

For the pile design and construction technique outlined, full scale site testing confirms an adequate load safety factor and one of the major minipile characteristics is displayed in the performance under loading, namely small total settlement and very small permanent settlement.

Bearing in mind that minipiles have enjoyed widespread application throughout Europe since the 1950s, it is hoped that successful demonstrations of the type described herein will lead to their freer exploitation in other parts of the world where their particular characteristics will be of advantage to clients, consultants and contractors.

Acknowledgements

In the preparation of this paper the authors have benefited greatly from the help and advice given by Messrs L.R. Thompson, J.A. Perkins and R. Chung (Mitchell, McFarlane, Brentnall & Partners), Mr. A.W. Wilkinson (Prescott Stutely Design Group), Mr. A.G. Bailey, (Hong Kong Country Club), and Dr. G.S. Littlejohn (Colcrete Ltd.).

References

BS1775:1967: Steel tube for mechanical, structural and general engineering purposes.

BS5135:1974: Metal-arc welding of carbon and carbon maganese steels

CP110:Part 1:1972: The structural use of concrete Hong Kong Building (Construction) Regulation: 1976

Littlejohn, GS and Bruce DA: 1977: *Rock Anchors – State of the Art*, Foundation Publications Ltd.