Recent Examples of Underpinning using Minipiles

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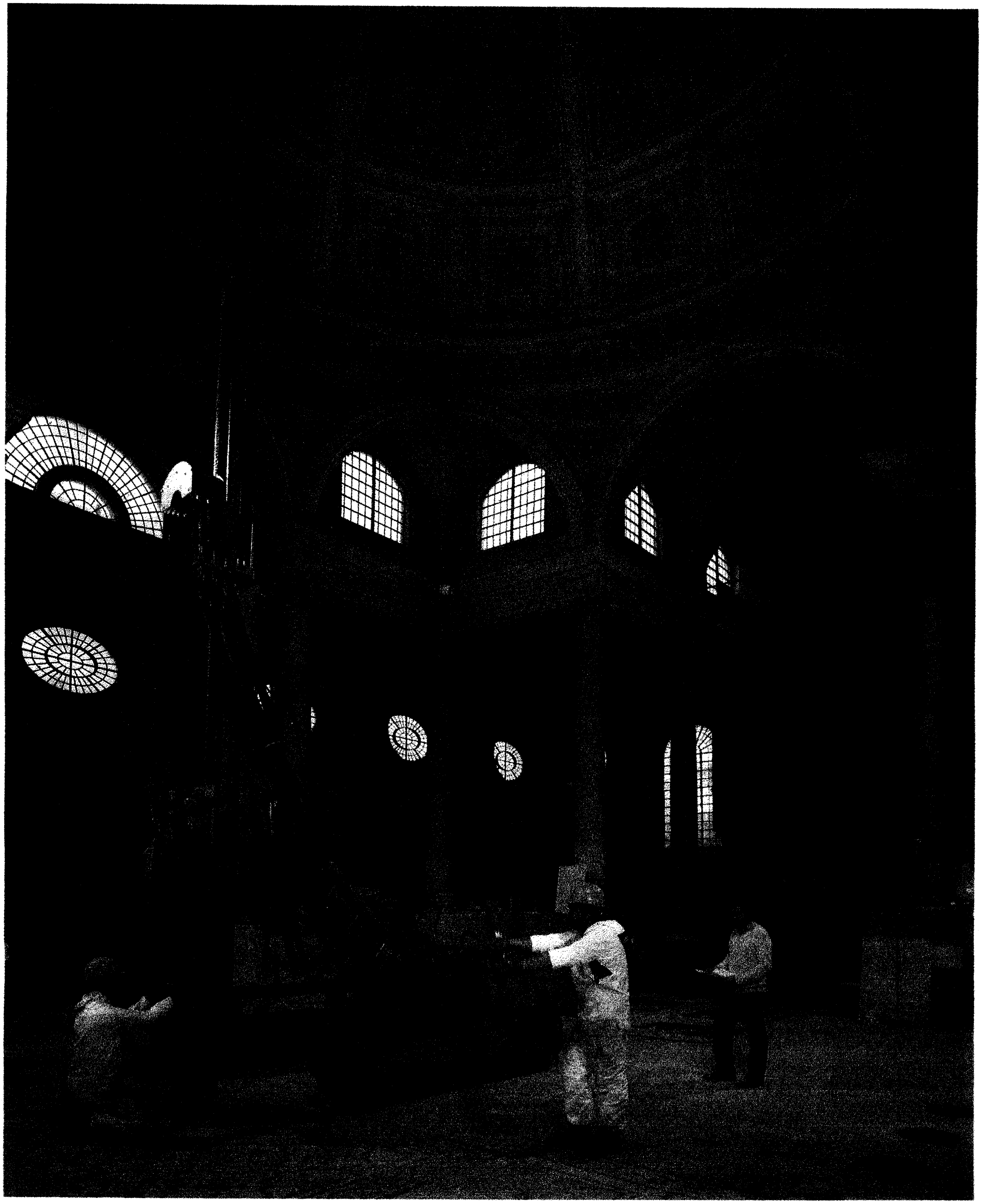

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SYNOPSIS

Small diameter bored piling ("minipiling") has become a very popular construction technique throughout the world. A principal application is in the underpinning of structures undergoing remedial and rebuilding operations. Minipiles have greatest value where ground conditions are very variable, where access is restrictive, where environmental pollution aspects are significant, and where structural movements in service must be minimal. The paper firstly provides a statement of contemporary minipile technology, and gives general background on design, construction and performance in particular. These topics are then amplified upon during descriptions of six recent minipile projects executed in England. It is concluded that the minipile concept is a tried and proven technique which still offers great flexibility and potential to engineers faced with underpinning problems.

1. SCOPE

The technique of minipiling was first applied in Italy in 1952, and particularly since the expiry of the original patents, has gained worldwide application and acceptance. This is reflected simply in the range of names: minipiles, micropiles, root piles, pali radice, needle piles, pieu racine, Wurzelpfahle, and Estaca Raiz. All, however, refer to a "special type of small diameter pile" (Koreck, 1978).

Their relatively small diameter is clearly distinctive although the actual insitu diameter is usually greater than the nominal designed. Mode of construction is also a definitive aspect, as it features the standard drilling and grouting equipment and techniques used in ground treatment or anchoring, as opposed to the larger scale plant of the piling contractor. For the purposes of this review, however, types of displacement piles installed by hammers, jacks or vibrators and which can occasionally provide attractive solutions, are excluded.

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

2. MAJOR CHARACTERISTICS

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

The construction is characterised by equipment ensuring minimum vibration, ground disturbance and noise, and capable of operating in awkward and restricted access conditions. Thus although their nature usually ensures that they are lineally more expensive than conventional piles, (e.g. driven, sheet – or H-piles) they may be the only guaranteed solution given a particular set of ground, site, and performance conditions.

Regarding their 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 1,000kN are feasible, whereas the allowable load on the 400mm minimum diameter conventional bored pile allowed in the relevant German DIN is 300 to 370kN. Similarly, Fenoux (1976) quotes a test on a 150mm diameter minipile which reached 1,700kN without apparent distress. A survey of the major European specialists confirms that ranges of up to 600kN working load are standard.

Table 1 provides a summary of typical performance data.

3. APPLICATIONS

The original applications in Italy were in the strengthening and underpinning of historic sensitive structures. From the mid 1960's, similar systems became popular in Germany, mainly in association with the underground construction of roads, subways and metros as in Munich, Hannover and Berlin. By the early 1970's, upon the expiry of the original patents, the growth of new proprietary systems had been stimulated by the challenge of providing solutions to complex foundation problems in urban environments, and so minipiles were being used in both compression and tension (Fig 1) and as retaining walls (Photograph 1).

Preloading facilities were then developed, thereby eliminating any subsequent structural settlement and distributing the pile reactions according to pre-established patterns. Resetting of the original configuration of structures distorted by differential settlements also, therefore, became possible.

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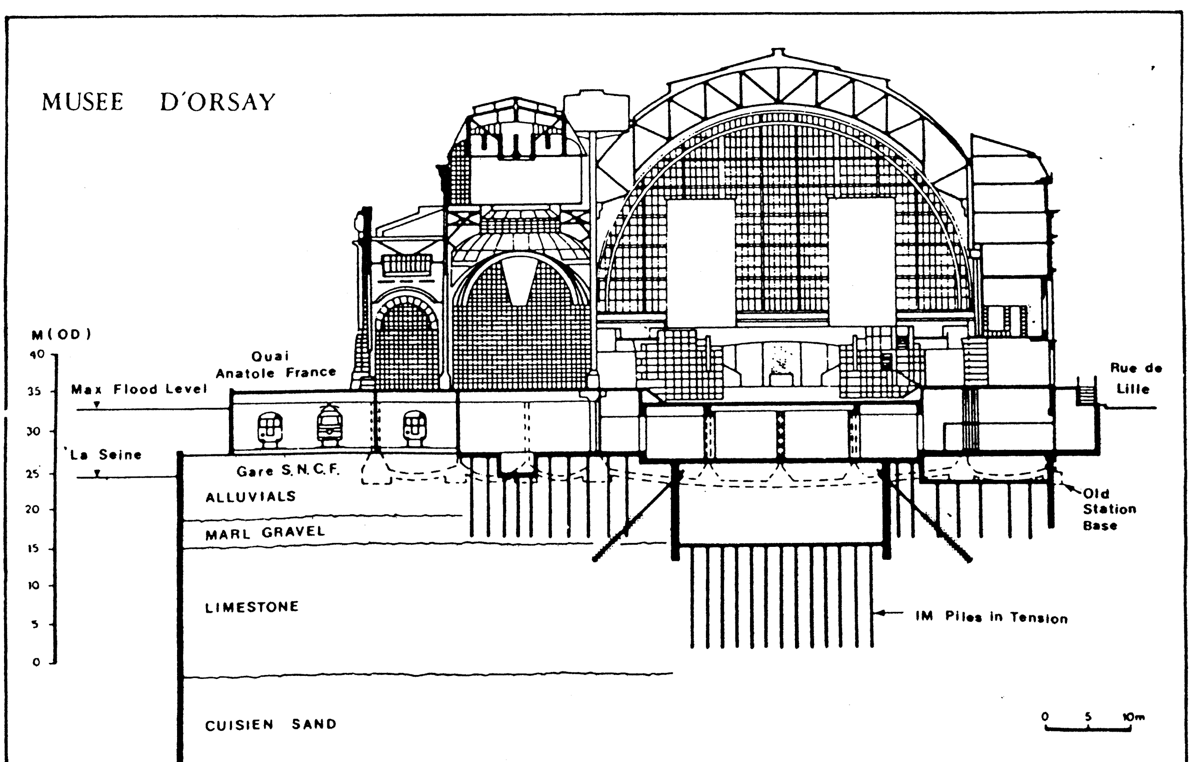


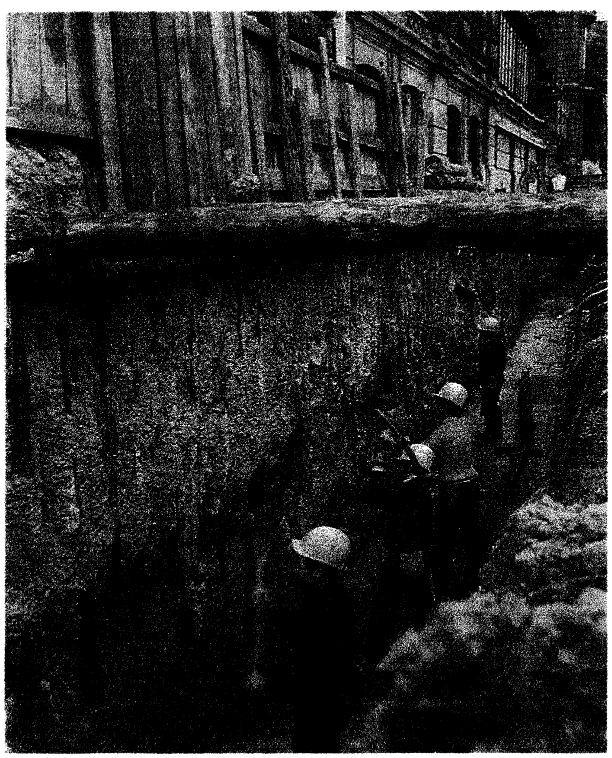
Fig 1. Musée d'Orsay Paris. Arrangement of minipiles (Soletanche Technical Brochure)

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

*Not necessarily failure load

Table 1. Summary of performance data (FHA Report 1976)



Photograph 1. The use of minipiles to form a retaining wall (Bruce & Yeung, 1983)

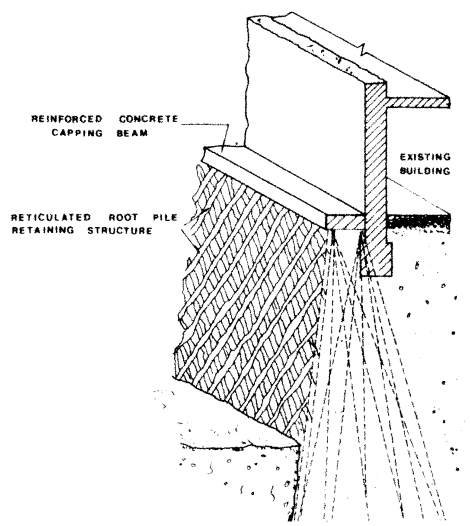


Fig 2(a). Principle of RRP System (FHA Report 1976).

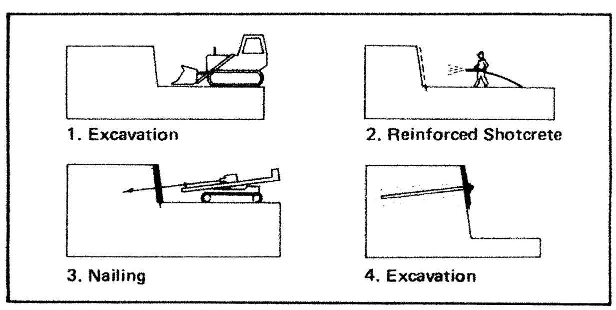


Fig 2(b). Principle of Soil Nails (Stocker, 1979)

Nominal Diameter	Nominal Cross Sectional Area	Mass	Hot Rolled Mild Steel Bar (BS 4449)		Cold Worked High Yield Bar (BS 44461)	
			Failure Load	Characteristic Strength	Failure Load	Characteristic Strength
mm	mm2	Kg/m	KN	KN	KN	KN
12	113.1	0.888	51	28	-	-
16	201.1	1.579	90	50	111	93
20	314.2	2.466	141	78	173	134
25	490.9	3.854	221	123	270	209
32	804.2	6.313	362	201	442	342
40	1256.6	9.864	565	314	691	534
50	1963.7	15.413	-	-	1080	834

Table 2. Details of Typical Reinforcing Steel

Most recently, systems of vertical/subvertical piles (Reticulate Root Piles – Fig 2a), and horizontal/subhorizontal piles ("Soil Nails" – Fig 2b) have been increasingly employed for the stabilisation of natural and excavated slopes or faces. Under such circumstances, akin to inserting reinforcing bars in mass concrete, blocks of ground are engaged, and these may be regarded as acting monolithically in stability calculations involving overturning, vertical base reaction, and horizontal shear.

4. DESIGN

4.1 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, whilst 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 develops from skin friction, rather than end bearing: a pile 200mm in diameter and 5m long, has a peripheral shaft area 100 times greater than the cross sectional area. Furthermore, settlements of 10 to 20 per cent are necessary to mobilise full end bearing capacity compared to 0.5 to 1.0 per cent for friction piles to mobilise maximum shaft resistance.

4.2 Internal Load Carrying Capacity

4.2.1 Steel

The reinforcing element usually consists of either a "cage" of High Yield bars, supported by a helical reinforcement, or one (or a group of) high strength bar. In certain circumstances where lateral stresses have to be resisted, pipes are used. Typical details are provided in Table 2.

For bored piles and for other pile types where no driving stresses are inflicted CP2004 (1972) allows working stresses of up to 50% yield stress. Assuming the yield stress to be about 85% of the ultimate, then for piles to be tested to twice working load the load component (at WL) ascribable to the steel should not exceed 42.5% of its ultimate strength.

Where such piles project above ground level the allowable stresses are as indicated in Table 3.

Slenderness ratio l/r	Grade 43 (mild steel)		Grade 50 (high yield stress steel)	
	N/mm^2	ton/in^2	N/mm^2	ton/in^2
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 3. Allowable stress in axial compression for piles projecting above soil level. (Cornfield, 1974)

A grout cover of at least 20mm is typical for permanent protection in normal soils. For tension piles in very aggressive conditions, a further protection in the form of a plastic or steel liner is prudent.

4.2.2 Grout

Typical materials 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 control over the properties of cement grouts is the water/cement ratio (w/c). It determines rate of bleeding, subsequent plasticity and ultimate strength of the grout. The extent to which these, and also fluidity, are related to w/c is shown in Figure 3 (neat Type 1 cement). Excess water causes bleeding, low strength, increased shrinkage and poor durability. Typically, w/c ratios 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 (5mm down to 75μm) and for long pumping distances the maximum size should ideally be reduced to 0.5mm 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 30-35N/mm^2.

Admixtures can be added in relatively small quantities to modify grout properties, and especially those to prevent shrinkage, allow reduction in w/c (whilst maintaining fluidity/pumpability), to accelerate or retard setting, and to prevent bleeding (thereby discouraging corrosion). Most commercial admixtures are compatible with Types I and III Portland cements, but many are incompatible with high

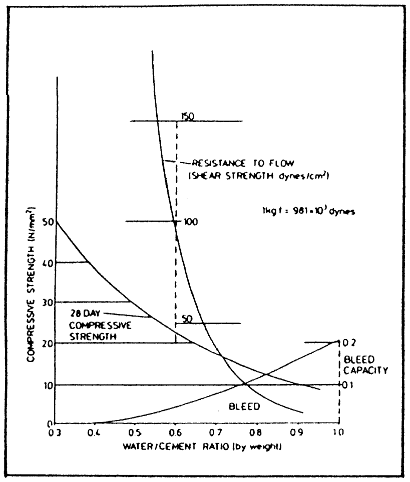


Fig 3. Effect of water content on cement grout properties (Littlejohn, 1982)

alumina and super sulphated cements. Admixtures should not be regarded as a replacement for good grouting practice and must not be used indiscriminately.

4.2.3 Grout/Steel Bond

This parameter has significance in that it is the mechanism of load transfer from steel to ground, and acts to promote composite action of the pile components. A great body of research has been conducted into the nature, magnitude, distribution and controls over grout/steel bond characteristics (e.g. Littlejohn and Bruce, 1977). Furthermore, CP 110 (1972) relates ultimate anchorage bond stresses (albeit in tension) to the characteristic concrete strength as in Table 4.

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 strength of the steel will be the limiting factor.

	Concrete grade			
Bar type	20	25	30	40 or more
	N/mm^2	N/mm^2	N/mm^2	N/mm^2
Plain bar in tension	1.2	1.4	1.5	1.9
Plain bar in compression	1.5	1.7	1.9	2.3
Deformed bar in tension	1.7	1.9	2.2	2.6
Deformed bar in compression	2.1	2.4	2.7	3.2

Table 4. Ultimate anchorage bond stresses (CP110:1972)

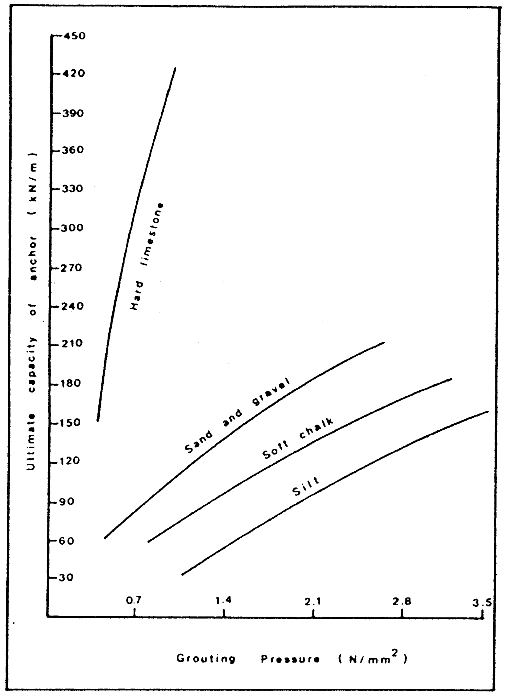


Fig 4. Influence of grouting pressure on ultimate load holding capacity (Littlejohn and Bruce, 1977)

4.3 External Load Carrying Capacity

All load is assumed to be transferred by side shear into the founding strata. Contributions from endbearing or from frictional resistance generated in upper looser horizons are generally discounted. Reference may be made in DD81 "Recommendations for Ground Anchorages (1982)", in respect of the details of parameter quantification.

4.3.1 Rock/Grout Bond

Typically, the bond is assumed to be uniformly distributed over the grout/rock interface, and so the working load, P, may be calculated from

$$P = \frac{\pi DL\ \tau ult}{sf}$$

where D = diameter
L = length
τult = ultimate skin friction
sf = safety factor

The choice of a suitable value for τult is often related to local knowledge and the back analysis of successful applications (Littlejohn and Bruce, 1977). Equally, trusted relationships such as the following can be employed. For strong rock with 100% core recovery,

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

This is valid provided the unconfined compressive strength of the grout is at least $42N/mm^2$.

4.3.2 Grout/Ground Bond

For low pressure (i.e. <10 bar) grouting systems in *cohesionless soils*, DD81 (1982) notes that conventional piling design technology is used:

$$Tf = Ln\tan\phi'$$

where Tf = ultimate load holding capacity
L = bond length
ϕ' = angle of shearing resistance
n = factor that apparently takes into account the drilling technique, depth of overburden, pile diameter, grouting pressure, insitu field and dilation characteristics.

Field experience (Littlejohn 1970) indicates:*

Coarse sands and gravels
($k > 10^{-4}m/s$): n = 400 to 600kN/m.

Fine-medium sands
($k = 10^{-4} - 10^{-6}m/s$): n = 130 to 165kN/m.

Similarly, for *cohesive soils*, (grouted at gravity pressure) undrained shear strengths are used to estimate capacities:

$$Tf = DL\alpha Cu$$

where Tf, and L are as above
Cu = average undrained shear strength over the bond length
α = the adhesion factor
D = pile diameter

It is common to select $\alpha = 0.45$ in conventional large diameter bored piling. However, due to their size, and construction procedures, minipiles are often designed with values of 0.6 to 0.8.

Especially in less competent materials, the magnitude of skin friction can be strongly influenced by the grouting pressure (Fig 4).

In addition, pressure grouting may increase 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 post grouting (Fig 5) an effective pile diameter in the range 300 to 800mm may be expected. In general, pressure grouting is most effective in improving pile capacity in ground 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 to post grouting in stiff clay. No experience of good behaviour in very soft non-consolidated clay or soft peat has been recorded.

4.4 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

*These figures were initially recorded in normally consolidated materials for borehole diameters of about 100mm, and where this diameter varies significantly, n is modified in the same proportion.

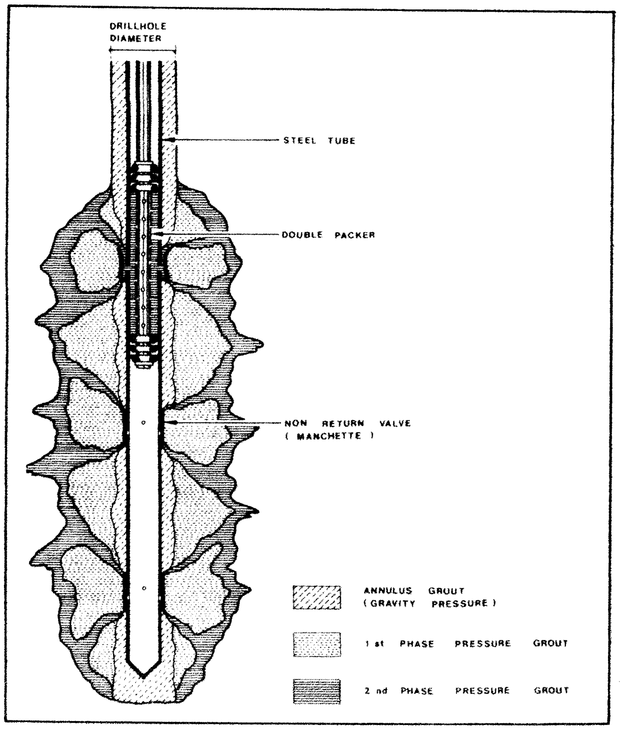
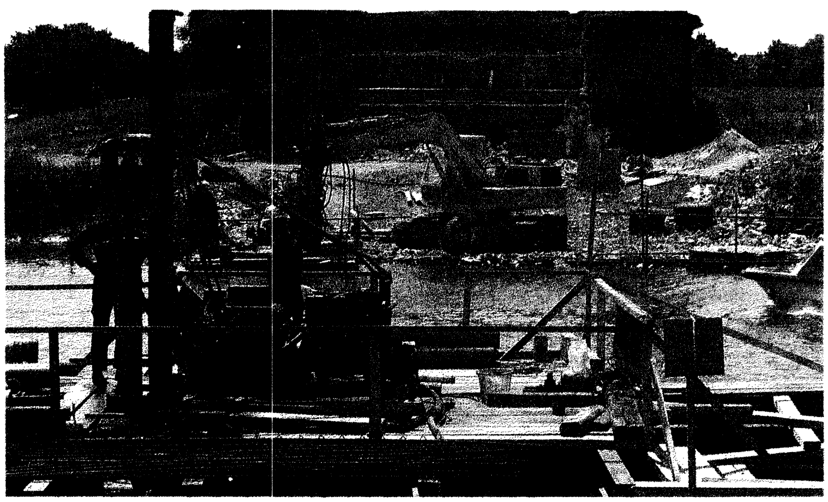


Fig 5. Concept of repeated pressure grouting (Rodio Technical Report)

Photograph 2. Diesel hydraulic tracked drilling rig. Church of St Stephen's, London



Photograph 3. Track and skid mounted drilling rigs at Wilford Toll Bridge, Nottingham

mechanical properties, such as loose silts, peat and non-consolidated clays, is even a possibility of failure through insufficient lateral restrai feasible.

Similarly, bursting can typically be discounted after analyses to BE 21/5/010, and BS 5400 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, increasing the spiral reinforcement or by maintaining a sacrificial casing through the dubious horizons.

4.5 Spacing of minipiles

General considerations often dictate the need for pile groups. The compromise is then between selecting a close spacing, so 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 are to be socketted into rock for appreciable distances.

5. CONSTRUCTION

As noted above, a key factor in the categorisation of minipiles is their mode of construction, featuring the use of powerful but compact drilling and grouting equipment and exploiting a variety of associated techniques.

In essence the following steps are followed (Fig 6):

(i) Drilling (with the insertion of temporary or permanent casing if necessary).
(ii) Installation of reinforcement.
(iii) Filling borehole with grout (may be done prior to (ii) above).
(iv) Extraction of any temporary casing, with or without excess grout pressure. Phase (iv) may then be followed by a later, post grouting operation in certain applications.

Regarding the vital operation of drilling, the key requirement is that the risk of potential damage to the structure or the ground must be minimised. The selection of machine type clearly depends on the diameter and depth of drilling, but manoeuvrability and size are often equally important factors in minipiling works. In this connection, diesel hydraulic crawler-mounted machines have marked advantages, and these are illustrated in Photographs 2-3.

The drilling method is invariably rotary, and so a duplex method (Bruce 1984) is typical in cohesionless soils, augers are common in clays, and down-the-hole hammers are fastest in rock. Water, air or mud flush is used where appropriate or necessary, but care must always be exercised to prevent damage to the surrounding ground due to washing out of finer materials.

The grouting equipment typically 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 (Gourlay and Carson, 1982) be specified, as this permits the production of a much more consistent high quality colloidal grout, at lower w/c 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.

6. CASE HISTORIES

The foregoing is a general statement of minipile technology. In order to illustrate and expand upon certain aspects, six case histories of recent contracts executed by Colcrete Limited are now reviewed. Summary details are provided in Table 5, which also serves to underline the flexibility of minipiling as an applied technique

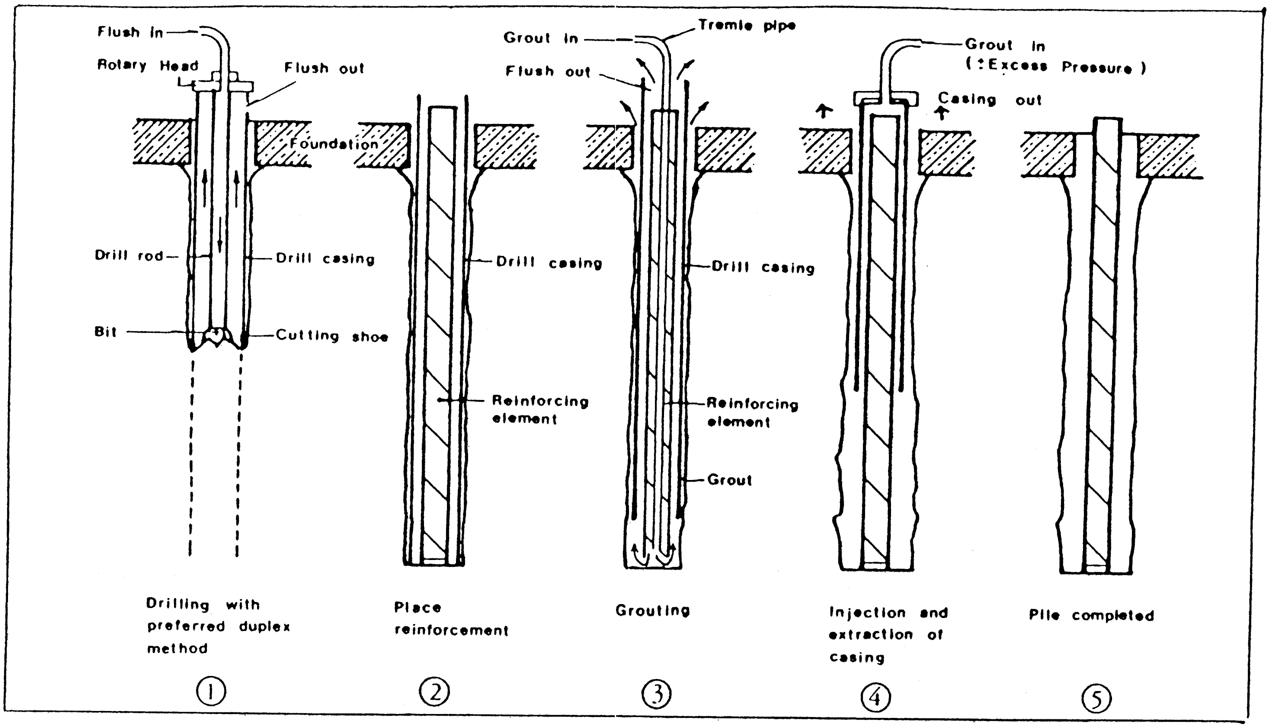


Fig 6. Stages in the construction of a standard minipile (after Koreck, 1978)

Name	Purpose of Minipiles	Ground Conditions	Working Load (kN)	Nr	Total Length Installed (m)	Av. Length (m)	Pile Diameter (mm)	Pile Construction
Church of St. Stephen's Wallbrook, London	To underpin existing church piers and as foundations for new construction	Fill and gravels over London Clay	200	73	1688	23	200	6Y20 bars and 6 mm helical reinforcement. 150 mm pitch. Permanent steel casing in fill. 1:1 sand:SRC grout, w/c = 0.4
Wilford Toll Bridge, Nottingham	To increase the load bearing capacity of pre-existing caisson foundations	Alluvial gravel (+ artificial obstructions) over sandstone	625	28	377	13.5	220	5 Hy 16 bars and 6 mm helical reinforcement 1.5:1 sand:OPC grout WSR = 0.5
Canning Dock, Liverpool	To underpin and support 19th Century dockwall	Masonry/fill/ sandstone	Compression 490. Tension: 700	C1:105 C2:105 T: 93	4120 1674	10.1 18	127 mm through masonry and fill 105 mm in rock	Compression: 50 mm Dividag GEWI bar in 80 mm corrugated pvc duct. Tension: 36 mm dia Dividag bar in 65 mm dia. pvc duct. Neat OPC grout w/c 0.45. Note: The C2 compression piles were extended above required bond length with 30 mm protected bars to "stitch" the masonry wall.
B.R. Bridges, 16 & 17, Romsey	To underpin existing bridge	Masonry/fill/ clay/gravel	Compression 490 Tension 450	29 18	312 335	10.8 18.6	133	50 mm Dyvidag bar in 80 mm corregated PVC duct.
B.R. Bridge, Herne Bay	To transfer upgraded loading on pre-existing bridge footings	Masonry/fill/ London Clay	130	36	504	14	170	32 mm McCall bar in 80 mm pvc duct. 1:1 sand:OPC grout WSR = 0.45
Warehouse at Narrow St., London	To support increased loadings arising from reconstruction	Alluvium over London Clay	190	138	1874	14.5	170	Permanent steel casing in alluvium 4 x Y12 reinforcing bars and 6 mm spiral reinforcement 1.5:1 sand:OPC grout WSR = 0.45

Table 5. Summary details of minipile projects reviewed

and the range of contract sizes resulting. Two contracts (The Church of St Stephen's and Wilford Toll Bridge) are cited to show typical circumstances which conspire to demand a minipile solution, two others (Liverpool and Romsey) illustrate details of design, whilst the last pair (Herne Bay and Narrow Street) provide useful data on piling performance in London Clay.

6.1 Church of St Stephen's Wallbrook, London

The Church, designed by Sir Christopher Wren, stands on a site which has been in use since the third century. Completed in 1679, the structure has suffered from progressive differential settlement over the years. The effects are most noticeable along a line mirroring the original course of the Wallbrook. Poorer ground associated with this watercourse is believed to have caused the 80mm settlement measured for certain piers.

Given the extreme delicacy of the structure, the anticipated highly variable ground conditions, and the very restricted entry/exit and access conditions, minipiling proved the logical solution for its underpinning. Support by grouting had been examined but was rejected on a number of technical grounds including the question of permanence. Minipiling was further favoured since it would allow future extension of the crypt if required.

In addition the equipment used (diesel hydraulic drilling rig (Photograph 2) and electric grouting plant) minimised noise emission – a critical factor, bearing in mind the Church's location in the heart of the City.

The safe working load of 200kN was developed over a 14m penetration into London Clay giving a design safety factor of 2.5 against pile/clay failure. The settlement under design load was anticipated as being less than 5mm. Columns supporting the cupola were underpinned by groups of 6 piles at 950mm centres, whilst the remainder had groups of 4 piles. Six piles were installed to support the altar and a central beam for a new floor slab.

The Krupp DHR80 drilling rig was introduced through a temporary opening approximately 3.2 x 3.4m in one of the walls (Photograph 4). Drilling through the fill, sands and gravels (max. depth 8m) and into the underlying London Clay was conducted with a 200mm diameter auger, which minimised vibration to the structure and prevented removal of fines – both of which would have aggravated the settlement problems. The length of each pile above the clay was permanently lined with a 200mm i.d. 8mm thick casing, to enhance the pile performance through the fill, to minimise grout travel, and to provide a positive cut off against the effects of ground water. Each pile was reinforced as in Table 5 with a 140mm diameter cage. The 1:1 sand:SRC grout gave strengths of about $45N/mm^2$ at 28 days.

Due to the severely restricted working space, load tests could not be conducted on the piles and so each pile was subjected to ultrasonic testing (transient dynamic method) to confirm its integrity.

Numerous obstructions to the progress of the drilling were recorded, principally hard rock inclusions in the fill, and ancient column and wall footings. Most were overcome by predrilling with a 240mm rock roller bit, thereby permitting the installation of the permanent liner and the progress of the subsequent augering. In other cases, where even this approach proved unsuccessful, pile locations were amended slightly, such flexibility being a recurrent feature of minipiling as a practical technique.

6.2 Wilford Toll Bridge, Nottingham

Severe corrosion of the existing lattice cast iron and wrought iron girders had reduced the capacity of the Wilford Toll Bridge over the River Trent in Nottingham to pedestrian traffic only. A major reconstruction programme was sanctioned and the 32.6m centre span and the two 29m side spans of the Victorian structure were removed, to be replaced by a new twin plate girder replacement span. This new deck is founded via a pair of steel box section portals, on concrete piers, in turn bearing on pairs of existing 2.15m o.d. diameter cast iron caissons thought to be set in compact alluvial sands (Fig 7). The revised loading conditions of the new structure necessitated increasing the load bearing capacity of each caisson by 4350kN. A minipile solution was adopted for the following major reasons:

(i) access to the working locations (in mid river: Photograph 3) would be very restrictive in terms of type of equipment viable. In fact uniflotes were used on the North caissons, and a long rubble bund was constructed from the South Bank.
(ii) the nature of the infill material of the caissons was not precisely known and it was held that the flexibility of drilling systems afforded in minipile installation would ensure the successful installation of the piles. In addition, any local variations between foreseen and actual geological conditions could be readily accommodated by minipiling techniques.
(iii) the limiting inside diameter of the caissons would have prevented even a small number of large diameter piles being installed.
(iv) environmental considerations such as river and noise pollution also argued for smaller scale equipment.

Using conservative values for estimated working bond stresses between pile grout and the founding sandstones, and conservatively ignoring the beneficial effect of potential extra bond in the overlying strata, 220mm diameter piles of 625kN working load were designed, penetrating 3m into the sandstone. Thus seven piles per caisson were foreseen, with the average overall lengths of about 12m giving a design safety factor against failure at working load of over 2.5.

However, upon opening each caisson, the granular infilling anticipated was found to be supplemented liberally with lumps of concrete and pieces of timber, whilst the drilling also encountered many major steel obstructions interpreted as being internal bracings. Nevertheless the flexibility of approach afforded by the rotary drilling method, operated off track and

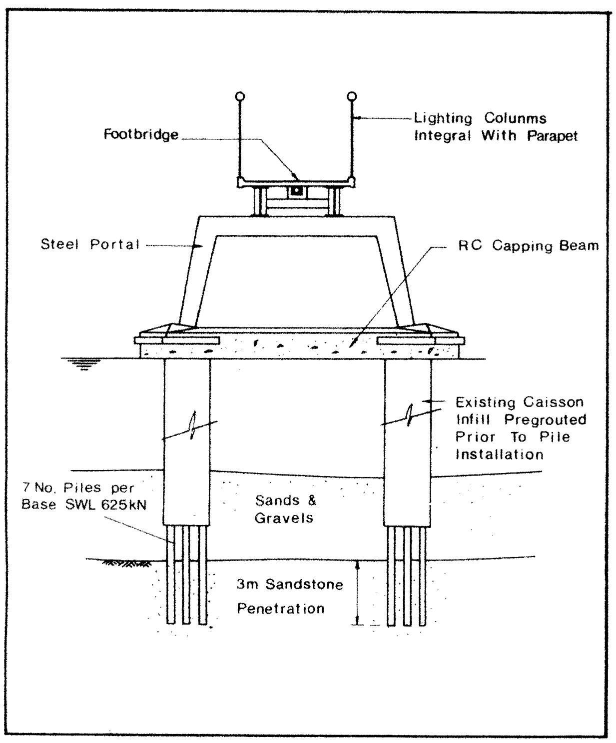
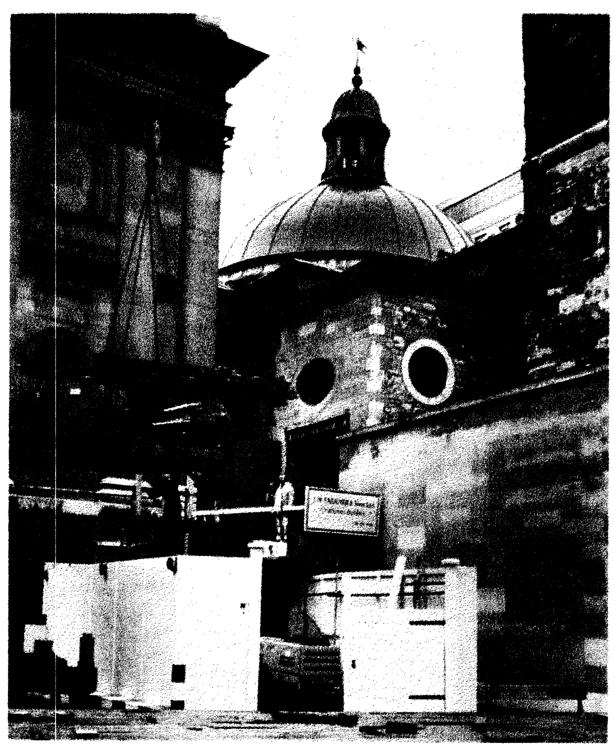


Fig 7. Arrangement of new foundations for Wilford Toll Bridge, Nottingham



Photograph 4. Drilling rig being introduced through temporary access (behind signboard) into Church of St Stephen's, London

skid mounted diesel hydraulic drilling rigs (Photograph 3), allowed all the piles to be installed in the design positions.

Again the fact that the sandstone proved to be harder and deeper (by approx. 1m on the North caissons, and 2m on the South caissons) posed no major problems.

A contemporary "Construction News" records that despite the construction changes necessitated, and the "unexpectedly harsh winter", the whole project was completed to programme.

6.3 Canning Dock, Liverpool

This contract illustrates how a minipile system can be designed to satisfy a complex combination of loading criteria, viz, from above, below, behind and in front of a seawall under different tidal conditions.

The wall itself is a sandstone masonry structure, built in the 19th century upon a timber pile framework founded in sandstone 3-4m below its base. Partial collapse of the wall had occurred, necessitating a temporary rubble embankment for support (Fig 8).

Forces acting on the wall can be summarised as follows:–

Pa, the resultant active pressure acting behind the wall from fill material and hydrostatic pressure (variable).

R, the resultant horizontal resistance provided by water in the dock (variable).

U, the uplift on wall due to hydrostatic pressure (variable).

Ww, the dead weight of the wall.

Wf, the dead weight of the fill.

Ws, uniformly distributed surcharge loading (variable).

Wc, line surcharge loading (variable).

To simplify the drilling operations, it was advisable to install the pairs of compression piles continuously through the wall before penetrating the underlying strata. The need to maintain a minimum "cover" to each borehole of 400mm to prevent blowout dictated the general pattern shown in Figure 8. Practical reasons also fixed the location and inclination of the tension piles, designed to resist overturning and sliding. It was assumed that the wall was intrinsically stiff, and that overturning could be about the outer row of compression piles (C1).

Conventional calculations showed maximum working loads to be:

C_1 = 572.2kN, C_2 = 628.6kN, and T = 703kN per metre run of wall.

Pile spacings were calculated based upon the internal characteristics of the pile selected and the relevant safety factors. Compression piles incorporated 50mm diameter Dividag Gewi bars (ultimate load capacity 980kN) and tension piles used 36mm diameter grade 1080/1280 Dividag bars (ultimate load capacity 1252kN). The system provided minimum factors of safety against overturning of 2.4 and against sliding of 2.0.

Bonded lengths into both the existing wall and the sandstone were designed according to the procedures of Section 4.3.1, and used working bond stresses of 0.5-0.7N/mm².

The installation of the piles featured a number of trackmounted diesel hydraulic drilling rigs, permitting, with equal facility, airflushed down the hole drilling in the competent materials, and water flushed duplex drilling in the softer horizons. An average grout volume of 23.0 litres/metre of pile was recorded, this being almost twice the nominal, and so suggesting the benefits of penetration into and consolidation of, the looser and more fissured strata.

Highly satisfactory performance of the structure has been recorded following removal of the temporary fill support.

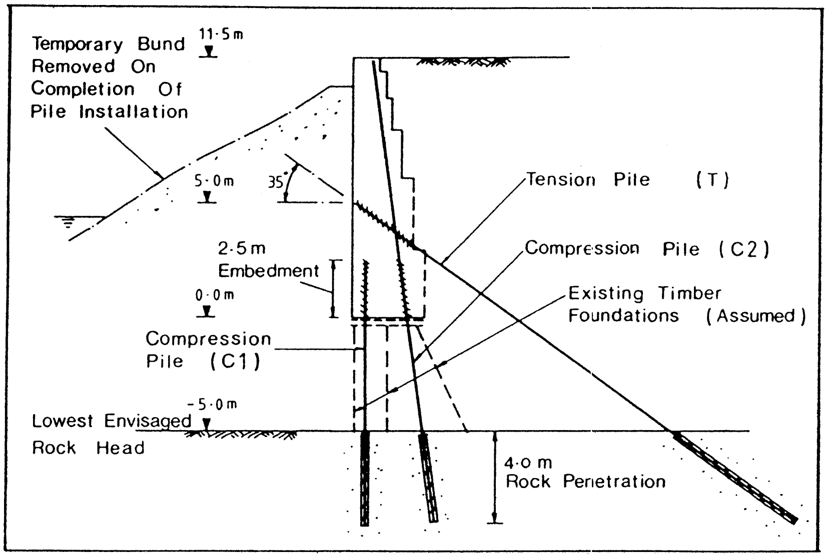


Fig 8. General arrangement of minipiles, Canning Dock, Liverpool

6.4 B.R. Bridges 16 and 17, Romsey

Structural surveys had indicated a requirement for a programme of repairs to Bridges 16 and 17, over Greatbridge Road on the Romsey to Dunbridge line of British Rail's Southern Region. Although some tie bar renewal and installation, grout injection, and brickwork repairs were also involved, the major remedial work consisted of underpinning the abutments of Bridge 17 (Fig 9).

Conventional piling would have been capable of transmitting the vertical loads into the founding dense gravels, but would have involved unacceptable line closures. A minipiling option was, therefore, exercised, the drilling equipment being able to operate in the extremely restricted access conditions under the bridge, further exacerbated by the presence of an adjacent

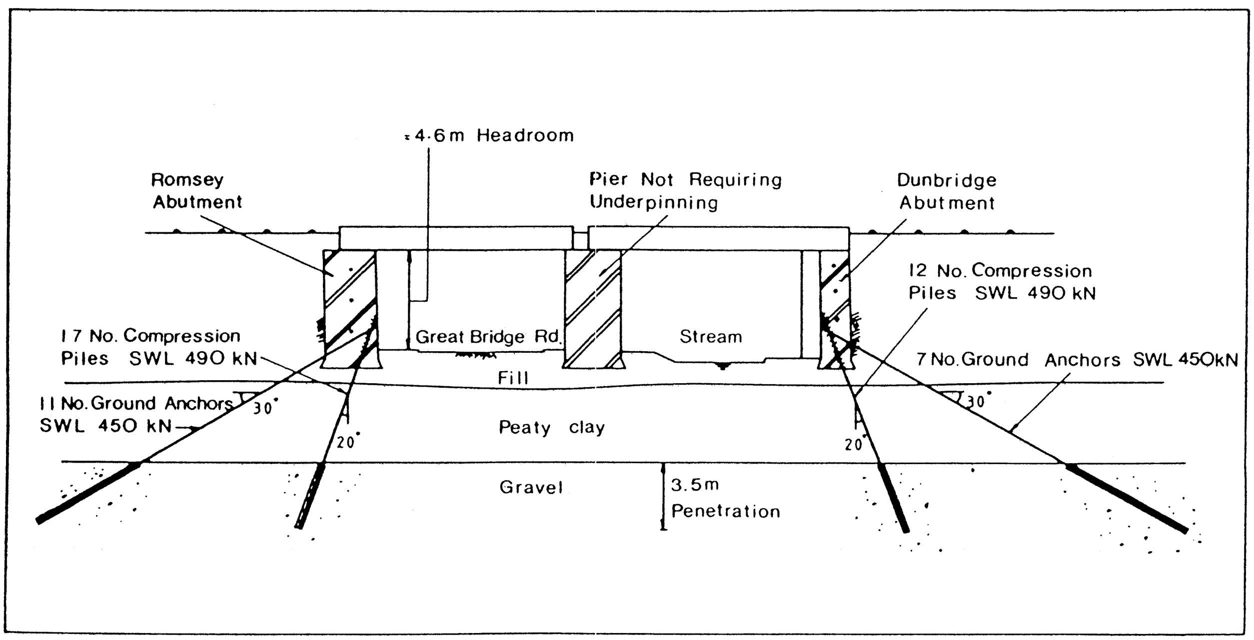
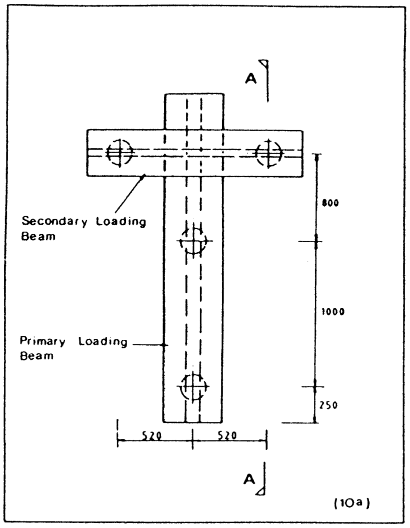


Fig 9. General arrangement of minipiles and anchors, B.R. Bridge 17, Romsey



Plan view of test beams

river, demanding very rigorous environmental controls. Disruption to the underpassing road traffic was also minimised as a result of the size and adaptability of the equipment used.

The loading conditions identified by the Client and the general design concept of Fig 9 indicated compressive loadings of 5753kN and 8288kN and tension loads of 3150kN and 4834kN for the Dunbridge and Romsey abutments respectively.

For the compression piles, 50mm diameter Dividag Gewi bars in 133mm diameter holes were selected. With an ultimate load capacity of 980kN and allowing a factor of safety for the bar of 2.0, 12 piles were therefore required for the Dunbridge Abutment and 17 for the Romsey Abutment.

Bond lengths through the pier were based upon allowable bond stresses in brickwork: a value of $0.6N/mm^2$ gave a 2m embedment for each 133mm diameter pile. The load transfer length (3.5m) in the founding gravels was designed in accordance with Section 4.3.2, featuring values of 35° for ϕ' and 500kN/m for n.

In order to minimise subsequent movement of the abutments in service, the tensile resistance was provided by a prestressed ground anchorage arrangement. Each tendon consisted of 3Nr 15.2mm dia. Dyform strands (ultimate capacity 900kN). The fixed length of each anchorage was increased to 6m, having consideration for their shallower inclination, and the peaty nature of the overlying soils.

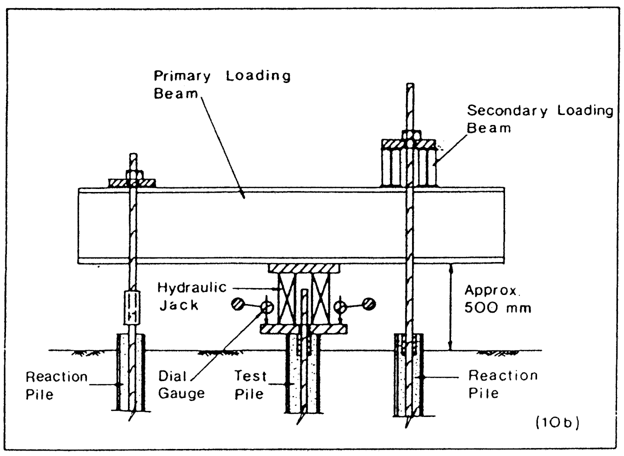
Both the construction phase and the subsequent service performance of the solution were highly satisfactory.

6.5 B.R. Bridge, Herne Bay

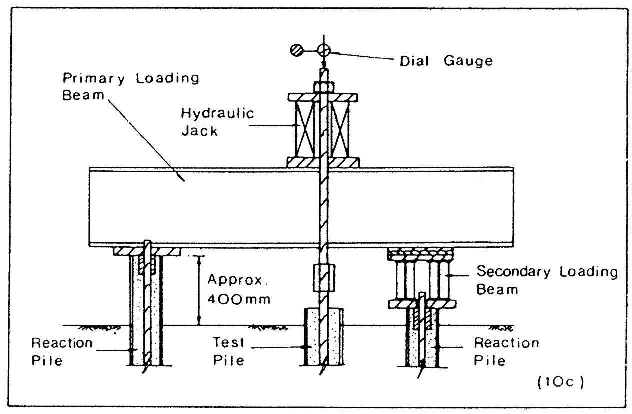
Piling was required to sustain increased compressive, overturning and sliding forces arising from the rebuilding of a road bridge across a railway line. These increased forces were to be transferred from the pre-existing foundations into the underlying London Clay (Table 6).

Thus, for a typical borehole diameter of 170mm, and using $\alpha = 0.6$, an ultimate load of over 316kN was calculated for a pile embedment length of 12.5m. Given the minimum practical inter pile spacing of about 1m, such ultimate pile loadings were consistent with the requirements of the overall design.

Both compression and tension piles were



Elevation of test beams for compression test



Elevation of test beams for tension test

Fig 10. Test beam arrangements for B.R. Bridge, Herne Bay

Depth	Value
0 – 1.65m	Negligible
1.65 – 7.25m	Average = $70kN/m^2$
Below – 7.25m	Average increasing by $16.5kN/m^2$ per metre i.e. Cu = 70+ (D7.25) x $16.5kN/m^2$

Table 6. Undrained cohesion data, London Clay, Herne Bay

Depth	Value
0 – 5m	Negligible
5 – 10m	Average $60kN/m^2$, Ultimate $135kN/m^2$
10 – 16m	Average $90kN/m^2$, Ultimate $200kN/m^2$

Table 8. Undrained cohesion data, London Clay, Narrow Street

Drilled length in clay:	13.5m
Drilled diameter in clay:	170mm
Grout mix design: sand:OPC =	1:1
WSR =	0.45 approx.
Plus 2% Colplus additive (by wt. of cement)	
Average Grout strengths:	4 days – $39.0N/mm^2$
	7 days – $43.5N/mm^2$
	14 days – $48.5N/mm^2$
Reinforcement: 32mm HY bar, characteristic strength = 443kN	

Table 7. Details of test piles, Herne Bay

Cased length in fill and alluvials:	3.0m
Drilled length in clay:	11.0m
Drilled diameter in clay:	170mm
Grout mix design: sand:SRC =	1.5:1
WSR =	0.6 approx.
Average grout strength:	28 days - $35N/mm^2$
Reinforcement:	4 Nr Y12 bars with 6mm spiral reinforcement at 150mm pitch. Characteristic strength = $140N/mm^2$

Table 9. Details of test pile, Narrow Street

involved in the design of the reconstruction works, and one of each type (Table 7) was subjected to testing generally in accordance with CP 2004.

Due to the very restricted access conditions on site, intermediate reaction piles were installed to permit testing by beam and hydraulic jacks, both in compression (Barley 1982) and tension (Fig 10). Each pile was subjected first to a maintained load test in which loads were held for various intervals (from 10 mins to 4 hours) to 1.5 times the working load, and thereafter to a constant rate of penetration test to establish the ultimate load. Results (Fig 11) indicated ultimate loads of 405kN (compression) and 384kN (tension).

Subsequently 36 similar production piles were installed successfully without notable problems then or thereafter.

6.6 Warehouse at Narrow Street, London

The Client planned to construct high quality accommodation facilities inside the shell of a former tea warehouse on the banks of the River Thames in East London. The existing foundations, however, were calculated as being insufficient for the new internal construction, a factor aggravated by the discovery of up to 6.5m of soft black alluvial materials in certain parts of the site. The site was extremely congested and so minipiles were selected to transfer the new loads to the underlying London Clay (Table 8). A further reason for selecting minipiling was the need to minimise possible disturbance to a sensitive party wall.

The site investigation had revealed the presence of artesian arenaceous Greensand Beds, underlying the London Clay: the maximum pile depth possible, therefore, was 17m, which allowed a typical maximum safe penetration into the clay of 11m.

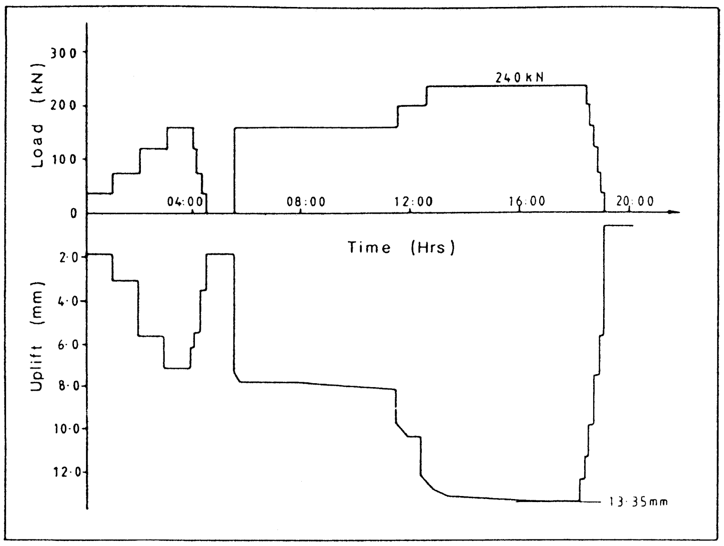
It was foreseen that an auger method of drilling would be adopted to minimise disturbance to both alluvium and clay, and so a nominal pile diameter of 170mm was adopted. A value of $\alpha = 0.6$ was selected, based on previous local experience, and so an ultimate theoretical load of 600kN was calculated, compared to the required working load of 190kN. In order to further minimise disturbance to the alluvium and to alleviate any prospect of pile failure by buckling or bursting in this soft horizon, a permanent steel casing was installed to the top of the Clay.

A preliminary test pile (Table 9) was installed and tested to twice working load. Tension piles were used as the reaction as the lack of space preempted kentledge.

The test results are presented in Figure 12 and on the basis of these data, the design was verified and the construction proceeded. By the conclusion of the contract a total of 138 piles had been installed satisfactorily.

CONCLUDING REMARKS

With particular reference to six recently executed projects in England, the paper underlines the major features which make underpinning by minipiles an increasingly popular and powerful technique. These major advantages include (i) the use of relatively small, clean and quiet equipment, (ii) the exploitation of a wide range of modern drilling and grouting techniques to combat all ground types, (iii) their ability to be installed from very restricted access locations, and in any direction, with the minimum of structural disturbance, (iv) the flexibility afforded to designers to solve complex problems,



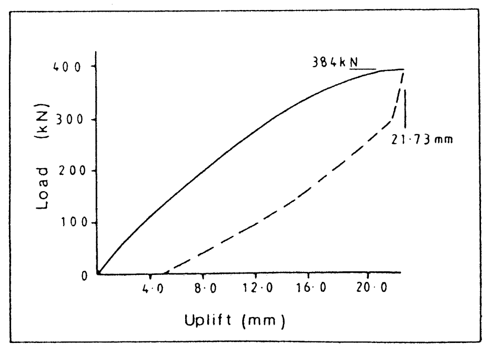
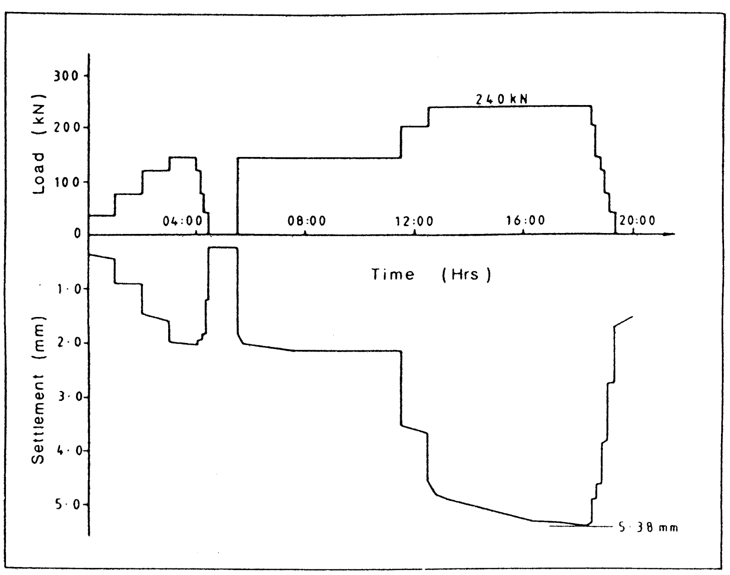


Fig 11(a). Tension pile test data, Herne Bay



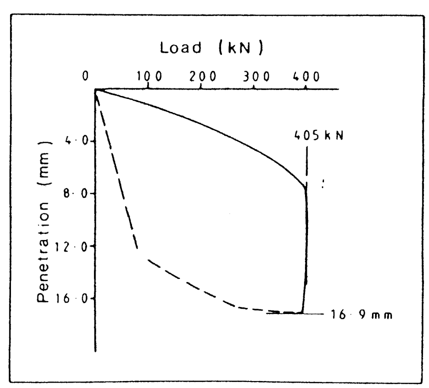


Fig 11(b). Compression pile test data, Herne Bay

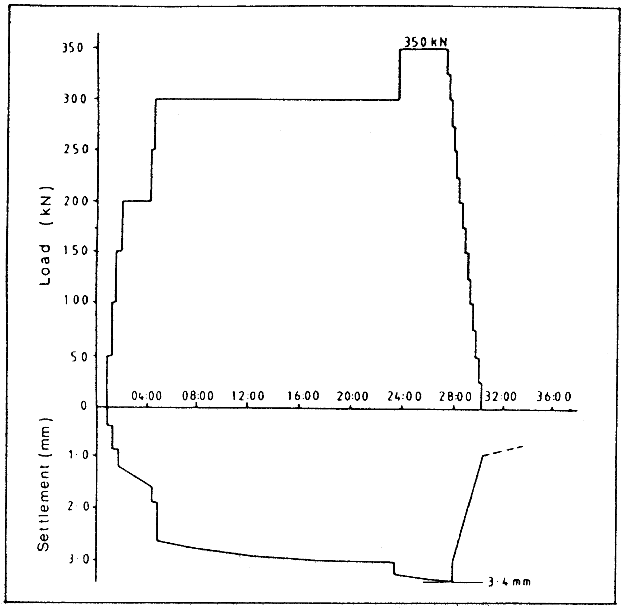


Fig 12(a). Load/time/settlement data, Narrow Street

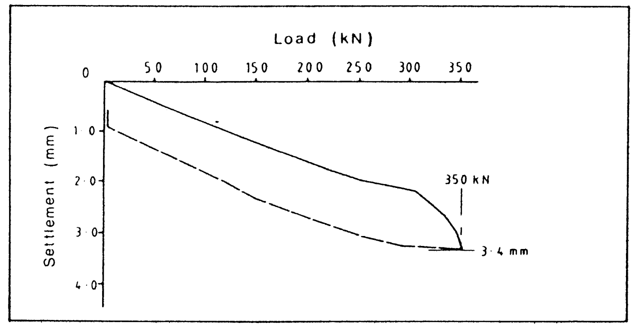


Fig 12(b). Load settlement data, Narrow Street

and (v) the excellent, demonstrable performance behaviour. Given the current trend towards expenditure for rehabilitation of existing structures as opposed to funding of new works, the potential for the further development of minipile technology in Britain appears very real.

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