A field construction and load test trial of high capacity mini piles

Epreuve d'essai de construction sur le champ et de chargement de mini-pieux à haute capacité

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ABSTRACT: Mini piles were selected as a possible means of providing foundations to a major structure to be built as an envelope to an existing sensitive building on a site in Cumbria U.K. There were two principal concerns with the first being the viability of pile construction in the difficult ground conditions at the same time as operating piling plant in a restricted space immediately adjacent to an existing sensitive structure. The second concern was in respect of the load-displacement performance of mini piles. A trial construction and load test programme was therefore carried out. The paper summarises the findings of the construction aspects of the mini piles and presents the results from the load tests.

RÉSUMÉ: Les mini-pieux ont été sélectionnés comme un moyen possible de procurer des fondations pour une structure importante à construire pour envelopper un bâtiment sensible sur un site en Cumbria, au Royaume-Uni. Il existait deux soucis principaux pour l'utilisation des pieux. Le premier souci était la viabilité d'une construction utilisant des pieux dans des conditions de sol difficiles, ainsi que la possibilité d'utiliser un matériel de battage de pieu dans un espace réduit immédiatement adjacent à une structure existante sensible. Le deuxième souci concernait la performance de déplacement en charge des mini-pieux. Une construction d'essai et un programme d'essais en charge ont donc été entrepris. Le document résume les conclusions concernant les aspects de la construction des mini-pieux, et présente les résultats des essais de charge.

1 INTRODUCTION

Some forty to fifty years ago a wide variety of structures were built at the Sellafield Nuclear Complex, Cumbria, in the north-west of England, UK. Certain buildings are now being prepared for decommissioning prior to their eventual removal.

This paper concerns a proposed new structure which is to form an envelope to an existing building that is to be decommissioned. Other buildings in the general area restrict the space on the ground that is available for the new structure. It was a requirement that disturbance to the existing building, such as settlement induced by the new structure, was kept to a minimum. Accordingly piles were one of the favoured foundation solutions.

2 SUMMARY GROUND CONDITIONS

The ground conditions consist of about 6m of variable, but predominantly granular, made ground. This overlies fluvioglacial and glacial deposits, again predominantly granular, with sandstone rock at or below 25m depth. The granular fluvioglacial deposits include layers of dense cobbles and boulders of volcanically derived very strong rock. The water table is just below the base of the made ground. A plot of the SPT profile is presented on Fig. 1.

3 DESIGN CONSIDERATIONS

Severe limitations on the access to the site and the confined conditions within the site preclude the use of large piling rigs. As a result, attention was focused on small diameter piles that could be installed with modest sized rigs.

The provisional loads from the proposed structure included both static and seismic conditions. The piles would therefore have to be able to act in compression and tension as well as bending. In addition, the piles had to resist indirect vertical and tractional seismic earth pressure loading from adjacent shallow foundations and also kinematic ground loading.

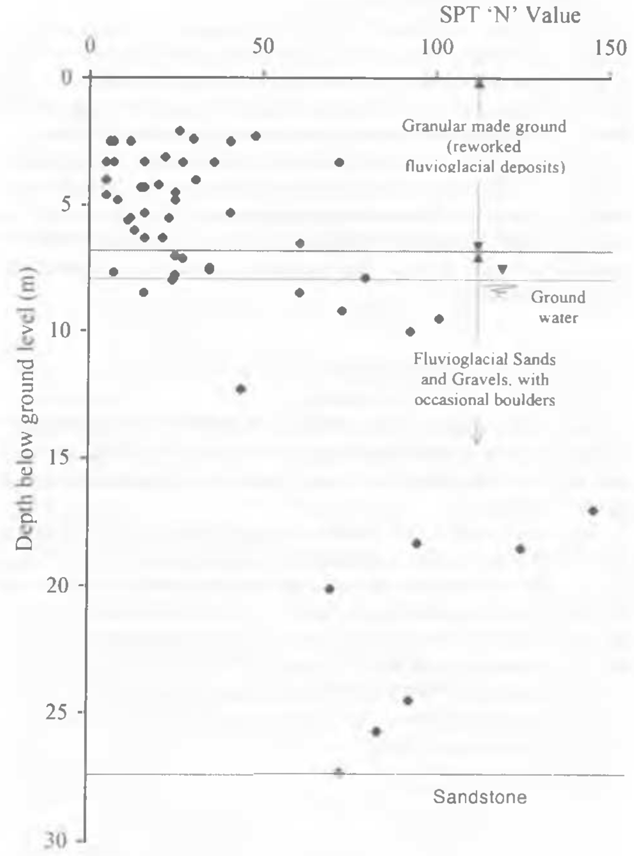

Figure 1 Summary of Ground Profile

To enable the piles to carry these loading conditions, they were designed as composite elements consisting of a grout filled circular hollow section (CHS) with a central bar reinforcement, Fig. 2.

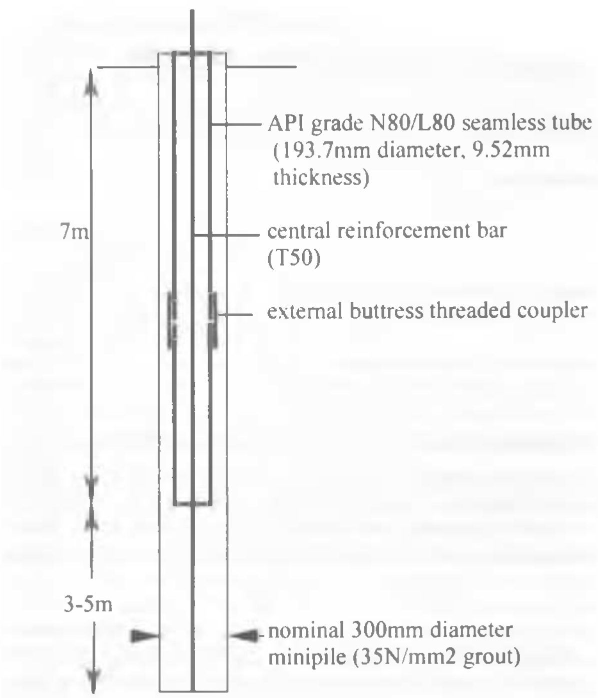

Figure 2. Structural Detail of Mini pile

A CHS section of high grade steel (550 N/mm2) was selected to minimise the necessary cross-sectional area of the reinforcement. The CHS would be required over the uppermost 7m of the pile and be combined with a central 50mm diameter high yield reinforcing bar over the full length of the pile. Sand-cement grout of 35N/mm2 strength was selected to infill the pile boring with the grout including a superplasticiser and a retardant. The grout was planned to be installed under tremie with the lower part of the grouted length being pressurised to about 4 to 6 bar.

Piles of 300mm nominal diameter were selected and planned to be about 12m long to accommodate loads in the range 300 to 600kN. There was marked uncertainty concerning the ability to construct such piles in the difficult ground conditions and within areas of restricted access. This uncertainty could be mitigated by trial pile installation.

4 GEOTECHNICAL DESIGN

The load capacity of the 300mm diameter pile was initially assessed in the normal manner by evaluating the shaft and end bearing carrying capacities on the basis of conventional bored pile construction.

The initial design was based on an assessment of the relative density of the ground conditions as given by SPT 'N' values, Fig. 1. The design line was checked by comparing it to data on the shear wave profile against depth. The generalised relationship of $V_s = aN^b$ (Ohsaki & Iwasaki 1973) was adopted. From previous studies at Sellafield, values of 51.0 for 'a' and 0.4 for 'b' were selected for the predominantly granular material.

The shaft friction was evaluated using the conventional expression (Tomlinson 1995):

$$Q_{s(ult)} = K_s p_o' A_s \tan\delta \qquad (1)$$

where

$Q_{s(ult)}$	is the ultimate shaft friction for unit length of pile
K_s	is the coefficient of horizontal soil stress
p_o'	is the vertical effective stress at depth considered
A_s	is the area of shaft/unit length
δ	is the angle of soil/pile friction

Allowing for a modest influence of the grouting pressure the value of K_s/K_0 was taken as 1.5 with K_0 being 0.45 for medium dense material and 0.35 for dense material. The value of δ was taken as equal to the friction angle of the soil, ϕ', and considered to range from 32° to 40°, the values increasing with increasing depth. On this basis, for a 12m long 300mm diameter pile the ultimate shaft friction was calculated to be about 340kN. The average shaft friction was about 27kPa and ranged from zero to 66kPa.

The ultimate base capacity of a 12m long pile was determined by allowing for a 10 per cent enhancement in the base diameter and adopting an ultimate base resistance of 10MPa (Tomlinson 1995). It was evaluated at about 860kN. This gave an ultimate total load capacity of about 1260kN. It was recognised, however, that for pressure grouted piles the shaft friction could be markedly higher, possibly 4 to 8 times the value of a gravity grouted pile.

Published information on typical shaft friction capacities for pressure grouted piles advised that they could be used but only with great caution (Bruce and Juran , 1997). Also there was a possibility that a pile could be end bearing on a boulder, thereby giving an enhanced effective size to the end of the pile. Such uncertainties in the assessment of pile-soil strength characteristics are furthered strengthened by the variability in the measured SPT 'N' values as exemplified by Fig.1. Accordingly, in addition to resolving the uncertainty with respect to the construction of the piles, there would be considerable benefit in undertaking a trial piling contract with load testing of the trial piles.

5 TRIAL PILE INSTALLATION

5.1 *General*

Two piles of 300mm nominal diameter were chosen initially to be installed for load testing. The test loads were to be applied by reaction against anchor piles. This allowed various techniques for the drilling and grouting of the piles to be attempted. Once the optimum construction method was established, it was decided to construct three other piles of 300mm diameter to demonstrate the viability of the technique and to confirm the likely programme time required to form a pile. One of these 'demonstration' piles was then to be load tested thereby forming the third test pile.

The actual working piles would be constructed in areas where there were severe headroom restrictions. Accordingly the CHS and bar reinforcement were to be in lengths not exceeding 4m and coupled together while they were being lowered into the pile bore. Coupling of the CHS was to be achieved with threaded external sleeve couplers.

5.2 *Drilling*

A Casagrande M9 rig was selected for drilling the pile bores. This is a compact rig suited to the site access restrictions and had the anticipated capability of being able to overcome the difficult drilling through the cobbles and boulders. A range of different drilling systems and techniques were attempted all with water flush as other flush mediums were excluded for environmental reasons. Problems with the drilling included the fracturing of drill rods, excessive wear of drill bits, high levels of vibration and failure of components of the drilling rig.

The rotary drilling method gave the highest average overall rate of drilling at 2.6m per hour. The technique broke down the material being drilled through to fine sand sized particles which remained in suspension in the flush water and could be returned to the surface with modest flush volumes. However, the drill rods incurred breakages and the rig experienced high levels of mechanical vibration and fatigue. Accordingly high plant down time would be expected for continuous operations under works conditions. As a result other drilling methods were preferred even though they had a somewhat slower drilling rate.

Test pile 1 was drilled to a depth of 12.4m but in test pile 2 difficult ground conditions were experienced particularly below about 10m. The boring was therefore termined at about 11.1m.

The method which was eventually adopted involved rotary percussive drilling with a full face drill bit. Temporary casing was installed with rotary action and a sacrificial casing shoe. This method was utilised for the demonstration piles resulting in a drilling rate of about 1.3m per hour. The gravel sized drill debris was relatively difficult to recover in the flush water and large water volumes were required particularly when drilling through cobbles and boulders close to the base of the pile.

This problem was mitigated by modifying the specification to allow the piles to be drilled to a minimum depth of 10m. Drilling then continued for up to 3 hours or until 12m overall depth was reached, if this were achieved in less than 3 hours. The overall time for the construction of each of the demonstration piles was between about 10 and 11 hours. For the demonstration pile that became test pile 3 a depth of 10m was achieved.

The water flush volume varied widely from one pile to the next. It was monitored carefully for the demonstration piles and ranged from 18 cu.m to 110 cu.m per pile on the basis of full recirculation of returned flush. If recirculation had not been employed, about twice this volume of flush water would have been required.

5.3 *Grouting and reinforcement*

The first five anchor piles of 300mm diameter were grouted by gravity injection. The subsequent five piles and the two test piles were pressure grouted over their lower length with 6 to 8 bar pressure. Table 1 summarises the grout take relative to the nominal 300mm diameter of the piles and the equivalent diameter of the piles. This has been evaluated on the basis that the grout take gives a uniform increase in the pile diameter. In practice there would be a greater increase over the lower section of the pile where the pressure was applied and a lesser increase over the upper length where the pile was gravity grouted.

Test pile 1 had a grout take of 250 per cent of the nominal diameter of the pile, the equivalent diameter therefore being 470mm. Test pile 2 had a grout take of 320 per cent giving an equivalent pile diameter of 535mm.

The central reinforcement bar and the CHS, typically of 244mm diameter, were installed in the pile after the grout had been placed. However difficulties were frequently experienced in installing the 244mm diameter CHS to the intended depth in the pressure grouted bore. Installation using a 168mm diameter CHS was attempted to determine whether this could be installed more easily in the pre-grouted pile bore. Little improvement was found with the main difficulty arising at the point where the buttress coupling, being of larger diameter than the parent CHS section, entered the grouted hole.

In an endeavour to overcome this problem, for the demonstration piles a CHS of 193mm diameter was selected. This size was of sufficiently small diameter to allow it to be inserted inside the temporary casing before pressure grouting took place. For the demonstration piles the grout pressure had to be limited to between 2 and 4 bar to avoid leakage of grout around the outside of the temporary casing. The range of grout takes for this pressure is included in table 1. The demonstration pile which became test pile 3 had a grout take of about 300 per cent giving an equivalent diameter of 520mm.

Table 1 Grout take details

grout pressure	range of grout take (all piles)	average grout take	equivalent pile dia.
Gravity	160% - 250%	200%	420mm
2 - 4 bar	220% - 300%	250%	470mm
6 - 8 bar	250% - 390%	310%	530mm

6 TRIAL PILE TESTING

The three 300mm diameter piles which were subjected to load testing had load cycles applied at intermediate points up to the maximum test loads. The intention was to load the piles in compression up to 1800kN, being three times their nominal design load, and in tension up to 800kN. The compression load-deflection performance of the piles is shown on Fig. 3 and summarised in Table 2 (compression test) and Table 3 (tension tests).

In respect of test pile 3, the pile cap began to rotate when the load reached about 1100kN. Although the pile was unloaded and the loading jack repositioned, further rotation began to occur at 1700kN and therefore the test was halted.

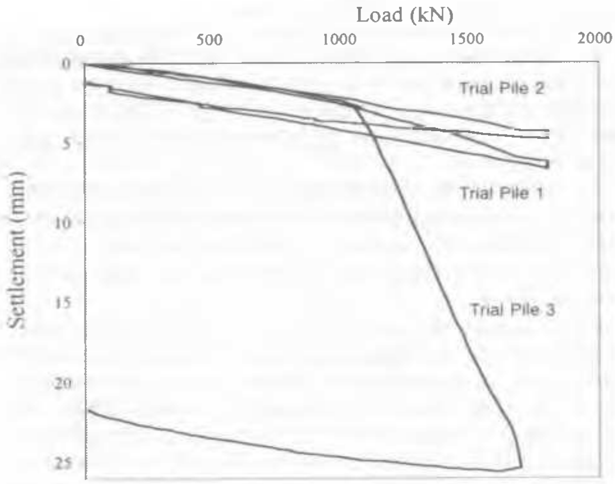

Figure 3 Load-deflection curves from compression tests

Table 2 300mm diameter piles – Compression tests

Displacement (% pile diameter and mm)	Compression test load (kN) pile 1	pile 2	pile 3
at 0.5% i.e. 1.5mm	680	750	700
at 1.0% i.e.3.0mm	1100	1300	1050
max. test load	1800	1800	1700
displacement at max. load (mm)	6.7	5.8	25.0
recovery (mm) on unloading	1.2	1.3	22.0

Table 3 300mm diameter piles – Tension tests

Displacement (% pile diameter and mm)	Tension test load (kN) pile 1	pile 2	pile 3
at 0.5% i.e. 1.5mm	540	500	600
at 1.0% i.e.3.0mm	800	700	>800
max. test load	800	800	800
displacement at max. load (mm)	3.0	3.6	2.2
recovery (mm) on unloading	0.9	1.2	0.5

7 DISCUSSION

Regarding the construction of the piles, the trial piling programme succeeded in demonstrating that 300mm piles could be

Table 4 Pile details and maximum test shaft/base loads

Pile Ref.	Drilled Length (m)	Pressure Grouted length (m)	Grout Pressure (bar)	Grout take (%)	Shaft Load (kN)	End bearing (kN)
TP1	12.4	6.4	6-8	250%	1120	680
TP2	11.1	3.5	6-8	320%	840	960
TP3	10.0	4.5	2-4	300%	850	850

constructed satisfactorily in the difficult ground conditions which included cobbles and boulders of strong to very strong rock. It established that piles of about 10m to 12m length could be formed in one working day. This included the installation of CHS reinforcement and pressure grouting. An average drilling rate of up to about 1.5m per hour could be relied upon but above this there was increased risk of equipment damage.

The need to couple short lengths of CHS because of the confined headroom led to decision that it was preferable to install the CHS before grouting took place rather than afterwards. Installation of the CHS after grouting could result in only part of the full required length being inserted with refusal likely to be met at the point where the coupling entered the grout.

The combination of compression and tension test results for test pile 3, with this pile reaching failure in compression, allows an evaluation to be made of the apportionment between shaft and end bearing resistances. Furthermore, whilst test piles 1 and 2 did not reach failure and performed significantly better than trial pile 3, a consistent understanding of the performance of the piles can be obtained.

For all the test piles at small displacements the end bearing capacity increased reasonably linearly with increasing displacement. Adopting this approach the shaft and end bearing mobilised have been evaluated at the maximum test loads and are given in Table 4.

By comparing the performance of test pile 1 and 2 under compression load and under tension load, it can be seen that the shaft frictions were close to their ultimate values. For test pile 1 the ultimate shaft capacity is estimated to be about 1200kN and for test pile 2 it is about 900kN. These are slightly higher than the value of 850kN determined as the ultimate shaft friction for test pile 3.

On the basis of the nominal pile diameter of 300mm the results equate to ultimate shaft friction values, averaged over the length of the pile, of between 86 and 103kN/m2 with the mean being 93kN/m2. This is about 3.5 times the average value adopted in the original geotechnical design. This increase in capacity remains less than the range of values that would be expected for a pressure grouted pile (Bruce & Juran 1997).

However, the actual length of pile which was pressure grouted varied from about one third to about half of the pile length. The pile with the shortest pressure grouted length, TP2, had the lowest ultimate shaft friction per unit length of pile and TP1, which had the greatest pressure grouted length had the highest overall shaft friction. The gravity grouted section of the piles may, conservatively, have an ultimate shaft friction that is no higher than the original design value of of 27 kPa. On the basis of this assumption, the average ultimate shaft friction for the pressure grouted length of TP3 is about 170kPa, the pressure being between 2 and 4 bar. For TP1 and TP2 where a pressure of 6 to 8 bar was used, the ultimate shaft friction is about 195kPa. These are considered to be the most optimistic values that could be interpreted from the pile test results. They equate to increases in the equivalent gravity grouted frictional resistance by a factor of 3.5 to 4.5 over the lower length of the pile. These increases are consistent with the lower bound pressure grout improvement factors reported by Bruce & Juran (1997). They justify the initial cautionary load capacity assessment and demonstrate the benefit gained by trial installation and test loading of piles

For test pile 3 the mobilised end bearing equates to 12MPa based on a nominal 300mm diameter base, or 4MPa based on the enhanced diameter (Table 1). The pile was at a deflection of close to 10 per cent of its nominal diameter and therefore these end bearing values are considered to be ultimate capacities.

Extrapolation of the base load-deflection response to higher deflections allows an estimate of the ultimate end bearing that would be mobilised at approximately 10% of the nominal pile diameter. On this basis the ultimate end bearing capacities for test piles 1 and 2 have been estimated to be 1500 and 2000kN, respectively. These end bearing loads equate to ultimate pressures of between 21MPa and 28MPa based on nominal 300mm diameter piles, and 8.6MPa and 8.9MPa based on the enhanced base diameters.

8 CONCLUSIONS

1. The trial installation of mini piles demonstrated that they could be constructed in the difficult ground conditions which included cobbles and boulders of very strong rock. However limitations had to be placed on the length of pile in order to avoid undue risk of damage to drilling plant and to allow a pile to be completed within one working day.
2. The insertion of a coupled CHS in a pressure grouted shaft was found to be essentially impractical. The alternative of grouting after installation of the coupled CHS was a feasible approach.
3. Shaft friction capacity was found to be markedly enhanced by utilising pressure grouting. The average ultimate shaft friction values over the pressure grouted length of the pile were evaluated to be in the range of 150 to 200 kPa. This information allowed more optimistic values to be adopted in the final design of the piles. It demonstrated the benefit of carrying out trial pile construction and testing.
4. The ultimate end bearing capacity was found to be variable. For one pile, it compared closely with the original design and there appeared to be little enhancement of the base size as a result of the pressure grouting. For two other piles, the capacity was found to be markedly higher. This could either be the result of the base size being enhanced by pressure grouting or by the piles end bearing on cobbles and boulders.

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REFERENCES

Bruce, D.A. & Juran, I. 1997. *Drilled and Grouted Micropiles. State-of-Practice Review.* Volume 2. U.S. Department of Transportation.

Ohsaki, Y. & Iwasaki, R. 1973. On Dyanmic Shear Moduli and Poisson's Ratios of Soil Deposits. *Soils and Foundations.* Volume 13 number 4.

Tomlinson, M. J. 1995. *Pile Design and Construction Practice. 4th Edition.* Spon