FACTORS AFFECTING THE DESIGN & CONSTRUCTION OF HIGH CAPACITY MINIPILES

D EGAN
A L BELL
Keller Ground Engineering, Oxford Road, Ryton-on-Dunsmore, Coventry, CV8 3EG.
M R GAVINS
Systems Geotechnique Fondedile, Endeavour House, Abbotsfield, St Helens, Merseyside, WA9 4HU.

SUMMARY: Very high load capacity can be achieved in drilled minipiles, often higher than is predicted by traditional design methods used for large diameter piles. This implies that minipiles can be more efficient than larger piles in mobilising their geotechnical capacity. Data obtained from numerous minipile pile tests are tabled and form the basis of the paper. Key factors which influence capacity in minipiles are isolated and discussed and can be categorised as either construction related or soil-structure interaction effects. Recommendations are made regarding design and installation methods appropriate for high capacity minipiles.
Keywords: Minipile, pile testing, geotechnical capacity, drilling methods

INTRODUCTION

Innovative developments in drilling and grouting techniques have spawned a large variety of different minipiling systems. Such techniques have permitted the development of high bond stresses between the pile and the ground. Working capacities of up to 1200kN are common. Minipiles can be installed using small lightweight rigs suited for normal or restricted access and low headroom situations, so providing a solution when larger piles are not possible.

For this paper drilled minipiles are defined as piles with a diameter of less than 300mm (which is in accordance with BSEN 14199:2005[1] the execution standard for Micropiles) formed by rotary, auger or percussive methods. Top or bottom driven preformed or driven cast in situ piles are not considered.

Minipiles although of relatively small diameter are able to sustain relatively high loads, often much higher than predicted by traditional large diameter pile design methods. Through the consideration of a database of over 100 minipile tests undertaken for a wide range of different projects in different ground conditions throughout the United Kingdom and Ireland, factors affecting the design, construction and performance of drilled minipiles are considered. The data represent over 20 years of practical field experience in the installation and testing of minipiles undertaken by three different companies (Keller Ground Engineering, Systems Geotechnique and Fondedile).

To exploit high geotechnical load capacity minipiles require a high strength grout body incorporating appropriate reinforcing steel to enable transfer of the applied loads to the ground. Grout design strengths of 40 N/mm^2 to 50 N/mm^2 can be achieved with a 1:1 sand cement mix and water/cement ratios of 0.4 to 0.45. Concrete may also be used in the construction of minipiles at the larger end of the diameter spectrum. All of the test results presented in this paper were from minipiles constructed using grout. The design of the composite steel and grout section is undertaken assuming the pile is a short braced axially loaded column in accordance with BS8110[2] or BS5400[3] (which offer advantages over the simplified limiting shaft stress approach set out in BS8004[4]). Minipiles with high lateral load capacity may also be constructed, utilising steel circular hollow sections giving enhanced moment capacity.

DRILLING SYSTEMS

The influence of the installation process arising from the drilling system adopted is a key factor in minipile capacity. Table 1 summarises commonly available drilling systems and media used for the construction of minipiles. This summary is not exhaustive as many variants of the generic systems have been developed throughout the world and been in vogue at different times. The requirements of creating a stable pile bore with minimal disturbance or weakening of the host ground remain guiding principles which, with careful and experienced workmanship, enable the construction of durable elements of high structural integrity.

The selection of the drilling system for any project is guided by a number of aspects including the prevailing ground conditions, access constraints (which may influence the type rig used), choice of flushing medium, ease with which arisings can be handled and disposed of and environmental constraints.

The prevailing ground conditions may also dictate whether cased or un-cased holes are required. Temporary casing may be used to keep the borehole open when advancing through collapsing ground. Open hole drilling can be adopted where the pile bore will remain self supporting during the drilling and grouting process.

THE MINIPILE TEST DATA

The results archive of over 100 standard working and preliminary load tests undertaken on minipiles installed by different construction methods in a wide range of soils and rock has been reviewed. Table 2 summarises the size range, working and test loads encompassed by the data.

Table 1. *Methods of forming minipiles.*

Generic Minipile Method	Flush Options			Comments
	None	Air	Water	
Auger	Y	N	N	Augers removed prior to grouting. Casing essential through collapsing soils
Hollow Stem Auger	Y	N	N	Grouting usually undertaken through the base of the central hollow stem. Can be used uncased through collapsing soils.
Rotary Percussive	N	Y	Y	Usually with top drive percussion. May use counter rotating duplex casing and drill rods where cased hole required.
Rotary e.g. Rock roller	N	Y	Y	May use counter rotating duplex casing and drill rods where cased hole required.
Down the hole Hammer	N	Y	Y	Air hammers are most encountered although water hammers are available. May be used with temporary or permanent casing.
Simultaneously Cased Drilling e.g. Odex or Symmetrix	N	Y	Y	Can use temporary or permanent of full depth casing if required.

Key: Y= Yes, N=No

All tests were carried out as part of regular minipiling contracts and archived data reflects the practical application of the minipiling technique undertaken in accordance with standard industry practice. Three aspects of the behaviour of minipiles are considered in light of the test data.

- The load displacement response of minipiles in soil and rock
- The value of adhesion factor, α , for minipiles in cohesive soil
- The value of shaft friction generated by minipiles in cohesionless soil and rock.

Fig. 1 shows the normalised displacement of the minipile head plotted against normalised pile load for all test data. The normalised displacement is defined as the pile head settlement divided by the nominal pile diameter. The normalised load is the applied load divided by the Design Verification Load (DVL). The trend line can be considered as a possible upper bound.

Table 2. *Range of minipile size and working loads analysed.*

Nominal Diameter (mm)	SWL (kN)	Maximum Test Load (kN)	Bond Length (m)	Total Length (m)
133	400-450	1100	1.5-3.0	11.0-19.0
140	250-480	625-1200	1.0-5.0	9.5-20.5
170	400-525	600-1300	3.0-25.0	6.0-30.0
220	280-600	420-1800	2.0-29.0	6.0-31.0
300	300-1050	960-1575	3.0-9.0	5.0-14.5

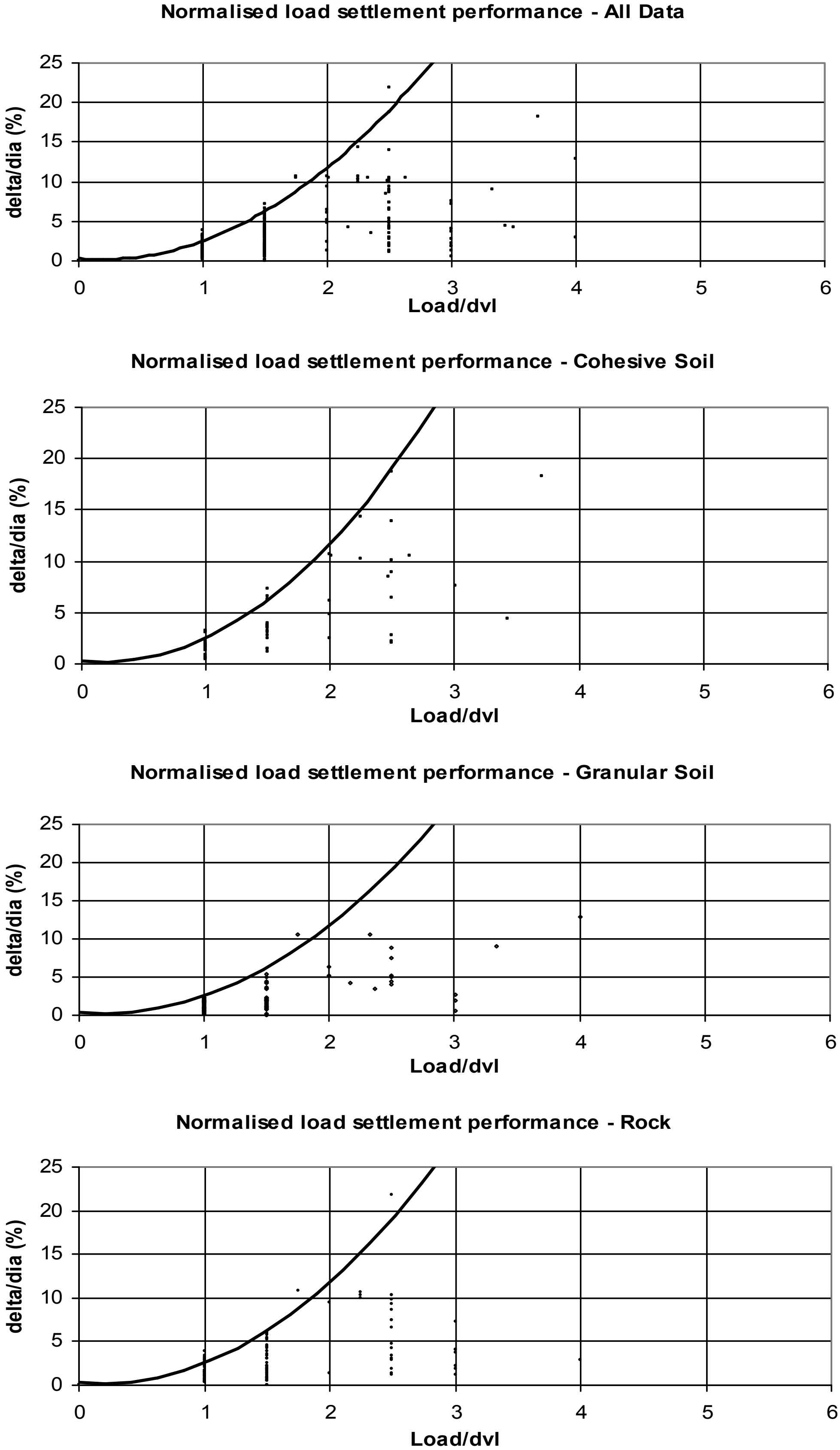

Fig. 1: *Normalised load settlement response.*

Taking all available data statistical treatment yields the mean settlement and standard deviation at varying loads and for piles in different strata as shown in Table 3. The data show (with 95% confidence based on evaluation of the whole data set) that minipile settlement will not exceed about 6mm and 12mm at DVL and DVL+50%SWL respectively, which is usually well within acceptable total and differential settlement limits for most structures. Minipiles in cohesionless soils behave more stiffly than those in cohesive ground. The mean settlement for rock socket minipiles lies between that for cohesionless and cohesive ground, but a greater spread of response is observed. Where the upper length of a rock socket minipile is sleeved or passes through weak ground elastic compression of the upper length of the shaft can be a significant proportion of the total recorded settlement. This is a normal response and should not automatically be taken as an indication that the rock socket pile is defective.

It can further be seen from Fig. 1 that the majority of piles tested did not develop normalised settlements in excess of 10% of the pile diameter (often considered the failure criterion) at 2xDVL or even 3xDVL, suggesting these minipiles had geotechnical factors of safety well in excess of the 2 to 3 usually required.

Table 3. *Summary of load settlement data.*

	DVL		**DVL + 50%SWL**	
	Mean Settlement	**SD**	**Mean Settlement**	**SD**
Soil Type	mm	mm	mm	mm
Cohesionless	2.6	1.5	4.6	3.0
Cohesive	3.2	1.5	6.2	3.5
Rock	3.1	2.2	5.2	3.5
All Data	3.0	1.8	5.2	3.4

OBSERVATIONS ON MINIPILE GEOTECHNICAL CAPACITY

From standard design practice the geotechnical capacity of a pile can be estimated by:-

$$Q_T = Q_S + Q_b \qquad (1)$$

Where Q_T is the total axial capacity, Q_S is the shaft capacity, and Q_b the base capacity.

Most minipiles have a high slenderness *(l/d)* ratio and hence end bearing capacity, Q_b, may be small due to the small base area and will not be fully mobilised at working loads. In this respect the estimate of the ultimate shaft capacity is of most practical interest in minipile design. Where required for this paper, the contribution of end bearing is estimated and subtracted in the back analyses of overall pile capacity to give the derived shaft load. The mobilisation of shaft capacity is considered separately for minipiles in cohesionless soil, cohesive soil and rock.

Shaft Stress in Cohesionless Soil

The shaft load for piles in a cohesionless soil with diameter, *d*, bond length, *l,* at a depth with the vertical effective stress, σ'_v, and interface angle of friction, δ, is given by:-

$$Q_s = \int_0^l \pi.d.k_s\sigma_v' \tan\delta.dz \qquad (2)$$

For a drilled and grouted minipile δ may be taken to equal the soil angle of friction ϕ'. The coefficient of lateral stress is defined as k_s. For large diameter CFA or bored piles, k_s typically takes a value between 0.6 to 0.9. (Fleming et al[5]). The base load is given by :-

$$Q_b = \frac{\pi d}{4}.Nq.\sigma_v' \qquad (3)$$

Where N_q is the bearing capacity factor, taken in this paper as that proposed by Berezantzev et al[6]. Summarising $k_s \tan\delta$ as β (equation 4) Fig. 2 shows β plotted against the length of pile shaft embedded in a cohesionless stratum.

$$\beta = k_s \tan\delta \qquad (4)$$

The value of β is shown to between 1.1 and 5.0 (the high value of 5 relates to a pile formed by pressure grouting – which clearly enhances capacity). Rollins et al[7] suggested that β decreased as pile length increases (as shown by the curve in Fig. 2) however the data presented here are too scattered to verify this assertion and hence the current practice of considering β to remain constant with depth (in a consistent soil stratum) remains reasonable. Also shown in Figure 2 is the value of k_p which is shown for reference.

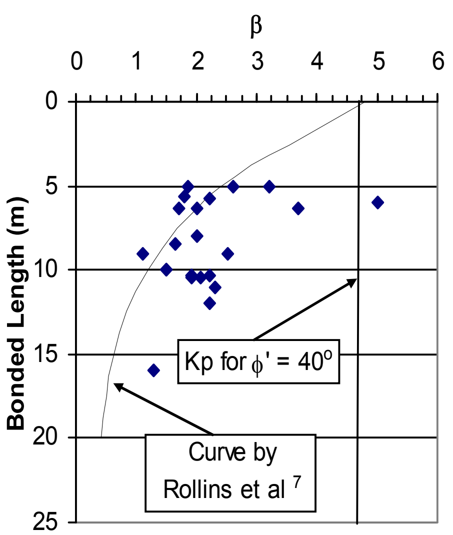

Fig 2: *β plotted against minipile bonded length*

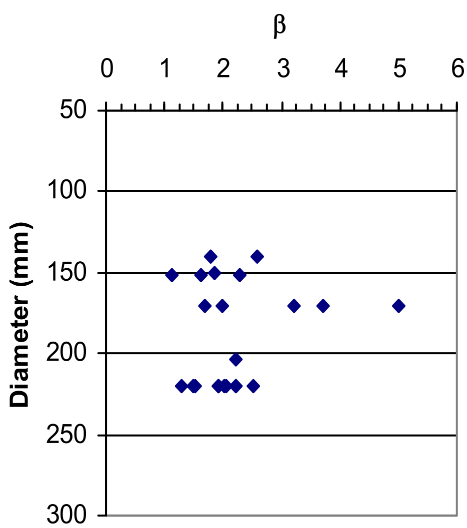

Fig 3: *β plotted against minipile diameter.*

Fig. 3 shows β plotted against pile diameter and no significant variation of β with changing diameter is observed. The diameter range for minipiles is small, by definition, and may account for this finding.

Where relevant data were available β was plotted against SPT N in Fig. 4. The SPT data comes from contract boreholes for which the frequency and number of SPT's and the proximity of boreholes to test piles all vary. Nevertheless a relationship whereby β increases with N is observed. Taking a practical range of angle of friction, ϕ', of between 30° and 40° proved values of k_s range between 1.3 and 4.4; far in excess of the values of 0.6 to 0.9 quoted by Fleming et al[5] for larger diameter piles.

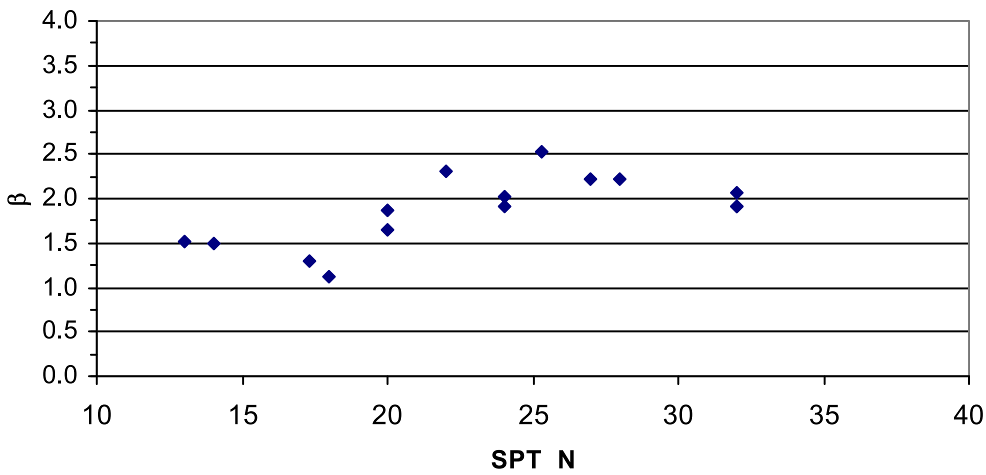

Fig 4: *β plotted against SPT N value*

Shaft Stress in Cohesive Soil

For piles in cohesive soils the total stress approach is still the most commonly used in UK. In soil with an undrained shear strength, c_u,

$$Q_T = \int \pi d \alpha c_u dz + \frac{\pi d}{4} N_c c_{ub} \tag{5}$$

Where α is the adhesion factor, and N_c bearing capacity factor. Typically α ranges between 0.5 and 0.7 depending on clay plasticity and strength and N_c is taken as 9.

In most practical situations the values of diameter, d, and undrained shear strength, c_u, are known leaving the designer to select an appropriate value of α. Table 4 shows the results from the evaluation of α from back analysis of minipile test data in clay where sufficient information on the undrained shear strength, c_u was available. (Reliable information on c_u was rarely found in the archive data thus making a robust evaluation of α impossible in many cases.)

Table 4 *Summary of adhesion α, for minipiles in clay.*

Pile Length (m)	Pile Diameter (mm)	Average Cu kN/m^2	Alpha
12.5	220	125	0.88
15.5	220	135	0.85
18.0	300	125	0.58
21.0	250	163	0.48
21.5	220	157	0.53
26.0	250	170	0.53
27.5	220	170	0.45

The data demonstrate that the α value is usually between 0.4 and 0.6. Two cases for relatively short piles have been recorded in London Clay where back analysed α values were in excess of 0.8. However on the same site longer piles were shown to have a much lower value of around 0.5.

The values of α are generally in line with those adopted for the design of piles with larger diameter confirming that similar values of adhesion can be justified for minipiles.

Shaft stress in rocks

Minipiles demonstrate extremely high shaft capacities in rock. For minipiles in rock the design ultimate capacity is usually estimated as shown in equation 6.

$$Q_T = \pi d q_s l + \frac{\pi d}{4} q_b \qquad (6)$$

Where q_s is the ultimate shaft bond stress and q_b ultimate base stress which are each evaluated via correlations based on rock strength (usually unconfined compressive strength, UCS). As noted above end bearing capacity is usually of less interest in minipiles compared to larger diameter piles and will not be considered further (although it can be used to good effect in certain circumstances).

A number of correlations can be used to evaluate bond stress, q_s, for large diameter piles (for example Horvath[8] or the lower bound 0.1 x UCS) A useful summary of some of the different methods is provided by Gannon et al[9], however these are generally conservative for minipiles which are acknowledged to achieve higher shaft stresses. Rock socket minipiles can be safely designed by referring to the relationships and proven test bonds described by Barley[10], which although developed from ground anchor tests can, with experience, be employed in minipile design where similar diameters and drilling and grouting methods are used.

In addition to ultimate compressive strength (UCS) of the rock the shaft capacity is also related to the rock type and the roughness of the socket formed. Therefore the relatively high shaft stress that can be achieved in minipiles compared to large diameter piles in similar rock can be attributed to some or all of the following features.

- The ability of grout to penetrate into joints, fissures, and other small discontinuities
- The relative effect of over break or shaft roughness is greater due to much smaller minipile diameter. The normal stresses set up as the pile bore attempts to dilate as a result of the roughness at the rock-pile interface have proportionally greater effect the smaller the pile diameter
- The development of a clean pile bore and removal of arisings as a result of vigorous flushing of the hole.

For this paper 50 load tests for rock socket minipiles were reviewed. Although many were tested to over 2.5 times the design load, only two proved the ultimate shaft capacity of the rock socket friction available.

Surprisingly Fig. 1 also shows that at working loads rock socket piles often exhibit slightly greater settlement compared to minipiles in soil at working load. This is attributed to elastic compression of the shaft above rock head rather than movement within the socket length. Although not the subject of this paper, this aspect of rock socket pile behaviour has significant implications for the rigorous analysis of minipile performance based on the Chin[11] or Fleming[12] load settlement relationships, and they should be used with care and by experienced personnel.

A summary of the range of back analysed shaft stress for rock socket piles is given in Table 5. It is emphasised that these are not ultimate values which could be considerably higher.

Table 5. *Tested shaft stress for rock socket piles.*

Rock type	Tested shaft stress kN/m^2
Mudstone / Marl	200 to 600
Sandstone	420 to 950
Limestone	875 to 3500
Granite /Dolerite	1400 to 2700

Unfortunately current ground investigation practice rarely yields sufficiently comprehensive data on rock strength and condition and such data were not available for the back analyses. However qualitatively, as would be expected, the stronger the rock the higher is the tested shaft stress.

FACTORS AFFECTING MINIPILE GEOTECHNICAL CAPACITY

As already indicated the factors affecting the mechanism of load transfer between the minipile shaft and the ground are many and complex but nonetheless they can be grouped into three categories, pile geometry (diameter and length), grout/ground interface properties and behaviour and interface stress field during loading.

Diameter and Length

The diameter and length of the grouted body obviously affect the minipiles geotechnical load capacity through the area of the grout body in contact with the soil. Length is easily determined however the final diameter of the grout body is more difficult, if not impossible, to determine – other than by exhumation. The final grouted diameter is affected by aspects of the drilling and grouting process, which are in turn governed by the ground conditions as discussed below.

Over break may affect the diameter and this is likely to be most significant in cohesionless soils and when using water or air flush. Back calculations of pile diameter from grout takes is likely to be prone to error and misleading due to the difficulties of accurately measuring grouting volume. The rheology of the grout changes under pressure as water is driven out of the grout into the ground when pressure grouting. Experience has shown that permeation of cementitious grouts is likely in cohesionless soils with gradings no finer than the medium sand range Littlejohn[13].

Expansion of the pile bore can be achieved by pressure grouting. Here the process of cavity expansion links the expanded socket diameter to the expansion pressure mobilised at the borehole wall. The process is extremely complex, as described by Egan[15] in relation to ground anchors, and affects both the grouted diameter and the normal stress acting at the borehole wall and both affect the shaft capacity.

Extension of the shaft through flow of the liquid grout into cavities or fissures existing in the ground can also enhance capacity as explained by Barley[16].

Interface Properties and Stress Field Under Loading

The properties of the interface between the grouted body and the ground will be affected by the drilling method, soil or rock type and grouting technique. There is no universal method of calculating what the interface properties and behaviour will be for a given set of conditions. Faced with such difficulty, experience with drilling methods in local conditions is the best guide, backed up by a sufficiently robust regime of pile testing where any residual uncertainty in the design can be investigated. It is for this reason that the development of a database of minipile performance over a wide range of ground conditions is so valuable.

In drilled minipiles disturbance to the host ground is minimal however some disruption to the pre-installation stress field in the narrow interface zone adjacent to the grouted body is inevitable. The full stress path that the ground adjacent to the borehole is subject to can not, at present, be accurately quantified, other than by recognising that a regime of stress increase and decrease will be imposed during penetration of the drilling tool or casing, removal of ground, removal of the tool, grouting (which may be under pressure), final casing removal, and volume change during hydration of the grout. As the pile shaft is loaded a further set of displacements and stress changes will be imposed within the interface zone.

Rarely will a pile bore be perfectly smooth and small scale irregularities will create some degree of interlock with the ground. The degree of interlock and its effect will depend on the drilling system adopted and the soil or rock into which the pile bore is formed.

In sands and gravel interlocking maybe developed by partial cementing of soil grains into the rigid grout body, possibly supplemented by small scale grooving if the host soil has sufficient cohesion or cementing. In clay some small scale grooving may occur which may lead to a slight increase in effective pile diameter – a philosophy sometimes adopted in the design of helical displacement piles. In rock some roughness of the pile bore is inevitable which allows interlocking to develop.

Even where a limited degree of interlocking is present, over-riding of the interlocking elements lead to the generation of dilation forces as the pile shaft displaces with respect to the host soil. The radial strain generated by the dilation process can be considered analogous to the radial strain created during the expansion of a cavity. This in turn can lead to a significant increase in the normal stresses and hence shear stress acting on the pile soil interface.

Thus the mobilised shaft stress would be expected to increase for piles of reducing size formed in a given soil. This hypothesis is supported by Trouton and Stocker[16], who reported data showing significant increase in shaft stress in both cohesive and cohesionless soils with decrease in socket size from 1500mm diameter piles down to 60mm diameter anchors.

CONCLUSIONS

Over 100 load tests on drilled minipiles installed in a wide variety of soil and rock have been analysed to consider factors affecting their performance and load capacity. The tests encapsulate over 20 years of the practical minipiling experience of three UK contractors.

From the test data analysed there is ample justification that drilled minipiles installed by experienced practitioners using suitable drilling techniques can be designed using more optimistic parameters that would be normal practice for large diameter piles in cohesionless soil and rock.

The combination of enhanced design parameters and the ability to install minipiles with lightweight and small drilling rigs makes their use ideal for restricted or difficult access situations where high loads are still required.

The load settlement response of drilled minipiles has been proven to be excellent for a wide range of applications and loads and settlement will usually be less than 6mm at working load. However if the upper part of the minipile shaft is de-bonded over a long length or greater loads are adopted higher settlement should be anticipated.

It would appear that drilled minipiles constructed in cohesionless soils can achieve higher ultimate capacities that predicted by design methods traditionally used for large diameter piles. In particular the data suggest of k_s values in excess of the 0.6 to 0.9 range often adopted for large diameter piles are appropriate. Generally these will tend to be higher for more dense cohensionless materials.

Drilled minipiles formed with sockets in competent rock also showed higher capacities that predicted by traditional large diameter pile design methods. The design of minipiles in rock may be facilitated, with appropriate experience, using the approach suggested by Barley[9] which appears to remain appropriate.

Unlike minipiles in cohesionless soil and rock, minipiles piles in cohesive deposits appear to mobilise ultimate shaft capacities that are in general agreement with the total stress design approach. Values of the adhesive factor, α, adopted for large diameter piles seem appropriate for minipiles also.

ACKNOWLEDGEMENTS

In writing this paper the authors wish to acknowledge the help and input of many engineers and colleagues who have, over the years, provided valuable data, advice and comment and without whom this paper could not have been produced.

REFERENCES

1. British Standards Institution. BSEN14199:2005. *Execution of special geotechnical works. Micropiles.* Code of practice. BSI, London, 2005.
2. British Standards Institution. BS8110:1997. *Structural use of concrete.* Code of practice. BSI, London, 1997.
3. British Standards Institution. BS5400:1990. *Steel, concrete and composite bridges.* Code of practice. BSI, London, 1990.
4. British Standards Institution. BS8004:1986. *Foundations.* Code of practice. BSI, London, 1986.
5. Fleming W A K, Weltman A J, Randolph M F, and Elso W K. *Piling Engineering.* 2nd Edition. Taylor and Francis London. 1992.
6. Berezantzev V G, Khristoforou V, & Golubkou V. *Load bearing capacity and deformation of piled foundations.* Proc. 5th Int. Conf. S. M. & F. E., Vol.2: 11-15. 1961.
7. Rollins K M, Clayton R S, Mikesell R C and Blaise B C. *Drilled shaft side friction in gravelling soils.* J. Geotech & Geoenvironmental Eng. ASCE. August. 2005.
8. Horvath R G. *Field load data on concrete rock 'bond' strength for drilled pier foundations.* Dept of Civil Engineering, University of Toronto Publication 78-07. University of Toronto, Toronto, Ontario, Canada. 1978.
9. Cannon J A, Masterton G G T, Wallace W A and Muir Wood D. *Piled foundations in weak rock.* Report 181. CIRIA. London. 1999.
10. Barley AD. *Ten thousand anchorages in rock.* Ground Engineering, Vol. 21. Nos 6, 7 &8. 1998.
11. Chin F K. *Diagnosis of pile condition.* Geotechnical Engineering, No 9. 1978.
12. Fleming W G K. *A new method for single pile settlement prediction and analysis.* Geotechnique, 42(3). 1992.
13. Littlejohn, GS. (1982). *Design of cement–based grouts* . Proc.Conf Grouting in Geotechnical Engineering ,ASCE , New Orleans
14. Egan D. *Compaction ground anchorage in sand.* PhD Thesis, University of Manchester. 1997.
15. Barley A D, Mothersille D and Weersinghe R. *Ground anchors in chalk – a review of current practice.* Geotechnical Engineering.
16. Troughton V M and Stocker M. *Base and shaft grouted piles.* Proc. Institution of Civil Engineers, Geotechnical Engineering. Vol 119 (3). 1996.