CANNON STATION REDEVELOPMENT, LONDON

Jim Martin, Byland Engineering Ltd, York, North Yorkshire, United Kingdom
Dr Hugh St John, Geotechnical Consulting Group, London, United Kingdom
Jim Fraser, Foggo Associates, London, United Kingdom
Tony Taylor, Foggo Associates, London, United Kingdom
Matt Smith, Expanded Piling Ltd, Dartford, Kent, United Kingdom

This paper discusses the geotechnical challenges overcome in providing the foundations for the Cannon Station Redevelopment, London. The original Station was constructed 150 years ago as a brick arch viaduct with an iron roof and has been substantially modified during its life. The present £350 million scheme dates back to 1995 and comprises major demolition works and the construction of a new steel framed office building with retail units, all supported on new columns which pass through the existing viaduct. The northern columns reuse the existing under-ream piles, supplemented by new settlement reducing piles. The southern columns found on novel micropile groups within the arches and on new under-ream and large diameter piles to the east where headroom is available. The micropile groups provide a safe and cost effective alternative to traditional hand-dug caissons with less impact on the important roman archaeology. This paper describes the foundation scheme, the preliminary test pile programme and the site works.

INTRODUCTION

Cannon Street Station was originally constructed in 1868 as a brick arch viaduct with an iron roof and has been substantially modified in the 20th Century. The 0.67 hectare site is situated North of Upper Thames Street and South of Cannon Street, with Dowgate Hill to the West. The original brick viaduct has generally been retained and supports the platform and concourse. The 1960's redevelopment included a new office block and revamping of the station concourse and infrastructure. The present £350 million scheme dates back to 1995 and comprises of the demolition of several existing buildings and the erection of a new 10-storey steel framed office building with retail units at lower levels (total area = 60,000m^2). In addition, these works will enhance the underground and mainline railway services for the some 25 million passengers per annum which use this major transport interchange.

The support of the new development above the station involves constructing four access cores plus several new internal columns through the existing viaduct with pile safe working loads (SWL) ranging from 4MN to 18MN. The northern columns are founded on the existing large diameter under-ream piles of 78 Cannon Place supplemented and stiffened locally by 10No new straight shafted 750mm diameter settlement reducing piles. The southern columns are founded on 11No micropile groups within the low headroom arch viaducts and on 11No new under-ream and large diameter piles to the east side where headroom is available. At the time of the original proposals the solution generally adopted for major piling in limited headroom was to use hand dug caissons, as the Charing Cross project had done. However, there were health and safety concerns with these techniques and a major design review was undertaken in 2003. Piling specialists were invited to put forward alternative methods and micropile groups were identified as a possible alternative. The micropile option was considered the most suitable because of the following reasons:

- Working room restrictions
- Obstructions from existing foundations
- Underlying Roman Governors Palace (Scheduled Ancient Monument)

The original micropile scheme comprised of circular groups of up to 24 micropiles penetrating ~20m into London Clay. The nominal 250mm diameter micropiles were installed at 500mm centres and formed with grout and a full depth central rebar. Design development increased the micropiles from 250mm to 300mm diameter using grout or concrete and with short reinforcement cages. The design focused particularly on resolving the conflict between penetrating the Victorian Railway arch footings and the preservation of Roman wall structures which fell within the micropile groups. The design sought to resolve the problems of piling tolerances in these areas as well as determining a process of ensuring relatively plumb piles which would maintain the shape of the group down to the base of the micropiles. This work led to examining alternative shapes for the groups such as squares and triangles.

Phase 1 of the micropile works (165No) was undertaken in mid to late 2008, with the remaining micropiles (65No), under-ream and large diameter piles installed during mid 2009.

GEOLOGY AND GROUND CONDITIONS

In Roman times this area was close to the north bank of the River Thames, just east of the confluence of the River Walbrook. The tidal shore of the Thames lay along the north side of the current Upper Thames Street, close to the southern boundary of the site. There have been four site investigations undertaken between 1959 and 2007, including several 30m+ deep bore holes, therefore the ground conditions below the site are reasonably well understood. The British Geological Survey Map indicates that the site is underlain by Made Ground, overlying drift deposits of River Terrace Gravels. The underlying solid geology comprises sequential deposits of London Clay, the Lambeth Group and Thanet Sands. The Upper Chalk is present at depth and is a major aquifer in this area. Scour holes in the London Clay have been identified in the historic Thames foreshore area to the south of the site. The geotechnical data indicates a north south valley, perhaps a paleo (Pleistocene) channel in the surface of the London Clay, which lowers the level of the London Clay from +4.0mOD (ordnance Datum) to ~ +2.0mOD in the centre of the site. The London Clay extends down to ~ -42.0mOD, where the interface with the Lambeth Group occurs. The existing foundations and / or the pile cut off level for the micropile groups extend down to +2.0mOD and therefore the micropiles are wholly in the London Clay. The undrained shear strength versus reduced level plot is given in Figure 1 and is summarised as Cu = (80+5.13z) kN/m^2, where z is the depth below +5.0mOD.

EVOLUTION OF THE FOUNDATION SCHEME

At the outset of discussions the use of micropile groups were proposed as a system of temporary works, within which traditional hand dug piles (caissons), would be excavated. Replacing large volumes of stiff London Clay with concrete was not a particularly 'green' or economic option, which led to the development of composite open groups of micropiles. The conventional arrangement of closed micropile groups with grids of equally spaced piles in both directions was considered, but this was very disruptive to the archaeology. In addition, open groups loaded the micropiles equally and efficiently, unlike closed groups which can develop a large range of loadings. The foundation loadings had been assessed as between 4 and 18MN. Initially, a 16MN pile group was analysed to determine the behaviour of

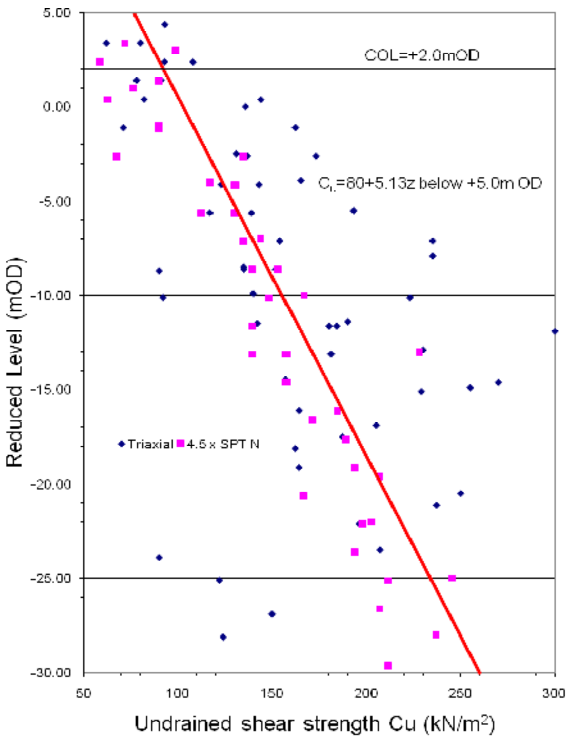

Figure 1 - Cu v Reduced Level

isolated groups, followed by global analysis of the pile groups' arrangement as required by the building. Figure 2 illustrates the overall foundation layout. These preliminary analyses were based on recent micropile experience in London, see Gill[1] et al (2008).

Settlement of Individual Pile Groups

The analysed pile group acts predominantly in skin friction, mobilised on the outer perimeter of the pile group. Accordingly, the settlement of the individual micropile group is small, being less than 10 mm immediate settlement with a minimal addition in the long-term. Elastic shortening of the micropile is around 4 mm. The stiffness offered by the micropile group-soil system to a column load acting on the group is therefore not greatly different to a single solid pile of equivalent capacity.

Interaction Between Adjacent Pile Groups

The results of the axi-symmetric analysis of an individual micropile group show that some of the pile groups are sufficiently close to each other to interact (<10m). It was assumed that the predicted settlement of the 16 MN pile group is similar to that of the smaller pile groups which have lower column loads but smaller pile caps. This assumption is reasonable because the pile groups act primarily in shaft friction and the average shear stresses mobilised on the periphery of the various pile groups are similar.

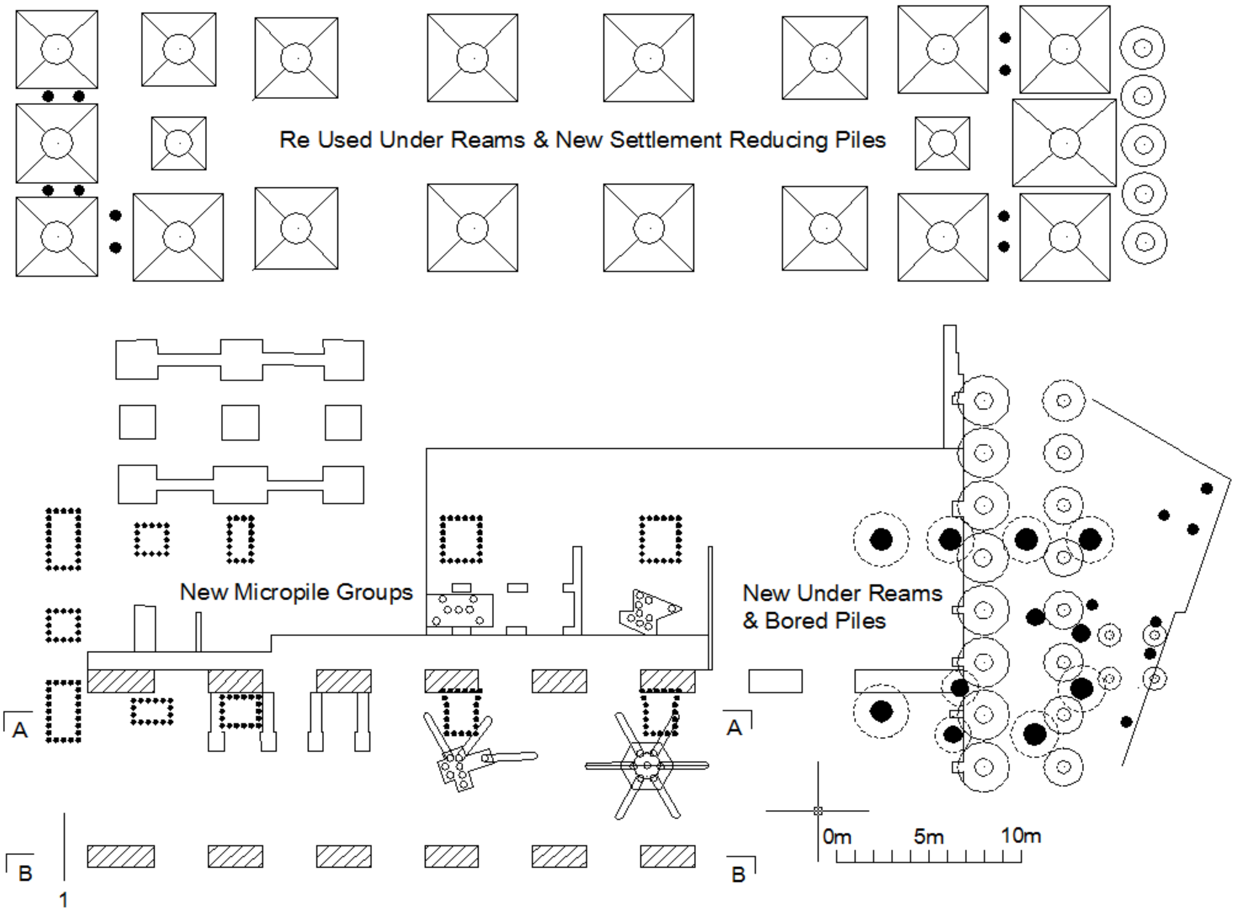

Figure 2 - Plan View of Foundations (new piles shown solid)

The plan arrangements of the various types of pile are shown in Figure 2. Finite element analyses of three closely spaced rectangular micropile groups gave immediate settlements of ~10mm, increasing to ~15mm in the long term. Differential settlements between adjacent micropile groups remain small, <3 mm in the short term, increasing to ~4 mm in the long term. These levels of differential settlements are unlikely to cause damage to the proposed development.

<u>Settlement of Existing Arches</u>

Figure 3 shows the immediate settlement profiles along Sections AA and BB, which cover the arches subject to the highest differential settlements as a result of the loading of the micropile groups. The same principle of superposition was used to derive these figures and the settlement profiles represent green field values. This is conservative because the stiffness of the arches will reduce these

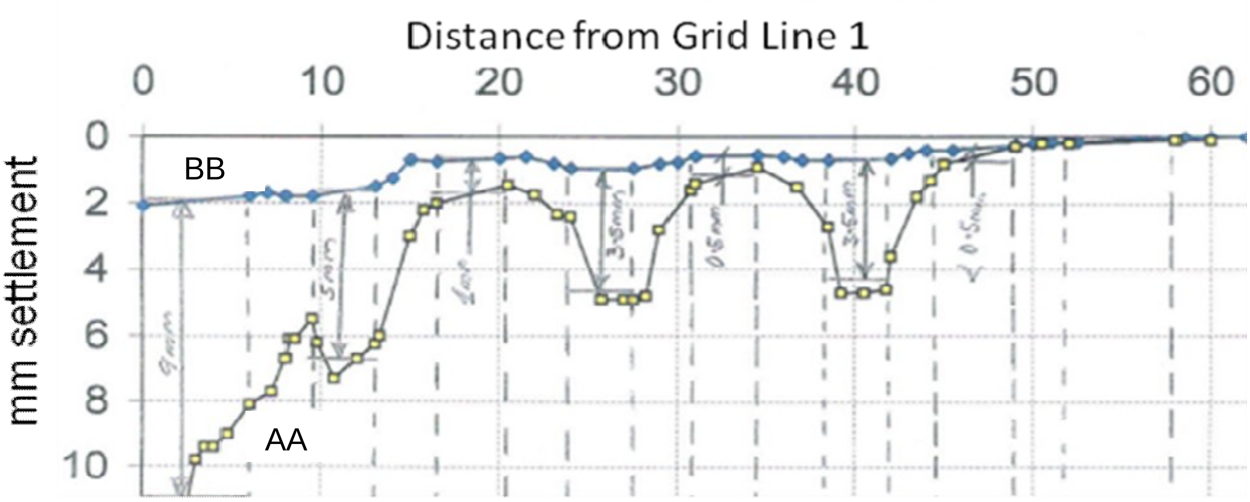

Figure 3 - Settlement Profiles of Arches along AA and BB

settlements. It can be seen that the maximum differential settlement in a North South direction is ~9 mm occurring over an arch span of 9.7 m. These differential settlements are predicted to increase ~2mm in the long term which is unlikely to distress the arches.

PRELIMINARY MICROPILE TESTS

In view of the innovative nature of the micropile works it was considered imperative to undertake a comprehensive preliminary pile test programme. Of particular interest was the difference in performance between grout micropiles and concrete micropiles. There were also concerns that the proximity of the micropiles to one another in the groups may result in a reduction in the shaft friction. The preliminary testing was designed to evaluate several issues:

a) Confirm the suitability of the drilling and construction techniques.
b) Confirm the verticality achievable.
c) Calibrate and confirm the suitability of dynamic tests as an alternative to static working pile tests.
d) Investigate the difference in performance between grout and concrete micropiles.
e) Investigate the difference between single and group pile performance.
f) Use the results to optimize the scheme.

The preliminary pile tests were carried out several months in advance of the main works to allow time to make full use of the data. Four static pile tests and several dynamic tests were undertaken between January and April 2007. The layout of the preliminary test piles is given in Figure 4, which allows multiple uses of the reaction anchor piles. The group micropile tests comprised of testing the central pile of a group of 3 piles at 500mm centres, with the outer piles installed first to replicate the worst case conditions (i.e. allowing for potential ground relaxation caused during construction of the outer piles).

<u>Micropile Boring</u>
The drilling of the London Clay was undertaken using nominal 0.75m lengths of 300mm diameter segmental augers and the drilling was relatively straightforward, as expected. There were intermittent band of claystones which did not cause any problems and it took on average 2.5 hours to drill to 25.0m depth. A 5-tonne Klemm 702 drill rig with a 2.2m mast was used in the limited 2.5m of headroom. The micropile group design was based on a 'vertical / plumb' micropile and there was some concern regarding the flexibility of the segment auger joints affecting the verticality. An experienced driller was imperative, who would repeatedly 'hold back' the augers (i.e. pull them

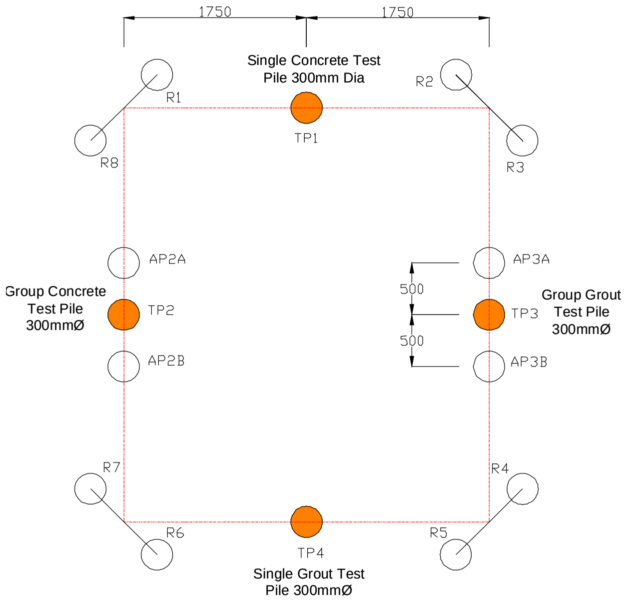

Figure 4 - Preliminary Test Pile Layout

back into tension) to prevent them being forced into the ground and going off the vertical. This meant that the augers were 'hanging' vertical, similar to a 'plumb bob', immediately prior to each short boring cycle. Boring became progressively slower below 20m depth and extra care was required over the final few metres when there was a considerable weight of augers in the bore. Excellent verticality was achieved and the bases of the micropiles were clearly visible when a bright light was shone down the bore. In view of the importance of good verticality for the group design, a simple method of checking and validating the micropile verticality was developed (See Figure 5). This comprised of hanging a centralising bracket (spider) in the base of the bore via a fine cable from piling platform level (PPL). The cable was then plumbed vertical using a steel support frame and a 1.2m spirit level. Finally, the distance from the centre of the pile casing at PPL to the plumbed cable was measured (northing and easting). The pile verticality was then simply calculated by dividing the pile depth by the recorded measurements. This method relies on the cable not touching the sides which must be visually checked by shining a light down the bore. The maximum out of vertical tolerance is therefore equal to half of the pile diameter, i.e. 150mm over 25,000mm or approximately 1 in 166. If the cable did touch the side, then the depth of the spider would be recorded and the verticality interpolated to the full depth (i.e. assuming that the pile carried on at the same inclination). This only happened on three occasions.

Figure 5 – Support Frame and Centralising Bracket

<u>Micropile Concreting</u>

Due to the restricted nature of the site it was necessary to deliver the concrete to the nearest public highway, and then pump it up to 50m to the micropiles. A C32/40 pump mix (grade DC2) with 50% fines and superplasticizer, with the following batch weights, was used:

- 310kg OPC (CEM1)
- 130kg Pulverized Fuel Ash
- 823kg Sand
- 823kg 10mm Aggregate
- 198lt water (0.45 Water Cement Ratio)
- + Superplasticizer

To ensure good pumpability and compaction, an S4 slump (180mm to 210mm) was specified. This mix gives a theoretical density of 2,284kg/m^3, which compares well with the average cube density of 2,272kg/m^3. The average 28-day UCS cube test result was 36MPa and the average 56-day result was 41MPa, the long-term prediction after several months is ~44MPa. Whilst these results are slightly lower than specified, they will have minimal effect on the micropile load capacity and the insitu strength is likely to be higher. The average concreting time was 1.25 hours with an over break of 20 to 30%, which is typical for this type of pile. The concrete was poured into the top of the clean dry bore until full (see Figure 6), and then the central 63mm diameter rebar plunged into the fluid concrete. An additional (6 x H20) reinforcement cage was incorporated over the critical upper section of the pile.

Figure 6 – Concreting of micropile

<u>Micropile Grouting</u>

The grout was a colloidally mixed (high shear) 1:1 sand cement mix with a 0.45 water cement ratio, with the following batch weights:

- 100kg Ordinary Portland Cement (CEM1 – Grade 42.5N)
- 100kg sharp concreting sand
- 45 litres of water

This standard mix produces 114 litres of grout, using a Colmono CX 4/10 grout mixer / pump unit. The theoretical density of the grout mix is 2,143kg/m^3, which compares well with the average cube density of 2,141kg/m^3. The average 28-day cube result was 61MPa and the average 56-day result was 81MPa, the long-term prediction after several months is ~90MPa. These results are higher than expected and indicative of good quality sharp sand and thorough colloidal mixing (see Figure 7).

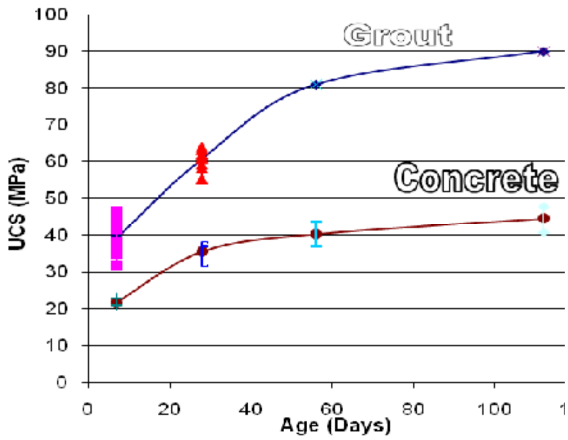

Figure 7 – Concrete and Grout Strength v Time

The average grouting time was 2.0 hours (22 batches per micropile) with an over break of 20 to 30%, which is typical for this type of pile. The grout was pumped into the base of pile bore until any debris was flushed out and it was completely full of clean grout (see Figure 8). The central 63mm diameter rebar was inserted prior to grouting due to concerns over the grout setting quickly at the base and preventing installation of the rebar. An additional (6 x H20) reinforcement cage was again incorporated over the critical upper section of the pile.

Figure 8 – Grouting of micropile (debris being flushed out)

The advantages and disadvantages of using concrete and grout are summarised in Table 1:

Advantages & Disadvantages of Concrete		Advantages & Disadvantages of Grout	
Fast	Reliant on concrete supplier	Mix as and when required	Slow
Cost effective	Difficult to obtain small quantities	Can mix small quantities	Expensive
Good pile performance	Require areas for concrete mixer delivery and pump	Small plant can be moved to minimise pumping distance	Noisy & dusty on site
	Inevitable debris left in base	Debris flushed out of base	

Table 1 – Advantages and Disadvantages of Concrete and Grout

The preliminary test piles incorporated a central 63mm diameter GEWI rebar to 24m depth, connected together in 3m lengths using full strength couplers. This central rebar was required to increase the structural capacity of the preliminary test piles to accommodate the maximum test load of up to 2,400kN. The central 63mm diameter rebar is not required in the working piles.

<u>Micropile Testing and Results</u>

Precision Monitoring and Control Limited carried out the static pile tests and Testconsult Limited carried out the dynamic testing, generally in accordance with the ICE Piling Specification. All testing was undertaken between 18 and 46 days after micropile construction. Dynamic tests were carried out before and after the static test on TP1. The definition of pile ultimate capacity was agreed to be equal to the required applied vertical force to develop 30mm of pile head settlement. The static test pile reaction framework is shown in Figure 9 and the results summarised in Figure 10.

Figure 9 – Static Test Pile Reaction Framework

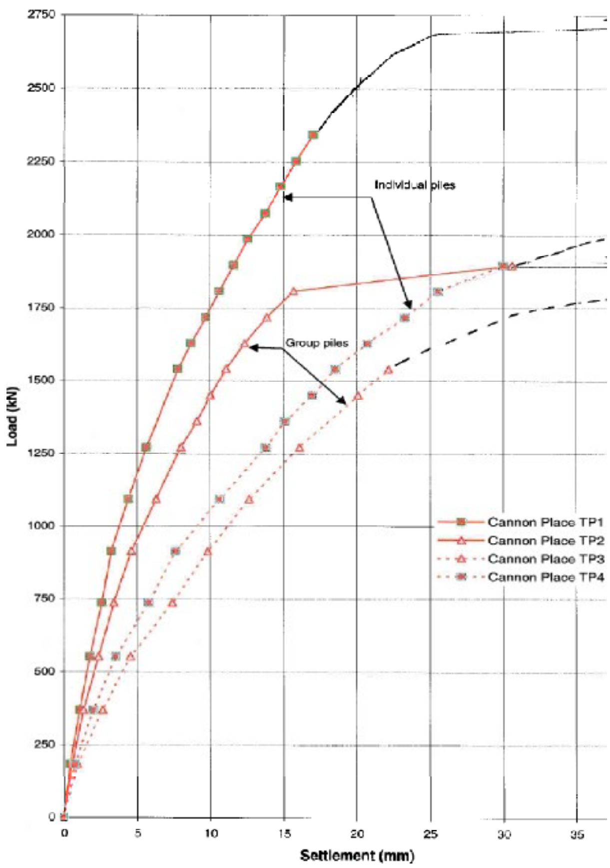

Figure 10 – Static Test Pile Results

The ultimate shaft and end-bearing capacities were calculated using Fleming's method [3], (see Figure 11).

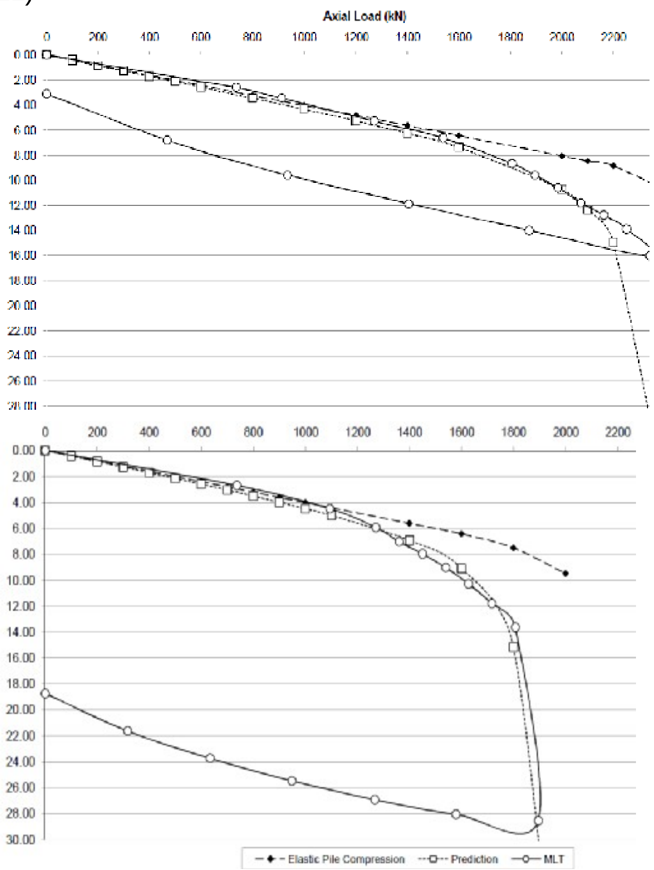

Figure 11 – Cemset Analyses of TP1 & TP2 using Flemings Method

The back calculated α-values from the test results and Cemset analyses are summarised in Table 2.

Pile Test No	Concrete / Grout	Group / Single	Ultimate Load from Test & Prediction, Q_{ult} (kN)	Back Calculated α-value
TP1	Concrete	Individual	2,700	0.592
TP2	Concrete	Group	1,900	0.405
TP3	Grout	Group	1,800	0.381
TP4	Grout	Individual	2,000	0.428
		Average concrete alpha α		0.499
		Average grout alpha α		0.405
		Average group alpha α		0.393
		Average overall alpha α		0.452

Table 2 – Summary of Ultimate Capacities & Back Calculated α-values

Dynamic Testing

Due to the high l/d ratios standard integrity testing would not be suitable. Testconsult were commissioned to undertake 9No SIMBAT dynamic pile tests. These involved dropping an 800kg weight through various distances of up to 1.0m onto the pile head. An accelerometer is attached to the pile head which records the pile head acceleration, from which a predicted load versus settlement curve is calculated. The dynamic tests showed good correlation with the static test results, with no noticeable difference before and after the static tests. The dynamic tests slightly overestimate the concrete pile load settlement behaviour and slightly underestimate the grout pile load settlement behaviour (see Table 3). This can be explained by the instantaneous nature of the dynamic test and the use of a standard Young's Modulus value. The grout has a lower stiffness than the concrete due to the lack of coarse aggregate. The SIMBAT dynamic load tests have proved to be an acceptable, quick and cost effective method of undertaking the working pile tests, where conventional static load tests would be difficult in the restricted locations.

		Dynamic		Static
Reference	Type	Settlement at 800kN before static test (mm)	Settlement at 800kN after static test (mm)	Pile head settlement at 800kN (mm)
TP1	Single Concrete	5.0	4.1	2.8
TP2	Group Concrete		4.6	4.0
TP3	Group Grout	-	5.5	8.2
TP4	Single Grout	5.3	-	6.5

Table 3 - Summary of Dynamic & Static Pile Head Settlement at 800kN

Discussion on Preliminary Test Pile Results

Analysis of the test pile results gave a relatively large range of α-values from 0.381 to 0.592. However, if the individual micropile tests TP1 and TP4 are excluded (all the micropiles in the main works will be in groups), then the range drops to 0.381 to 0.405. The high α-value in TP1 may be explained by the concreting of the micropile quickly after boring (minimizing relaxation of the clay), and the plunging of the central 63mm reinforcement bar into the fluid concrete to 26.0m depth. The central 63mm rebar was coupled together in 13 No x 2.0m lengths and it took nearly 2 hours to install. Consequently, the concrete in the lower half of the pile will have been in place for nearly 3 hours before the rebar was plunged into it, and will have started to stiffen. The rebar met increasing resistance with depth and additional downward force was required with the final lengths. The rebar may therefore have been displacing the stiffening concrete laterally as well as vertically, which would have the increased lateral stresses and thus the α-value. This is equivalent to the lateral post stressing of a pile ('wedge' pile). The central rebar in the remaining test piles was installed prior to concreting /grouting because of concerns over installing the rebar to the required depth, which also allowed more time for the clay to relax. The main works micropiles will not contain a 26.0m deep central 63mm diameter rebar.

Generally, the group micropile ultimate capacity is ~10% below the single pile ultimate capacity and this is probably due to ground relaxation caused by prior construction of the adjacent piles. The concrete micropiles are noticeably stiffer than the grouted micropiles, even though the concrete strength is ~40MPa compared to the grout strength of 80MPa. Eventually, it was decided to adopt an α-value of 0.4, which agreed with the conclusions of Gill et al[1]. This relatively low α-value may be partly explained by the phenomenon of progressive debonding, more commonly associated with small diameter tendon ground anchors. The British Standard for Ground Anchorages[2] BS8081:1989 clause 6.2.3.4 specifically recommends a maximum fixed anchor length of 10.0m to mitigate these progressive debonding effects. In essence, as the micropile approaches its ultimate capacity, the upper section of the shaft has moved 30mm relative to the surrounding ground. This will cause progressive debonding at the top of the micropile shaft leaving only residual shaft friction, which will extend further down the micropile as the applied load increases (see Figure 12). These micropiles have high length over diameter (l/d) ratios of ~100 and effectively behave as long stiff springs, with ~1/3rd of the pile head settlement at the 'ultimate' capacity being elastic compression of the shaft. An alternative hypothesis is that the low α-value may simply be due to the soil around the micropile shaft suffering a greater amount of remoulding during the boring process than a normal auger bored pile, or perhaps it is a combination of both.

The testing programme confirmed the micropile group design assumptions apart from the lengths of certain micropiles which required deepening to accommodate the reduced α-value of 0.4. An overall geotechnical factor of safety of 2.0 was adopted for the micropile design of each group, assuming that the group capacity was the sum of the individual micropiles. The alternative design model is shear failure of the perimeter of the micropile group. In this model much of the failure plane is through the clay itself rather than the disturbed shaft perimeter, in which case the α-value will be considerably higher at around 0.7 or more. So although the area of this perimeter is less than the sum of the shaft perimeters it is compensated for by the greater adhesion factor. In addition the end-bearing area of the micropile group is significantly larger than the area of the sum of the individual micropile bases, thus giving a higher failure load albeit with larger settlement.

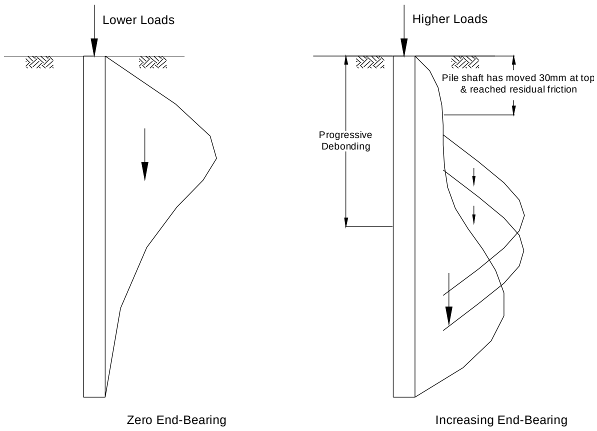

Figure 12 – Schematic Representation of Progressive Debonding

The preliminary test pile programme results indicate <5mm of pile head settlement at SWL, which ranges from 500kN to 735kN. The unreinforced 300mm diameter micropiles can structurally accommodate the axial forces from 6m to 10m below pile cut off level (COL) and the 20kN horizontal force per micropile requires (6 x H16) rebar extending to 3.0m below COL. It was decided to standardize on (6 x H16) rebar extending to 12.0m below COL in all piles. In summary, the following geotechnical design parameters were used in the main works design following the preliminary test pile programme:

- α-value (adhesion factor) of 0.40
- Undrained Cohesion (c_u) = (80 + 5.13 z) kN/m^2, where z is the depth below +5.00m OD
- Bearing Capacity Factor = 9.0
- Geotechnical Factor of Safety = 2.0

The decision was made to use concrete in the working piles due to the greater stiffness, faster construction programme and cost advantages. The above recommendations comply with the recommendations in the London District Surveyors Guidance Note 1[4].

Following completion of preliminary testing the design of the micropile groups was reappraised by GCG and more detailed assessments made of the group performance based on the back analysis of the test results on the concrete piles. In addition to the axisymmetric analyses that had originally been performed using FE analyses, the square and rectangular groups were modelled using the pile group analysis program REPUTE[5] in order to look at the distribution of loads in the individual piles under both vertical and horizontal loading. These analyses gave micropile loadings ranging from 500kN at the mid-points to 735kN at the corners.

MAIN WORKS MICROPILES

A total of 190No micropiles (75%) were installed during phase 1 between June and November 2008 using an electric Klemm 702 drill rig and a diesel Hutte 202 drill rig with 2.2m masts (see Figure 13). The electric drill rig was preferable because it did not produce fumes in the restricted basement areas. This total included 36No micropiles for tower crane bases. The majority of the minipiling works is in an archaeology sensitive area or alongside known remains. In preparing the ground for the micropile groups it was necessary to

Figure 13 – Micropile boring using nominal 300mm diameter augers

carefully excavate and record the archaeology to natural ground level. The method adopted is to accurately position steel casings into timbered excavations through which the micropiles are installed. The archaeology is protected by impervious sheeting, fibreboard and a suitable geotextile. These enabling works were undertaken by a separate contractor under guidance from the Museum of London Archaeology (MoLAS). The steel casings extended down to the London Clay and were sealed into position with a cement bentonite grout. The steel casings provided several advantages:

- Good positional accuracy at the pile head
- Improved verticality
- Mitigated potential damage & protected valuable archaeology
- Allowed micropiles to be constructed close to the roman walls
- Can be installed through cored holes in the railway viaduct foundations
- Removes obstruction risk in the Made Ground and existing foundations

The steel casings therefore removed considerable drilling risk for Expanded Piling Limited (EPL), with only ‘simple’ open-hole auger drilling remaining in the London Clay. It was possible to use concrete in all of the micropiles, which allowed the concrete level to be left 2 to 3m below piling platform level (PPL), leaving a minimum of 1.0m above COL. In approximately 40% of the micropile locations there was sufficient headroom and continuous thin walled expendable steel liners were used (see Figure 14). In the remaining locations where there was restricted headroom, standard segmental temporary drill casing was used. The drill rig would ‘break’ the temporary casing seal with the cement bentonite grout after the concreting of the pile and then remove and reuse the casing. There was a complicated logistical exercise to obtain the maximum reuse of the temporary segmental casing, and prevent the site becoming congested with casings.

Figure 14 – Thin walled expendable steel casing

The drilling and concreting operations went well, with up to 3No x 25m deep micropiles drilled and concreted in a single shift. The micropiles were always concreted the same day as they were bored to reduce the risk of relaxation of the London Clay. A slump check (180 to 220mm required) was undertaken on each load of concrete before it was accepted; several were rejected during the early stages of the works. The specified fluidity was essential to ensure good pumpability and concrete flowability around the 6 x H16 reinforcement cage to 12.0m depth. Excellent verticality was achieved with an average of 1 in 580 (<50mm over 25m) with a range of 1 in 114 to 1 in 1,786. The bores were substantially dry, with <10% recording between 100 and 200mm of standing water in the base. These small amounts of water will not affect the formation or performance of the micropiles. There were existing raking concrete piles passing below some bases; however the micropiles penetrated through these obstructions relatively easily in 30 minutes without being deflected. 6 No SIMBAT dynamic pile tests were successfully carried out during Phase 1 which confirmed the expected load versus settlement behaviour and provided validation of the pile performance.

REUSE OF EXISTING FOUNDATIONS & SETTLEMENT REDUCING PILES - THEORY

The northern half of the development reuses the existing large diameter bored piles supporting 78 Cannon Street. This proposal was adopted at an early stage because of the expense and disruption of removing and replacing the existing piles. There is information available on these foundations from several sources including Network Rail Archives. The structural plans detail the locations and pile diameters along with square underreams. In section the pile lengths and shapes are shown with information on original pile loadings. The site Investigation confirmed the location and extent of two of the underreams.

Engineering information on the original underreams was located in a 1966 paper on Large Diameter Bored Piles. This paper describes the piles at Cannon Street and gives particular details on the construction techniques and soil parameters used. Under 78 Cannon Street there are 18No x 2.13m and 2No x 1.83m diameter piles x 15.24m deep with a maximum load of 21MN. The piles have underreams of up to 6.55m square. The underreams were initially machine augered to 3.66m diameter followed by hand excavation to 'mine' out the larger square underreams. This information confirms details on the archive drawings.

A back analysis of an existing typical underream pile was carried out using current design techniques, which confirmed the stated pile capacity with an acceptable factor of safety. However, the quoted settlements appeared low, and this aspect of the design warranted closer inspection. The original design appears to underestimate these settlements stating that the centre piles would settle about 25mm and the western piles about 32mm over an initial 2 year period. In a discussion on the paper, it was suggested that deterioration of the clay took place while the pile shaft remained open for several days during hand underream enlargement. If such deterioration had taken place then it is likely to have had an initial adverse effect on the pile performance resulting in increased settlement. However the clay strengths would stabilise with time and under the current unloading / reloading a stiffer pile should result. The unloading of these piles during demolition is being monitored by precise levelling techniques and extensometers at key locations in the ground. At present the upward movements are within expected limits.

A detailed loading analysis of the existing building was carried out to establish the likely pile loads including wind forces. This confirmed that the existing piles have the capacity to carry the proposed new building loads. However the new loads are of a different nature and disposition to those under the existing building. The main differences occur at the east and west ends of the structure. In these areas the existing piles supported heavy concrete shear wall structures which provided lateral stability for the 14 storey tower. The consequential wind loadings are a relatively high proportion of the pile loads. However in the redevelopment the wind loads are reduced and the lateral restraint is provided by four new concrete cores. In summary the dead loads and superimposed live loads are increased, with residual horizontal forces on the end piles following erection of the superstructure. The end result is that the long term loads at the east and west end are higher than the existing loads but still have an acceptable factor of safety. This extra loading would cause additional pile settlement from that previously experienced. The mitigation measures taken to reduce the additional settlement was to construct intermediate 'settlement reducing' piles. These additional piles reduce the proposed loads on the existing piles to similar loads to those exerted by the previous building. The re-use of existing piles raises several issues of long term durability and insurability. A reuse scheme entails a thorough materials testing investigation to prove that the pile concrete and reinforcement are in a suitable state for reuse.

In addition to the proposed monitoring it was considered prudent to allow for the non destructive dynamic load testing of one or more of these reused piles during the construction phase. However this was not possible due to site constraints and scale of load test. Hence the extensometer tests and precise levelling programme have been increased.

New straight shafted settlement reducing bored piles are constructed between the reused underream piles in order to limit their settlement under working loads. A low factor of safety is applied to the ultimate capacity as they are designed to operate at close to or at their ultimate capacity in the working condition. An upper and lower bound geotechnical capacity has been considered. The pile toe level is determined by the lower bound capacity and the structural design of the piles is required to satisfy the upper bound. The pile design only considers the contribution of the pile shaft capacity from below the base of the adjacent underream piles. To allow a better prediction of the settlement reducing pile's load settlement behaviour and ultimate capacity, the piles have been constructed with spoilt bases, thus allowing the base capacity to be ignored in the design.

SETTLEMENT REDUCING PILES (SRP's) – CONSTRUCTION

The SRP's comprise of 10No new straight shafted 750mm diameter x 33m to 37m deep bored piles, each with a 25m long x 6H32 reinforcing cage. A number of these piles were constructed in only 4m of headroom, negating the use of conventional large piling rigs and equipment. During June 2009 the first piles were installed utilising rotary techniques with temporary segmental casing through the gravels. A specialist low headroom rotary piling rig incorporating a service crane was used, see Figure 15. Due to restrictions on the duration that the pile shaft could be left open, coupled with the time that it took to bore the piles in the low headroom, it would take 2 shifts to complete each pile. The upper 16m of the pile shaft (above the level of the adjacent underreams) was ignored in the design and therefore this could be drilled on day one and left open over night. On day 2 the pile was drilled to depth, the base 'spoiled' and the piles concreted using a 27m long segmental tremmie pipe. The reinforcing cage was made up of 13 sections; each individually marked up for sequence and orientation within the cage, with positional torqued couplers on each bar (72 per cage).

During boring of the second pile, fast water ingress was encountered at approximately 16m which coincided with the base of the existing hand dug underreams. We believe that fissures had formed in the clay surrounding the underreams during their prolonged construction period. These fissures had opened a pathway from the water table within the gravels above to the base of the underream. The pile position was moved ~300mm away from the closest underream and the pile was then redrilled successfully. After exploring a permeation grouting solution it was decided, for surety, to temporarily case the remaining piles to 18m using segmental temporary casing. This technique had the added benefit that if we hit the existing underreams then we could core through them.

Figure 15 – Low headroom piling rig & crane

UNDER REAM AND LARGE DIAMETER PILES

In the southern half of the development the new columns are founded on 11No micropile groups to the east and 11No underream and straight shafted piles to the west. These comprised of 7 No x 1,500mm diameter piles with 3,900mm underreams and 2No x 1,200mm diameter piles with 2,300mm underreams, up to 31m deep. During June 2009 a large 95 tonne hydraulic rotary rig was delivered to site with a 70 tonne attendant crawler crane. Although the piling area had no headroom restrictions the limited plan area, tight access and proximity of network rail infrastructure required detailed logistic and pile construction sequencing (see Figure 16). This was compounded by the presence of claystone bands at various depths within the shaft and in the underream itself. Due to this issue and previous problems other contractors have had constructing underreams in this area of London the decision was take not to remove any spoil from site until the concreting process was under way. The underream piles took up to 80m^3 of concrete and generated up to 150 m^3 of bulked spoil (see Figure 17).

Figure 16 – Restricted Western Site layout

Protection of the archaeology required pre installation of oversized (1,800mm diameter) pile casings in two locations. This operation was further complicated by the discovery of a steel casing, presumably a counterbalance shaft from an old loading crane, which intercepted one of the pile shafts. This casing was cored out and the bore backfilled with a weak concrete mix, which in turn was partially cored through when installing the permanent casing. CCTV was used to survey the completed underreams. U100 samples were taken from the shelf to check for remoulding of the clay at locations specified from the CCTV survey. The underreams were constructed at a rate of one pile per day, within the specified 12 hour period.

Figure 17 – Underream Tool

CONCLUSIONS

Testing and analysis have demonstrated the feasibility of using micropile groups in restricted headroom to support column loadings of up to 18MN at Cannon Place, London (see Figure 18). This innovative solution has several advantages over large hand dug caissons that would have been the only viable alternative. The pile test analyses conclude that an α-value (adhesion factor) of 0.4 for long slender micropiles in London Clay should be adopted. Excellent verticality's of ~1 in 580 (average) are achievable when care is taken during drilling, in conjunction with vertical steel guide tubes. The use of pumped concrete produces a stiffer pile, with associated cost, programme and environmental advantages. Dynamic pile tests provide a useful and cost effective method of validating micropiles in restricted headroom, where the dynamic test has been calibrated against a preliminary static test pile. The importance of preliminary test piles with sufficient time to develop the optimum design and construction techniques is imperative. Measurements taken during the construction of the superstructure will be used in further studies of micropile groups currently being undertaken as part of a research project at City University with support from a group of industrial sponsors. The Cannon Place Redevelopment has presented several geotechnical challenges in urban regeneration. These challenges have all been successfully overcome by the innovative use of micropiles, the reuse of existing piles, settlement reducing piles, bored piles and underream piles.

Figure 18 – New Columns on Micropile Bases

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