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Field measurements of the stiffness of jacked piles and pile groups

A. G. YETGINER*, D. J. WHITE† and M. D. BOLTON†

KEYWORDS: field instrumentation, piles

INTRODUCTION

This Technical Note presents a back-analysis of the axial response of some field-scale tubular displacement piles installed by jacking. This study was motivated by the recent development of large pile-jacking rigs. These rigs are sufficiently strong to offer an alternative to bored piling in urban areas where conventional dynamic methods of displacement pile installation are not permitted (White *et al.*, 2002). The most notable feature of the axial response was the very high stiffness.

Some pile jacking machines 'walk' along the pile wall under construction, so all piles must necessarily be installed at close centres, or indeed touching. This geometry is in contrast to conventional design guidance, which suggests that a separation of typically two or three diameters is required between piles, to prevent harmful interaction effects (BSI, 1986; GEO, 1996). Load test results from jacked piles installed closer than this recommended minimum spacing are presented.

The study comprised a series of field tests carried out in Kochi, Japan. Full details are provided by Yetginer *et al.* (2003), and brief details are repeated below. Test piles were installed singly and in groups forming a 'cell foundation', comprising jacked piles at close centres around an enclosed soil block.

In this Technical Note, pile group efficiency is defined as the average performance per pile (strength or stiffness) when a group is loaded in unison, divided by the performance of a single pile tested alone. Using this definition, the behaviour of trial piles installed prior to the main group can be linked to the group response. This definition is used by Vesic (1969) and Fleming *et al.* (1992), and is shown in equation (1) for strength and stiffness. Group strength efficiency, ζ_{GROUP}, is defined in equation (1a), where Q_{SINGLE} and Q_{GROUP} represent the capacity of a single (isolated) pile and the capacity of a group of n piles respectively. Group stiffness efficiency, η_{GROUP}, is defined in equation (1b), where K_{SINGLE} and K_{GROUP} represent the stiffness of a single (isolated) pile and the stiffness of a group of n piles respectively.

$$\zeta_{\mathrm{GROUP}} = \frac{Q_{\mathrm{GROUP}}}{n \cdot Q_{\mathrm{SINGLE}}} \tag{1a}$$

$$\eta_{\mathrm{GROUP}} = \frac{K_{\mathrm{GROUP}}}{n \cdot K_{\mathrm{SINGLE}}} \tag{1b}$$

An alternative definition of efficiency is the performance of a group of piles loaded in unison compared with a pile within that group tested alone. This definition is appropriate for extrapolating from a single pile load test among an already-installed pile group to the response of the entire group, but is not the definition used in this Technical Note.

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* Fugro Limited, Wallingford, UK (formerly Department of Engineering, University of Cambridge, UK).
† Department of Engineering, University of Cambridge, UK.

FIELD TESTING METHODOLOGY

Two cell foundations, each comprising a group of 12 piles (see inset in Fig. 1), a single pile, and walls of two and three adjacent piles were installed at the same site to the same founding depth. The ground conditions are shown in Fig. 2 (White *et al.*, 2000). Prior to installation of the test piles the made ground was removed and replaced by sand.

Each pile group consisted of 12 tubular steel piles with an external diameter D of 101·6 mm, wall thickness t of 5·7 mm and an embedded length of 5·85 m. The test piles were installed using a Giken AT-150 press-in piler. Plugs

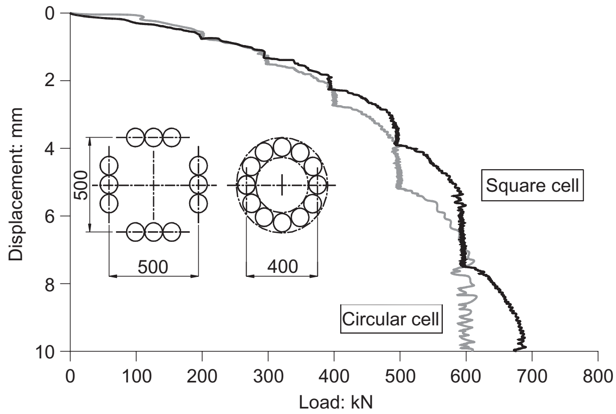

Fig. 1. Maintained load tests on square and circular cell foundations

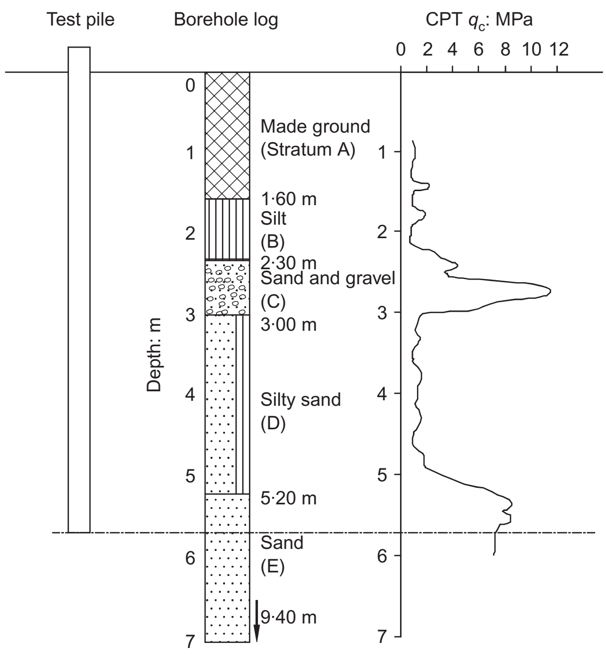

Fig. 2. Ground conditions at test site in Takasu, Kochi, Japan

formed in all the piles during the jacked installation process, and the piles remained plugged during subsequent load testing, behaving as if closed-ended. The installation and failure loads that were measured during these tests are presented in Table 1, with failure defined as the load at a settlement of 10 mm ($D/10$).

LOAD TEST RESULTS

Pile bearing capacity

The single pile showed notably high stiffness when load-tested, with ultimate capacity reached within a settlement of $D/50 = 2$ mm (Fig. 3). Pile design is usually governed by stiffness rather than strength (Randolph, 1994; Tomlinson, 2001). Driven piles usually reach plunging failure at a settlement of $\sim D/10$ (White & Bolton, 2005), whereas at $D/10$ settlement a bored pile has typically mobilised half of the base resistance of a driven pile in the same conditions (Ghionna *et al.*, 1993). Lee & Salgado (1999) found that whereas bored piles mobilise a base resistance typically 20% of q_c at $D/10$ settlement, this value rises to 40% for (dynamically) driven piles. In this test, the mobilisation of ultimate capacity at a settlement of only $D/50$ is therefore significant.

The installation and failure loads of the single pile were identical, at 50 kN, suggesting that for jacked piles in sand, tested within a few days of installation, the jacking force offers a reliable indication of the ultimate capacity (Table 1). Other field data suggest that pile capacity in sand increases with time, so jacking force can be considered as a conservative estimate of pile capacity (Chow *et al.*, 1998), except for the rare cases in which installation generates negative excess pore pressures, such as in highly dilatant silt (York *et al.*, 1994). The load–settlement curves of the two cell foundations show a more progressive reduction in stiffness with increasing load than the single piles. The full capacity is mobilised at a settlement of $D/10 = 10$ mm (Fig. 1).

Group efficiency

The capacities of the circular and square cell foundations in Takasu, defined by the load at a settlement of $D/10 = 10$ mm, were 600 kN and 700 kN respectively (Table 1), indicating a group strength efficiency ζ_{GROUP}, as defined by equation (1), of 1·02, when Q_{SINGLE} is taken as the installation load of the first pile in the group, which is comparable to the single pile load test capacity. This group efficiency value of unity indicates that in this case any positive or negative interaction effects balance each other. The first piles installed will receive additional lateral stress from the installation of the subsequent piles (as can be inferred from the driveability: see below). On the other hand, the interaction of the loaded regions around each pile appears to counter this benefit.

Enclosure effects: driveability

Enclosure effects are defined as the changes in the installation load of individual piles, as the construction of the cell foundations advances. The recorded values of maximum jacking resistance during construction of the cell foundations show that increased resistance is encountered as construction progresses (Table 1, Fig. 4). This is a well-known effect (e.g. Tomlinson, 2001), which can lead to problems of driveability in close-centred pile groups. Each pile displaces soil sideways during installation. If this sideways movement is obstructed by the presence of nearby piles, additional jacking force is required.

This recorded increase in resistance has implications for the deployment of pile-jacking machines for the construction of cell foundations. It is not sufficient to mobilise a machine with a jack capacity greater than the strength of a single isolated pile. Instead, for the geometry of the cells and ground conditions encountered in this study, a jack capacity of approximately twice the capacity of a single pile is required to install the final pile.

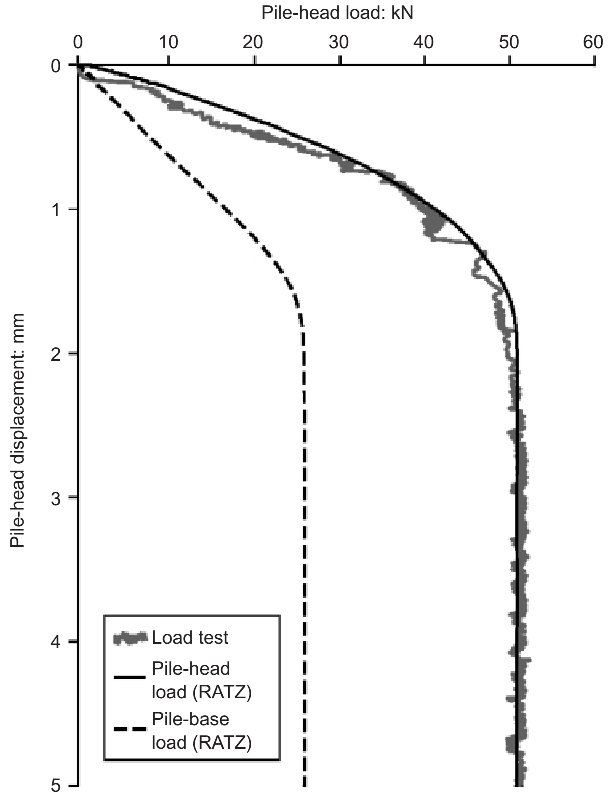

Fig. 3. Single pile load–settlement response, with RATZ back-analysis 1, $w_{bf} = 1$ mm

LOAD–SETTLEMENT ANALYSIS

Influence of installation method on pile base stiffness

A back-analysis using a t–z approach was conducted to examine the origin of the high pile-head stiffness evident in Fig. 3, using the RATZ software (Randolph, 2003b). The installation process of a jacked pile, compared with a bored or driven pile, leads to a different stress history for the soil

Table 1. Load test results

Pile group	Single pile	Square cell (in order of installation)	Circular cell (in order of installation)
Installation load(s): kN	49·9	57·4, 73·2, 72·9, 60·4, 99·8, 118·7, 83·3, 87·7, 101·8, 89·4, 108·2, 98·5	49·2, 71·5, 83·7, 79·3, 90·1, 89·6, 88·7, 115·6, 105·8, 109·5, 116·7, 128·5
Failure load ($D/10$ settlement) [per pile]: kN	50·0 [50·0]	700 [58]	600 [50]

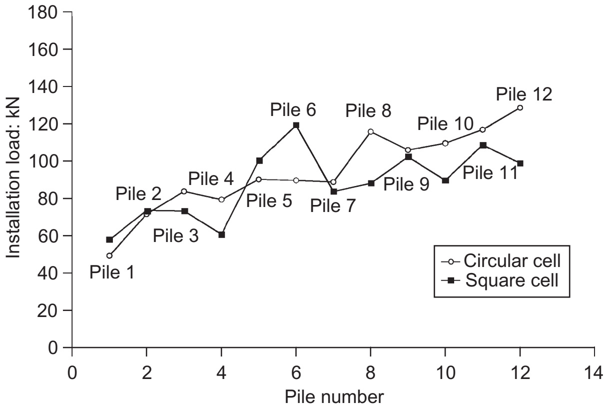

Fig. 4. Installation loads for the circular and square cell foundations

beneath the pile base, which influences the stiffness of the base response. Bored piling involves minimal disturbance of the soil below the base. Hence at the end of the installation there is no significant change in either the stress close to the pile base or the density of the soil. When the pile base is subsequently loaded, the soil is failed for the first time.

In the case of jacked piles, the full static base resistance of the pile is mobilised during installation. As shown earlier, the jacking force during the final installation stroke is comparable to the capacity in a subsequent maintained load test. Hence the soil beneath the pile base is reloaded in the same manner when the pile is load tested after installation. Dynamically driven piles represent an intermediate case, in which the soil is compressed by vibration, in addition to an increase in stress level.

These three soil states beneath a pile base that result from bored, dynamically driven, and jacked installation, can be considered analogous to virgin compression, compaction, and true overconsolidation of a soil element. Coop (1990) shows that the stiffness of truly overconsolidated samples is higher than that of compacted samples that begin at the same specific volume (Fig. 5). This difference can explain the relative stiffnesses of driven and jacked piles. In both cases, during the installation of the piles the soil is subjected to the same amount of compression: that is, sufficient compression to allow for the penetration of the pile into the ground. However, the base load–settlement response of a dynamically driven pile may not be as stiff as that of the jacked pile, because the soil around the pile will have not experienced the same high confining pressure as the soil around a jacked pile. Clearly, this analogy is a simplification, because the soil deformation below a pile base involves shear as well as compression.

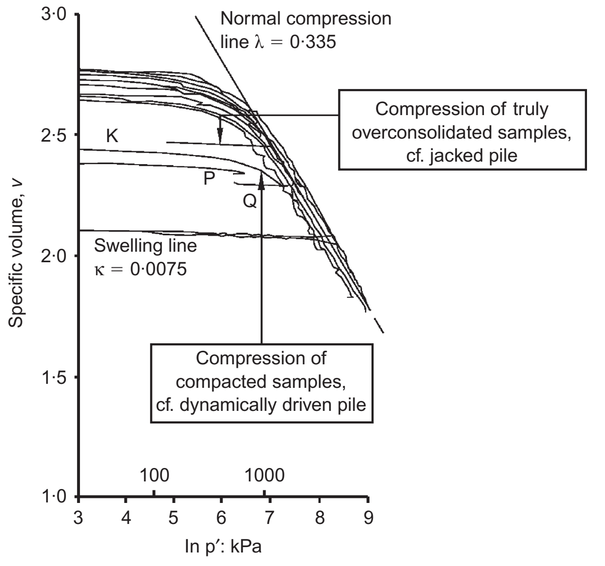

Fig. 5. Isotropic compression data for Dogs Bay Sand (after Coop, 1990)

Load transfer theory

The load–settlement response of an axially loaded pile depends on both the compressibility of the pile and the load transfer t–z curve for the soil around it. The t–z curve represents the integrated effect of shear strains in the soil, leading to a relationship between the shear stress applied at the pile wall and the resulting local displacement of the soil (Randolph, 2003a). The soil continuum is idealised as a number of separate horizontal layers, each with its own load transfer curve. The pile is then idealised as a number of elastic springs connected in series (Fig. 6).

In order to account for non-linearity in the stress–strain response of soil, the load–displacement response of both the pile base and the shaft are taken to be parabolic (Fig. 6). To define the shaft response, only the ultimate unit shaft friction, τ_f, and the initial gradient of the τ–w response are required. This initial gradient, k, is related to the operative soil shear stiffness G, the dimensionless extent of the zone of influence ζ (typically 4), and the pile diameter D, following the elastic solution of Randolph & Wroth (1978)

$$\frac{\tau}{w} = k = \frac{2G}{\zeta D} \tag{2}$$

The pile base is modelled in the same parabolic manner as the shaft. The response is determined by the ultimate unit

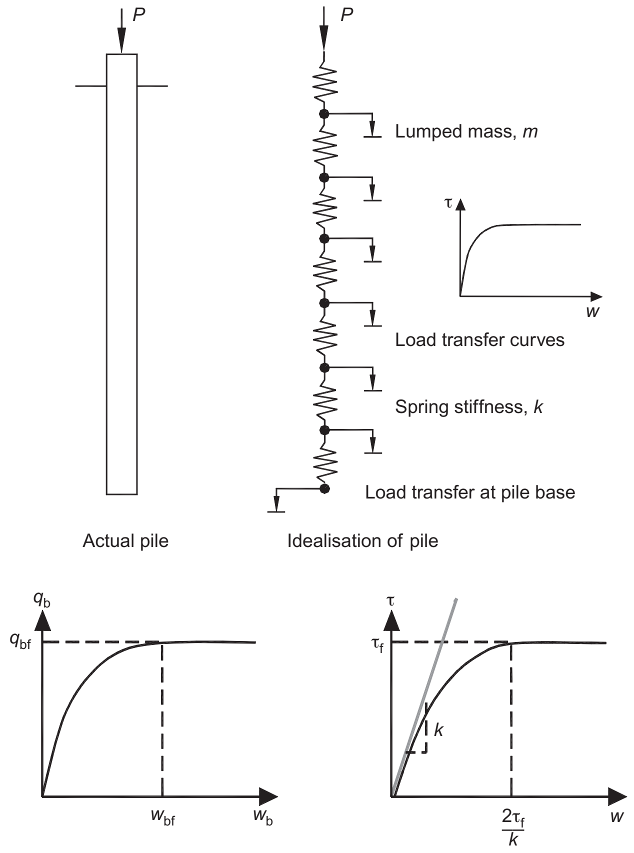

Fig. 6. Idealisation of pile in load-transfer analysis (Randolph, 2003a)

base resistance, q_{bf}, and the settlement required to mobilise this capacity, w_{bf} (Fig. 6).

Load-transfer analysis 1

Three RATZ analyses are presented, with increasing refinement. An initial set of input parameters G, τ_f, w_{bf} and q_b were estimated as follows. The ultimate shaft friction τ_f was estimated following the API (2000) guidelines. The small-strain soil stiffness G_0 was calculated following the Baldi *et al.* (1989) correlation with CPT resistance q_c. The operative shear modulus of the soil G was taken to be half the initial stiffness G_0 to allow for disturbance during installation. These parameters governing shaft resistance were held constant while the ultimate unit base resistance q_{bf} was varied to fit the measured plunging capacity, and the displacement value w_{bf} to mobilise q_{bf} was varied to fit the first part of the load–settlement response.

For analysis 1, the closest fit to the field data was obtained with $q_{bf} = 3050$ kPa at a displacement $w_{bf} = 1$ mm (Fig. 3). The good agreement between the back-analysed and measured responses suggests that the load-transfer analysis with parabolic responses for the base and shaft is realistic. However, the selected value of base settlement is implausibly low. It implies an operative soil stiffness below the base of ~200 MPa, based on the elastic rigid punch solution and the initial slope of the base response, which is eight times greater than the estimated value of G_0: an unlikely value, even allowing for the pre-loading during installation.

Load transfer analysis 2: modified base stiffness

A more appropriate settlement to mobilise full base capacity w_{bf} is 5 mm, which is equal to D/20. This value is in agreement with data from tests on a 100 mm diameter instrumented pile equipped with a base load cell reported by Lehane (1992) and Chow (1996). During testing of this pile at two sand sites, the ultimate base resistance was mobilised at a base settlement of ~5 mm (D/20). A second analysis was conducted using RATZ, with this modified value of w_{bf} (Fig. 7). This revised analysis shows reasonable agreement with the field response at low loads, dominated by the shaft response. However, at higher loads, the back-analysis is too compliant.

Residual base load

Residual base loads locked in during installation are an additional mechanism by which the installation method may influence the pile stiffness. At the end of installation, when the head load is removed, the shaft friction on the upper part of the pile is reversed as the pile head rebounds upwards. As the shaft response is stiffer than the base response, the base does not become fully unloaded during this rebound. As a result, base load acts at the pile tip to preserve the equilibrium of the pile, even though the load at the pile head is reduced to zero (Fig. 8). This residual base load that is locked in during the installation of the pile means that the base load–settlement response does not begin from zero during a subsequent load test. If a high residual load is present, the ultimate capacity is reached at a smaller settlement, as a smaller movement of the pile base is required to fully mobilise the base resistance.

Poulos (1987) suggests that, for driven or jacked piles, the residual base load can be as high as 70–80% of the ultimate base capacity. Randolph (2003b) notes that a lower residual load might be expected for open-ended piles, unless they plug during installation, which is the case for the tests presented in this Technical Note.

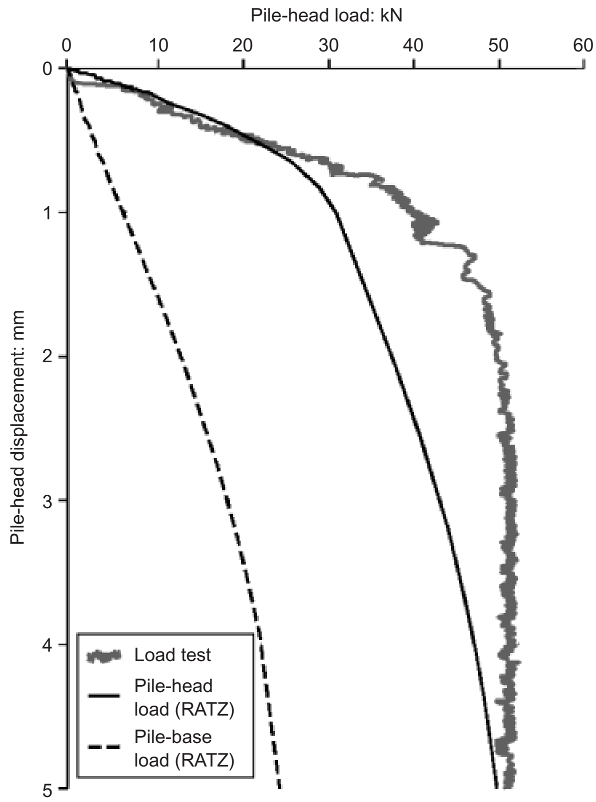

Fig. 7. RATZ back-analysis 2, $w_{bf} = 5$ mm

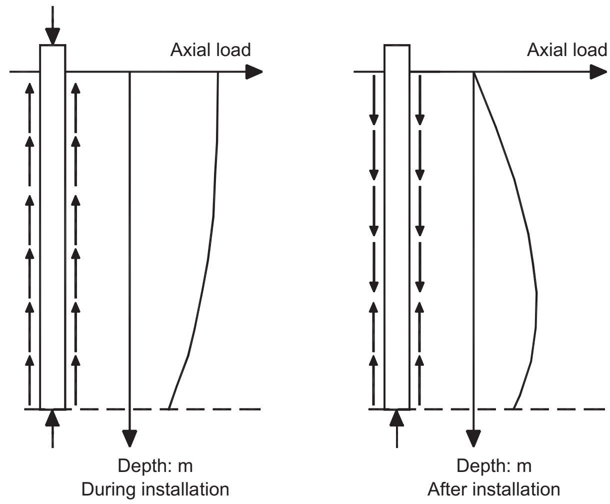

Fig. 8. Residual loads during and after installation

Load transfer analysis 3: with residual base load

A further load transfer analysis was conducted to evaluate the possible influence of residual base load on the load–settlement response of the single test pile. The residual base load, $q_{b,RESIDUAL}$, was calculated within RATZ by unloading from fully mobilised base resistance, and was equal to 2·0 MPa, which is 65% of q_{bf}. This value corresponds to a base load equal to 70% of the ultimate shaft resistance. The load–settlement response of the pile when this residual base load is included shows excellent agreement with the field data (Fig. 9).

Load–settlement response of the cell foundations

The load–settlement response of the cell foundations was investigated using the parameters from RATZ analysis 3 for

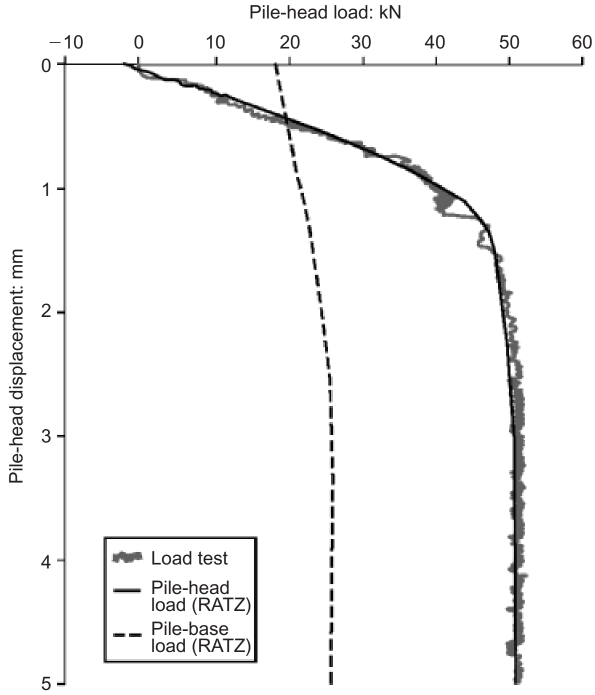

Fig. 9. RATZ back-analysis 3, w_{bf} = 5mm, with residual base load

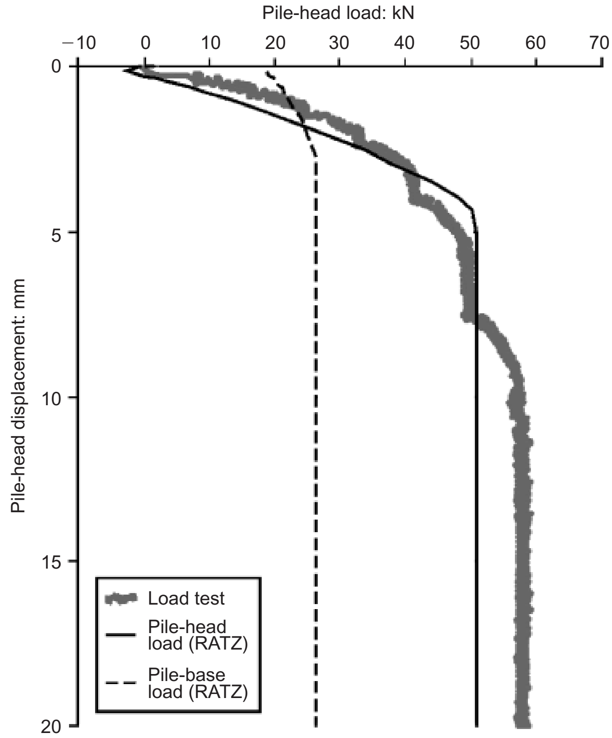

Fig. 10. RATZ analysis, square cell foundation

the single pile, to assess whether this back-analysis could be used to predict the response of a cell foundation without further modification of the input parameters.

In order to account for interaction between neighbouring piles, Randolph (1979) suggests that the compliance of the 'elastic' part of the load transfer curve should be increased to account for the additional settlement created by neighbouring piles. He proposes that the influence zone parameter ζ in equation (2) should be replaced by

$$\zeta^* = \ln\left(\frac{r_m}{r_0}\right) + \sum_i \ln\frac{r_m}{s_i} \tag{3}$$

where $\zeta = \ln(r_m/r_0)$, and is equal to 4 for typical pile geometries; r_m is the maximum radius of influence of the pile (the 'magical' radius); r_0 is the radius of the pile; and s_i is the spacing of the other piles in the group from the pile in question,

The modified influence zone parameter, ζ^*, is calculated by summation over the remaining i piles in the group, which are located at distances s_i from the pile under consideration. This modification has the effect of reducing the stiffness of the 'elastic' part of the load-transfer curve by a factor $R_s = \zeta^*/\zeta$. The 'plastic' part of the curve remains unchanged.

Using this methodology, the group settlement ratio R_s was calculated as 6·6 and 7·6 for the square and circular cell foundations respectively. The resulting load–settlement predictions are shown in Figs 10 and 11. The pile-head load presented in these figures is equal to the total load acting on the group divided by the number of piles in the group. The values for τ_{sf}, G, q_{bf}, w_{bf} and residual base load are as used in RATZ analysis 3 for the single pile (Fig. 9).

Reasonable agreement is shown between the predicted load–settlement curves and the measured response, although a more progressive reduction in stiffness is seen in the field data. In this case, the response from a load test on a single pile is sufficient to make a reasonable, but conservative, estimate of the load–settlement response of a pile group using the group settlement ratio approach described by Randolph (1979). The agreement is notable given the very close spacing of the piles within each cell foundation—far closer than is conventional for pile groups.

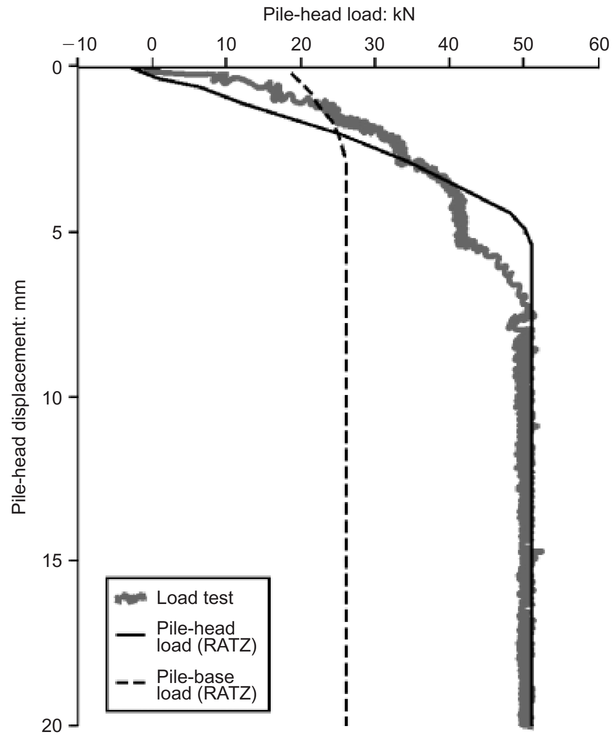

Fig. 11. RATZ analysis, circular cell foundation

CONCLUSIONS

Back-analysis of a field investigation into the performance of jacked piles in sandy conditions is reported. Field tests were conducted and analysed to examine whether conventional design approaches could be applied to jacked piles, particularly when installed in closely spaced groups.

The most notable feature of the maintained load test results was the very high stiffness. Back-analysis using a load transfer approach shows that this response can be linked to (a) the higher stiffness at the pile base due to the preloading applied during the final installation stroke, and b) the presence of residual base load. Excellent agreement was found between the load-transfer analysis and the measured data when considering the residual base load and using a parabolic base load–settlement response requiring 5 mm (*D*/20) to mobilise. This response is significantly stiffer than is found for driven or bored piles.

The parameters used for the back-analysis of the single pile were then applied to the cell foundations, using an elastic method to account for interaction. This proved to be a conservative method for predicting the working settlement of the cell foundations.

These results, if confirmed for a wider range of pile sizes and ground conditions, suggest that there may be potential for the high stiffness of jacked piles to be exploited for more economic foundation design. It may prove possible to apply a lower safety factor than is normally applied to ultimate capacity in order to meet serviceability limits.

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