Measured and predicted t-z curves for a driven single pile in lightly overconsolidated silt

B.A. McCabe
Department of Civil Engineering, National University of Ireland, Galway, Ireland

B.M. Lehane
School of Civil & Resource Engineering, University of Western Australia

ABSTRACT: Data from a static compression load test on a single driven pile in a lightly overconsolidated silt are presented in this paper. The pile was equipped with vibrating wire strain gauges along its length enabling its load distribution and t-z curves to be derived. Strong soil stiffness non-linearity is reflected in the shape of the t-z curves. Predicted t-z curves are developed based upon the stiffness degradation relationship of Lee and Salgado (1999). A comparison of predicted and measured curves allows an estimate to be made of the lateral variation of the mean effective stress in the vicinity of the pile.

1 INTRODUCTION

A programme of full scale load testing on instrumented driven piles has been conducted at a soft silt test site near Belfast in Northern Ireland. Piling research at this site has included a study of the behaviour of closely spaced pile groups (McCabe and Lehane 2006a) and the behaviour of piles subjected to combined axial and lateral loading (Phillips and Lehane 2004). The focus of this paper is on the local shear stress (t) - displacement (z) behaviour along the shaft of a single pile subjected to static loading in axial compression. The field t-z curves were derived from vibrating wire gauge strain measurements incorporated within the concrete pile. Predicted t-z curves based on a well-known stiffness degradation formulation (Lee and Salgado 1999) are used to estimate the lateral variation of equalized mean effective stress in the soil surrounding the pile.

2 GROUND CONDITIONS

The test site is located at Kinnegar, approximately 10km north east of Belfast city centre. The geotechnical profile comprises approximately 7.5m of lightly overconsolidated estuarine organic silt (known locally as *sleech*) with typical properties shown in Table 1. The sleech is overlain by ≈1.0-1.5m of substantially granular made ground and underlain by medium dense sand at ≈8.5m depth. The water table varies both seasonally and tidally between 1.0m and 1.5m below ground level. Detailed characterization of the made ground and *sleech* is reported by McCabe (2002) and Phillips (2002).

Small-strain stiffness values (derived using local Hall effect transducers) from K_o-consolidated undrained triaxial tests on 54mm diameter Geonor piston samples indicated the marked reduction in shear stiffness with strain shown in Figure 1. In this figure, the secant shear stiffness G_{sec} is normalized by the mean effective stress at the commencement of shearing (p_o').

Table 1: Typical properties of the sleech

Clay Friction (%)	20 ±10
Fines Content (%)	90 ± 5
Water Content (%)	60 ±10
Plasticity Index (%)	35 ± 5
Organic Content (%)	11 ± 1
Peak Vane Strength, $s_{u\text{-}vane}$ (kPa)	22 ± 2
Yield Stress Ratio	1.1 to 2.0
Peak friction angle in triaxial comp. (°)	33 ± 1

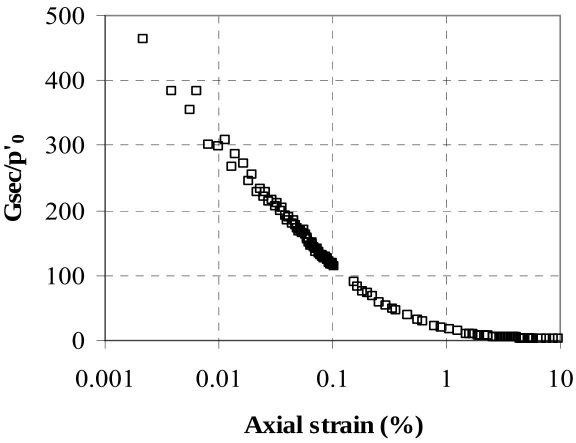

Figure 1: Variation of G_{sec}/p'_o with axial strain (p'_0~30kPa)

3 SINGLE PILE LOAD TEST

The pre-cast concrete pile employed was 250mm (=B) wide and 7m in length. A set of four Soil Instruments acoustic vibrating wire (AVW) type TR55 gauges (one gauge fixed to each of the four T16 reinforcement bars) was incorporated at each of three different levels before the pile was cast. The pile was installed with light driving to an embedment of 6.0m, with the AVW gauge sets located at 2.0m, 4.5m and 5.95m below ground surface (the latter set was located immediately above the pile base). The free-standing pile length was then trimmed to ≈300mm above ground level to facilitate the load testing assembly.

After allowing a period of 82 days for equalization, a maintained-load axial compression pile test was performed using the Kentledge assembly shown in Figure 2. The pile head was equipped with an electronic load cell and displacement transducers. Load was applied in increments and held in each case until the pile head displacement rate fell below a threshold of 0.24mm/hour. The AVW gauge outputs were logged manually from a readout unit at regular intervals throughout loading. The test lasted approximately 6 hours and was terminated at a pile head displacement of 15mm.

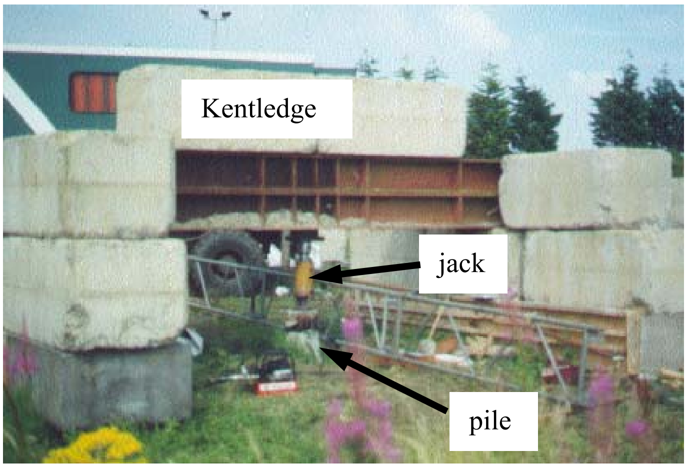

Figure 2: Single pile load test assembly

4 LOAD TEST RESULTS

The compression load displacement behaviour measured at the head of the single pile (designated S1/C in McCabe and Lehane, 2006a) is shown in Figure 3. The dotted portion represents the extrapolation of the curve from the final measured settlement of 15mm to 25mm (i.e. 0.1B, which is a commonly adopted measure of ultimate pile load) using the procedure of Chin (1970).

The load applied at the pile head and the loads deduced from the AVW gauges were used together to derive the load distributions at various stages of loading, which are shown in Figure 4 alongside the corresponding average shear stresses at the intermediate points 1.0m, 3.25m and ≈5.23m. Again, the dotted portions represent the loads and shear stresses which have been extrapolated to a pile head displacement of 25mm. It is apparent, with reference to Table 1, that the maximum local shear stresses (t_{max}) developed fall well short of the peak vane strength; t_{max} is typically 50-60% of both $s_{u\text{-}vane}$ and the triaxial compression undrained strength.

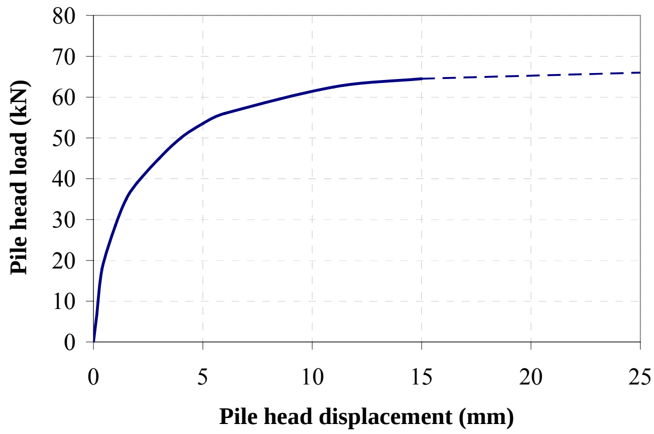

Figure 3: Load displacement response for the single pile

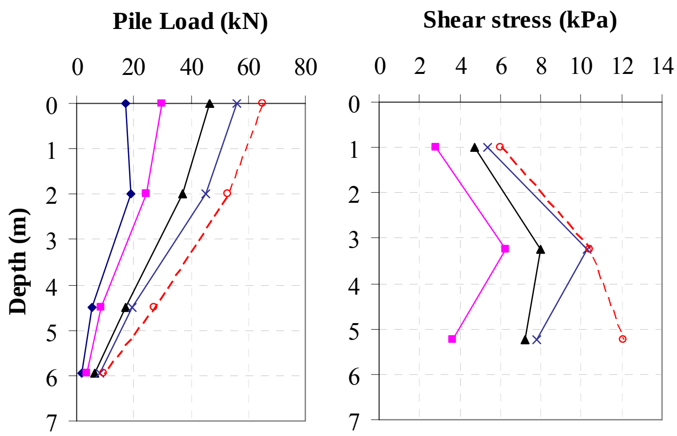

Figure 4: Single pile load distributions and corresponding shear stress profiles

The data shown in Figure 4 were used to deduce curves tracking the mobilization of shear stress at the pile-soil interface (t) as a function of local pile displacement (z) at 1.0m, 3.25m and 5.23m levels. The value of z is determined as the pile head settlement less elastic shortening from the pile head to the level of interest. The curves are presented in the normalized form t/t_{max} in Figure 5. It may be seen that a stiffer response is observed at the higher level (3.25m) within the *sleech* than at the lower level (5.23m). The variability of the upper fill material makes it difficult to put the shape of the t-z curve at 1.0m depth in context of the two within the *sleech*.

It is interesting to note that although the t/t_{max}-z relationship recommended by API RP2A (1993) for clays (also plotted on Figure 5) provides a reasonable 'average' estimate of t-z behaviour for the entire pile length at a typical working load of t $\approx 0.4t_{max}$, it fails to capture the strong non-linearity in the t-z behaviour measured at the pile-soil interface. This non-linearity in the stiffness of the pile is consistent with the well known non-linearity of soil stiffness. In addition, the post-peak reduction in t suggested in the API RP2A (1993) curve is not a feature of the measured data.

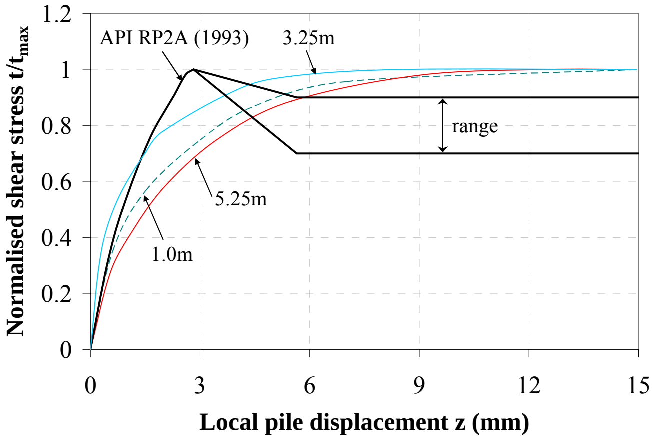

Figure 5: Measured t-z curves with API-RP2A (1993) predictions

5 PREDICTED T-Z CURVES AND LATERAL VARIATION OF MEAN EFFECTIVE STRESS

5.1. *Theoretical basis*

The lateral variation of shear stress (t) in the ground surrounding a pile is assumed to vary inversely with distance from the pile shaft, according to Randolph and Wroth (1978):

$$t = t_{\max}\left(\frac{R}{r}\right) \tag{1}$$

where r is the radial distance from the pile's vertical axis of symmetry and R is the pile radius or equivalent radius (r =R represents the pile soil-interface). G_{sec} is assumed to vary according to the non-linear relationship (Lee and Salgado 1999):

$$G_{\sec} = G_o\left(1 - f\left(\frac{t}{t_{\max}}\right)^g\right)\left(\frac{p'}{p_o'}\right)^n \tag{2}$$

where G_o is the initial elastic shear modulus, f and g are empirical curve fitting parameters, p' is the mean effective stress which has a far-field value of p_0', and n is a constant whose value [0.5-1] depends on shear strain level. Seismic cone tests implied that $G_0 \approx 10{,}000$kPa within the *sleech*, and best fit values of f and g were found by considering the G_o value in conjunction with the triaxial stiffness data provided in Figure 1.

The shear strains (γ) in the soil may be calculated by combining equations (1) and (2) according to:

$$\gamma = \frac{t}{G_{\sec}} \tag{3}$$

The vertical displacements at any location remote from the pile are obtained by integrating the shear strains from that location outwards to a suitable far-field boundary ($r = r_m$):

$$w = \int_r^{r_m} \gamma dr = \sum_r^{r_m} \gamma \Delta r \tag{4}$$

The value of r_m was arbitrarily but conservatively chosen as 200R (=28.2m in this case), representing a horizon beyond which the influence of pile loading is not experienced. The vertical displacement at the pile-soil interface is obtained by setting r =R. Equation (4) is readily implemented in spreadsheet format, using the steps outlined below. It is necessary to state that the solution obtained is based upon reasonable estimates of the soil parameters but is not necessarily unique.

Step 1: The parameters f, g and the lateral variation of p' were varied until the predicted t-z curves produce the best match with measurements in Figure 5. The best fit values of f and g were 0.95 and 0.3 respectively (which also provided a reasonable description of the stiffness degradation seen in triaxial tests).

Step 2: Using an appropriate value for the *sleech's* internal friction angle (Table 1), t_{max} was calculated from:

$$t_{\max} = p'\sin\phi' \tag{5}$$

Equation (5) is consistent with the stress paths measured in a range of isotropically and anisotropically consolidated samples of *sleech* (McCabe 2002).

Step 3: The t-z curves were plotted by varying t in increments up to t_{max}, and computing the corresponding values of z.

5.2 *Lateral variation of mean effective stress*

There is no substantial experimental evidence to suggest how p' might vary laterally once installation excess pore pressures have dissipated (i.e. full equalization has taken place), as required by Step 1.

However, Whittle (1991) presented Class A predictions (i.e. made without advance knowledge of physical measurement) of this p' variation for a single pile in the Carse clay at Bothkennar, Scotland. The Carse clay and the Belfast *sleech* share many geotechnical characteristics. Two different soil models, Modified Cam Clay and MIT-E3 were used. Both models predicted that the stress ratio p'/σ'_{vo}, (referred to as σ'/σ'_{vo} by Whittle 1991 in Figure 6) is approximately constant between the pile shaft (r =R) and a radial distance r ≈4R. A transition portion then leads to a virtually free-field p'/σ'_{vo} value for r>10R. The key difference between the two predictions is that the MCC model indicates that pile installation has caused a permanent increase in p' close to the pile, whereas the MIT-E3 model indicates a permanent reduction in p' in this region (see Figure 6).

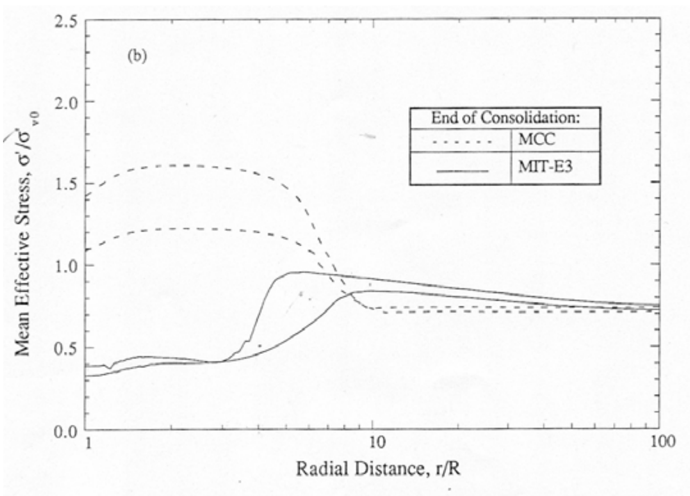

Figure 6: Predicted variation of mean effective stress with radius around a fully equalized pile in Bothkennar clay (after Whittle 1991)

With this discrepancy in mind, three different lateral variations of p' were considered for possible integration into equation (2), and these are shown in Figure 7. Two of these are simplified versions of the two shown in Figure 6, and are labelled 'stiffening' and 'softening'. A third intermediate case is added, labelled 'neutral', in which p' is unchanged with r and everywhere equal to its free-field value p'_0. A lateral earth pressure coefficient (K_o) of 0.63 was used in converting from the p'/σ'_{vo} in Figure 6 to the p'/p'_o in Figure 7. Also, the value of r used is an equivalent pile radius calculated as $B/\pi^{0.5}$.

Figure 8 shows t-z curves predicted using Equation (4) for each of the three options presented in Figure 7. The best match between measured and predicted t-z curves for both the 3.25m and 5.23m levels is given by the 'softening' option. It appears that the 'neutral' and 'stiffening' options produce quite similar t-z curves at both levels, but the 'softening' option produces a distinctly different and more non-linear response from these and provides the closest match to the measurements.

Despite the limitations of this modelling exercise, it can be tentatively concluded that a pile driven in soft clay/silt as exists at Belfast or Bothkennar will cause a long term reduction in p' (from the free-field value) in the vicinity of its shaft. The exact nature of this reduction requires further investigation and should consider other lateral variations of p' than those proposed in Figure 7.

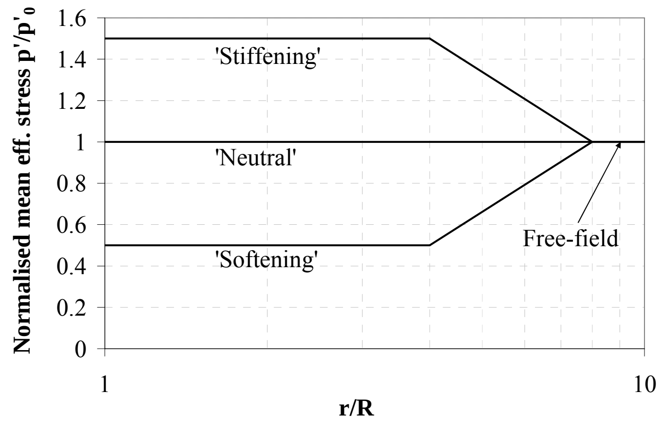

Figure 7: Assumed p' variations for use in equation (2)

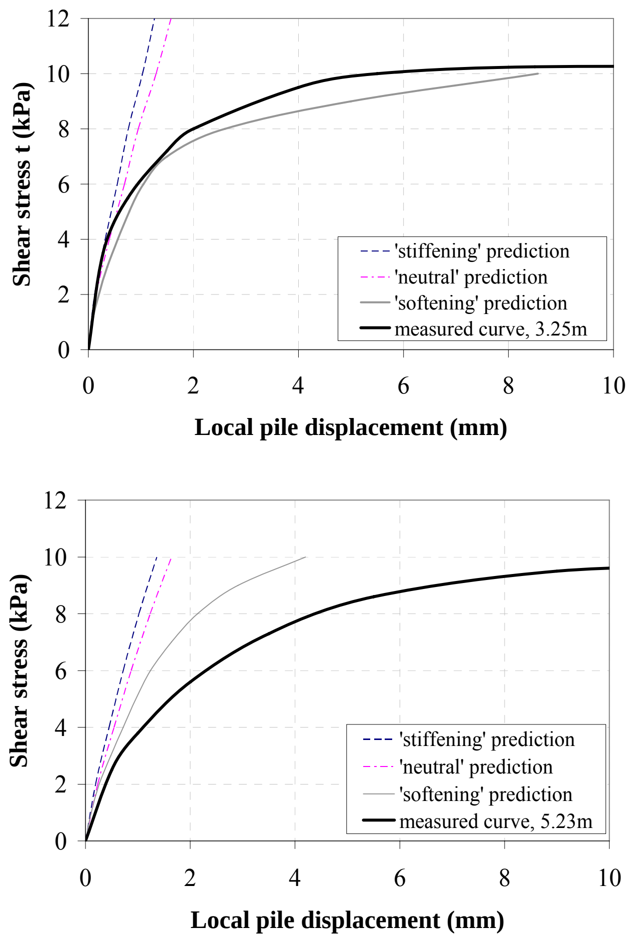

Figure 8: Predicted t-z curves based upon the p' variations of Figure 7; 3.25m above, 5.23m below.

5.3 *Displacement field in the vicinity of the pile*

The predicted displacement trough surrounding the single pile (using the best-fit softening p' shown on Figure 7) is compared in Figure 9 with:

(i) corresponding predictions using the linear elastic PIGLET program (Randolph 2004)

(ii) corresponding predictions using a simplified 2-D finite element analysis in OASYS SAFE using the BRICK non-linear soil model (McCabe and Lehane 2006b)

(iii) physical settlement measurements on the heads on non-loaded piles due to loading of a neighbouring pile (McCabe and Lehane 2006a)

Settlement decay curves are presented in normalized format w/w_{max} (where w_{max} is the settlement at r =R) as a function of r/R. Ranges are shown where some variation with load level and depth was observed. The simple prediction method used in this paper provides a much improved estimate of the settlement trough than the linear elastic approach. It still, however, over-predicts observed distributions and those predicted by McCabe & Lehane (2006b) using the much more computationally intensive finite element method. It is clear, therefore, that while matches to the t-z response at the pile shaft may be achieved, such matches do not necessarily imply that the selected approach and its assumptions provide an adequate model of the real behaviour. The simple approach considered here evidently fails to capture the extent of the stiffness non-linearity of the *sleech*.

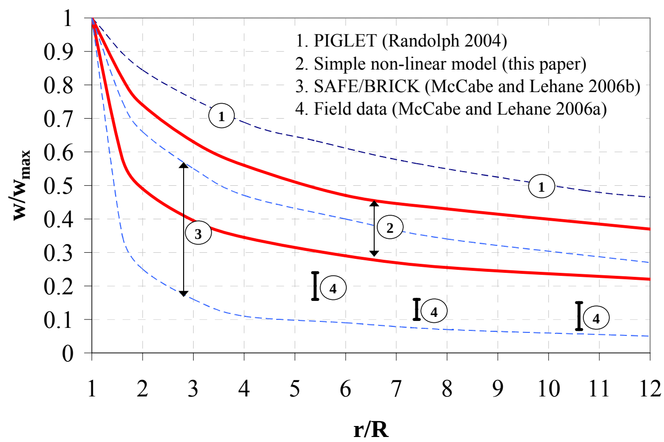

Figure 9: Lateral decay of ground settlement around a single pile

6 CONCLUSIONS

This paper has presented measured load distributions and interpreted t-z relationships at different levels for a friction pile in soft lightly overconsolidated silt. The high degree of stiffness linearity of the t-z curves is consistent with laboratory stiffness measurements but is not captured fully by standard predictive techniques. It is suggested that the effect of pile driving is to cause a long term reduction in p' in the vicinity of the pile (in this low YSR clay), although the exact form of this clearly requires further investigation. The prediction methodology described in this paper provides a better indication of pile behaviour than commonly deployed linear elastic assumptions.

7 REFERENCES

Chin, F.K. 1970 Estimation of the ultimate load of piles not carried to failure, *Proceedings of the 2nd SE Asian Conference on Soil Engineering*, Singapore, 81-92

Lee, J.H. and Salgado, R. 1999. Determination of pile base resistance in sands, *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, Vol. 125, No. 8, 673-683

McCabe, B.A. 2002. Experimental investigation of driven pile group behaviour in Belfast soft clay, *PhD thesis*, University of Dublin, Trinity College

McCabe, B.A. and Lehane, B.M. 2006a. Behaviour of axially loaded pile groups driven in clayey silt, *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, Vol. 132, No. 3, 401-410

McCabe, B.A. and Lehane, B.M. 2006b. Prediction of pile group response using a simplified non-linear finite element model, *Proceedings of the 6th European Conference on Numerical Modelling in Geotechnical Engineering*, Graz, 589-594

Phillips, D.T. 2002 Field tests on single piles subjected to axial and combined axial and lateral loads, *PhD thesis*, University of Dublin, Trinity College

Phillips, D.T. and Lehane, B.M. 2004 The Response of Driven Single Piles Subject to Combined Loads, *Proceedings of the 5th International Conference on Case Histories in Geotechnical Engineering*, New York, USA, CD Paper 1.17

Randolph, M.F. 2004 PIGLET: Analysis and Design of Pile Groups User's Manual, Version 4-3, University of Western Australia

Randolph, M.F. and Wroth, C.P. 1978 An analysis of the deformation of vertically loaded piles, *Journal of Geotechnical Engineering Division*, ASCE Vol. 104, No. 12, 1465-1488.

Whittle, A.J. 1991 Prediction of instrumented pile behaviour at the Bothkennar site, *School of Engineering Report*, Massachusetts Institute of Technology, USA