14 Apr 2004, 4:30 pm - 6:30 pm

The Response of Driven Single Piles Subjected to Combined Loads

D. T. P. Phillips
Waterford Institute of Technology, Ireland

B. M. Lehane
University of Western Australia, Australia

Proceedings: Fifth International Conference on Case Histories in Geotechnical Engineering
New York, NY, April 13-17, 2004

THE RESPONSE OF DRIVEN SINGLE PILES SUBJECTED TO COMBINED LOADS

D. T. P. Phillips
Waterford Institute of Technology
Ireland

B.M. Lehane
University of Western Australia
Australia

ABSTRACT

The behaviour of piles subjected to lateral load has generally been investigated experimentally using free-headed piles with a lateral load applied close to the pile head. In practice, however, there is some degree of restraint at the head of many piles and these are often subjected to a combination of lateral and vertical loads. The case history described in this paper involved full-scale field experiments comprising instrumented precast concrete piles subjected to a range of loading conditions including combined lateral and axial loading with partial rotational restraint at the pile head. The pile instrumentation, which included electrolevels and electrical resistance strain gauges, allowed accurate determination of the lateral soil reaction-lateral displacement (*p-y*) response adjacent to the test piles. This paper concentrates on the analysis and interpretation of the test data for the pile subjected to combined loading. These results are presented in conjunction with test data from an adjacent pile subjected to the same lateral load to allow the difference in behaviour to be evaluated.

INTRODUCTION

The impetus for realistic and economic predictive methods for assessing the lateral capacity of piles stemmed from the growth experienced by the offshore exploration industry in the early 1960's. To address the need for improved methods of analysis, a series of instrumented pile tests were undertaken, principally in the Gulf of Mexico. These tests validated the ability of vertical piles to resist lateral loads and led to the development of a semi-empirical design approach known as the *p-y* method. This method and various other approaches employed in the analysis of laterally loaded piles have been summarised in Elson (1984) while recent advances such as 3-D finite element analysis and the 'strain-wedge' method of Ashour et al. (1998) suggest improvements on the traditional methods of analysis.

Two factors emerge from the foregoing: (i) the success of any method of analysis can only be assessed by comparing the predicted results with results from full-scale field tests and (ii) the field tests conducted as part of the original database involved piles subjected to lateral load alone. In reality however, all piles resist a vertical load of some magnitude prior to being loaded laterally.

This paper extends the existing database by presenting the results from tests on a driven concrete pile subjected to combined loading. The influence of the combined loading is assessed by comparing test results from an adjacent pile tested contemporaneously under the same lateral load.

GEOLOGY AND SOIL PROPERTIES

The tests were conducted at a research site located on the outskirts of Belfast city, Northern Ireland. The geological succession of the drift deposits at the test site comprises glacial till underlying about 8.5m of estuarine clays, silts and sands. The estuarine materials were transported and deposited by the Lagan, Connswater and Blackstaff rivers, all of which confluence into Belfast Lough. The estuarine clays, known locally as *sleech*, were generally laid down on a peat layer and are estimated to be about 8000 years old. The clays underlie most of central Belfast, and have a maximum thickness of about 15m (Crooks and Graham, 1972). They are soft, with an average undrained shear strength of the order of 20kPa, and are lightly overconsolidated. The preconsolidation pressure is typically about 10 to 20 kPa higher than the in-situ vertical effective stress, which is consistent with a fall in water table level of 1m to 2m (Doran, 1992). During the construction of a nearby sewage treatment plant (about 35 years ago), 1m of fill material was placed on top of the estuarine material in the area of the test piles. While the fill can be generally classified as sandy gravel or gravely sand, poorly compacted brick and concrete rubble was contained within the soil matrix in the environs of the test piles. The site stratigraphy is summarised in Fig. 1 and the results from a series of cone penetration, in-situ vane, seismic cone and cone pressuremeter tests conducted in close proximity to the test piles are provided in Fig. 2.

INSTRUMENTATION AND LOAD TEST PROGRAMME

Two 350mm square and 10m long reinforced concrete piles, designated L1 and AL1, were cast with the instrumentation shown in Fig. 3; pile L1 was subjected to lateral load only while pile AL1 was subjected to combined axial and lateral loading. A series of electrical resistance (ERS) and vibrating wire (VW) strain gauges recorded the strain distribution profiles for the piles and displacement transducers measured the pile head movement. A string of electro-levels (ELs) was deployed in each pile to monitor the pile slope as the lateral load was applied and hence enabled independent verification of the displacement profiles determined from the strain data. Finally, a number of total pressure cells (PCs) were incorporated into pile AL1 to record the lateral stress at various levels.

The piles were driven into the deep medium dense sand layer (SPT N value =15) and terminated ≈9.6m below ground level. Two shallow pits approximately 0.5m deep were then excavated from in front of each pile to remove miscellaneous builder's rubble from the fill at these locations (see Fig. 3).

Two weeks prior to applying the axial load to pile AL1, eleven ELs were installed between the two piles: six in pile AL1 and five in pile L1. Each EL was adjusted at the pile head to ensure its output was within the linear calibrated range prior to sliding the EL to the desired position along the pile shaft. The procedure of adjusting and positioning the ELs took approximately 3 hours to complete.

Lateral Loading Arrangement

The lateral loading was achieved by jacking the piles apart; steel collars, placed over each pile were connected in series to a jack, load cell and a rigid steel strut. The arrangement was levelled and its alignment maintained (by temporarily propping off the side excavation) until a small initial load was applied to stabilise the set-up.

The application of vertical and lateral load to pile AL1 necessitated the development of a special detail above the pile head (Fig. 4). The detail consisted of smooth hardened steel bars housed directly beneath the main test beam to provide a 'structural roller'. A pin joint was positioned on top of a steel 'helmet' seated on the pile and a load cell and jack were sandwiched between the pin joint and the roller. The instrumentation cables were connected to the data acquisition system located in a mobile laboratory.

Load Test Procedure

The tests performed as part the research presented here are summarised in Table 1. The axial load of 168kN on pile AL1 had been in place for 24 hours in advance of starting the first combined load test i.e. test CLT1. The loading procedure involved increasing the lateral load in a series of small increments of ≈4.4kN. Each increment was held for a period of four minutes during which period ERS gauges, displacement transducers, load cells, ELs and PCs were logged every thirty seconds. VW gauges were recorded manually every two minutes.

The procedures adopted for CLT2 were identical to those in CLT1 except that the axial load on pile AL1 was reduced by ≈ 20% to 133kN about an hour before starting the test.

Table 1: Chronology of load test programme

Test Series Designation[1]	Test Details	Axial load (kN)	Max Lateral load (kN)
ALT	October 17, 1997, pile AL1	168	-
CLT1	October 18, 1997, pile AL1	168	59.75
LLT1	October 18, 1997: pile L1	0	59.75
CLT2	October 19, 1997: pile AL1	133	89.75
LLT2	October 19, 1997: pile L1	0	89.75
RT	May 18, 1999: piles AL1 & L1	0	74

[1] ALT refers to the Axial Load Test on pile AL1; CLT1 & CLT2 refer to initial and second Combined Load Tests on pile AL1 respectively, LLT1 & LLT2 denotes the initial and second Lateral Load Tests on pile L1 and RT is the Re-Test involving only lateral loads.

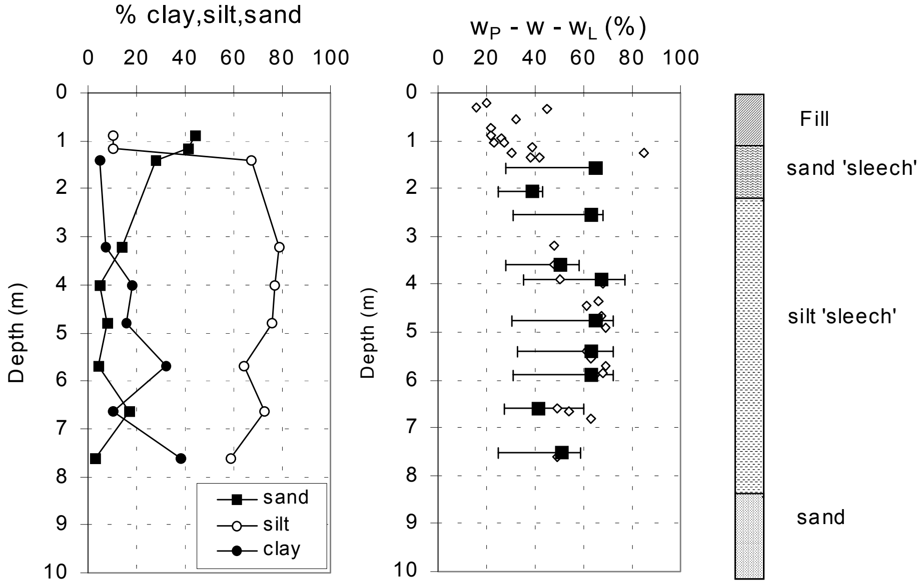

Fig. 1: Site stratigraphy and index properties

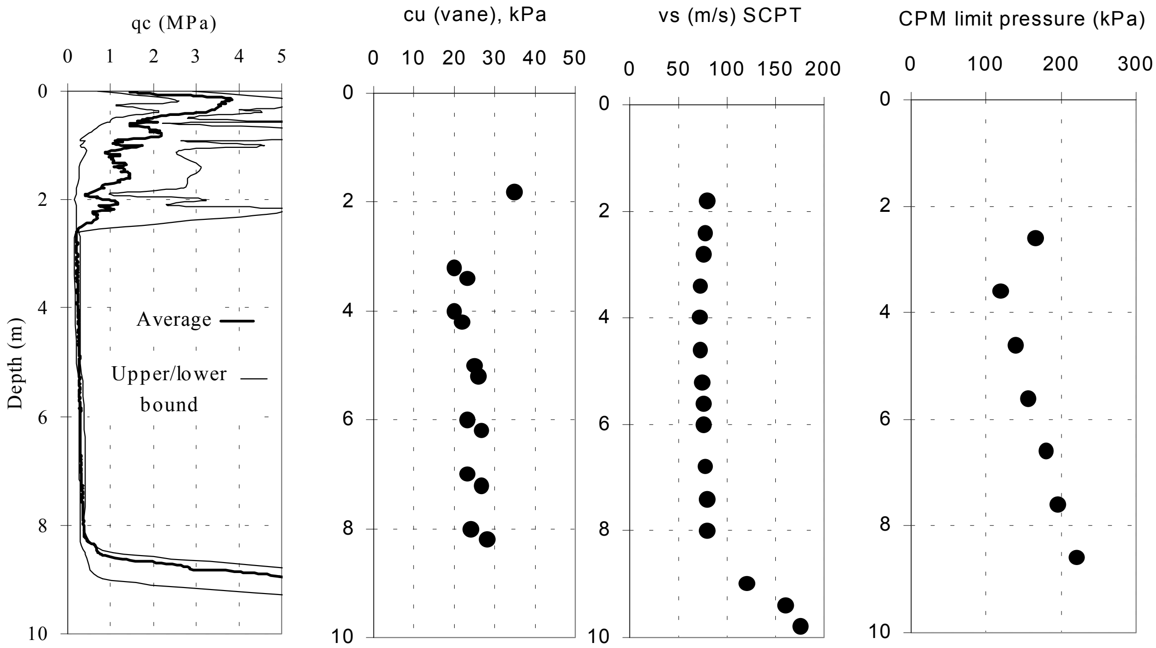

Fig. 2: In-situ test results

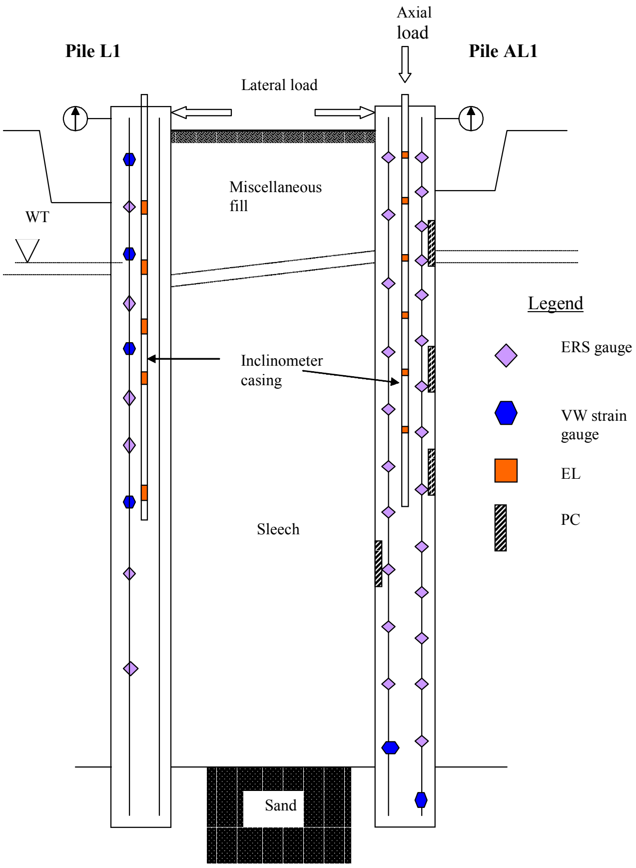

Fig. 3: Schematic illustration of instrumentation

LOAD- PILE HEAD DISPLACEMENT RESULTS

The lateral load-displacement behaviour at the pile heads is shown in Figs. 5 and 6 for the CLT and LLT test series. The results indicate a dramatic difference in the response between the piles particularly for the initial tests (Fig. 5). Pile AL1 exhibited a significantly stiffer response than pile L1 under the same lateral load. The same trend is again observed on re-loading during the second test up to the maximum previous load (Fig. 6). At the higher load levels, however, the displacement of AL1 converges rapidly and eventually exceeds that measured at L1. The explanation for such behaviour is discussed in detail in the next section.

ANALYSIS OF PILE SUBJECTED TO COMBINED LOADING

The test configuration at pile AL1 inadvertently provided a degree of restraint at the pile head; this was apparent from the large differences in the pile head displacements measured for each pile shown in Figs 5 & 6. To quantify the restraint, it is first necessary to understand the loading mechanism at the pile head. Figure 7(a) schematically illustrates the test configuration while Fig. 7(b) shows the resolved forces acting on the displaced pile. The behaviour of AL1 can be explained through a combination of field observations (e.g. level surveying of the test beam) and instrumentation data recorded during the CLTs. These data are discussed in Phillips (2002) and confirm that translation of the rollers beneath the test beam did not occur during the load tests. Instead, the test beam moved with the pile in a series of steps corresponding to each load increment applied up to 59.75kN in CLT1. As the lateral loads were increased above this level in CLT2, the pin joint started to rotate noticeably.

Fig. 4: Test setup at pile AL1

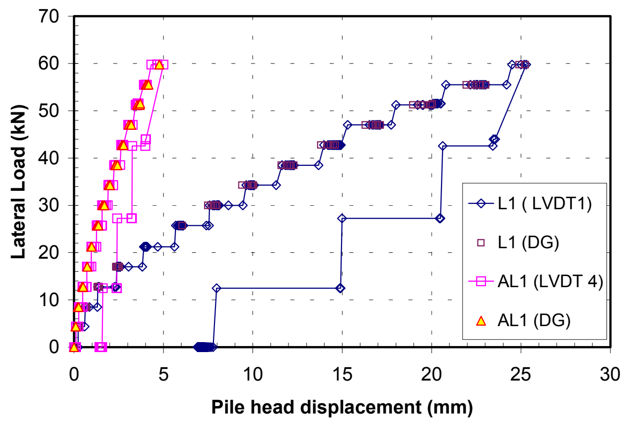

Fig. 5: Load-Pile head displacement during first time loading (i.e. CLT1 & LLT1)

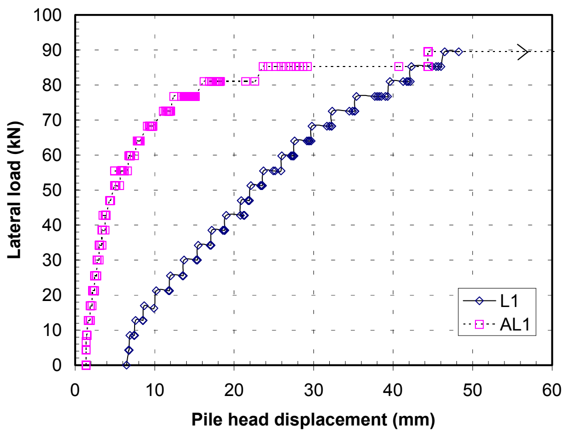

Fig. 6: Load-Pile head displacement during loading to failure (i.e. CLT2 & LLT2)

It was therefore concluded that a frictional force was mobilized between the 'rollers' and the test beam during the application of lateral loads. The mobilised frictional force, *h* shown in Fig. 7(a) had the effect of reducing the lateral load applied to AL1. Furthermore, as the frictional force was transferred across the pin joint, it created a restraining moment at the point of lateral load application. These additional load effects were due to the test setup and clarify why, during CLT1 and much of CLT2, AL1 experienced smaller bending moments and displacements than pile L1.

To quantify these effects, the shear (H_R) and applied moment ($M_{applied}$) at the point of lateral loading were back calculated using the bending moments inferred from strain gauges located above pit level and the slope measured by an electro-level (EL) located 240mm below the level of the applied lateral load (Fig.8). If it is assumed that the EL slope θ is representative of the pile slope at the point of lateral loading, then the displacement at the pin joint Δ can be estimated as;

$$\Delta = \delta + (\theta)(l_a - x) \qquad (1)$$

where δ = the displacement measured at the LVDT
$(l_a - x)$ = distance from the LVDT to the pin joint
θ = slope in radians

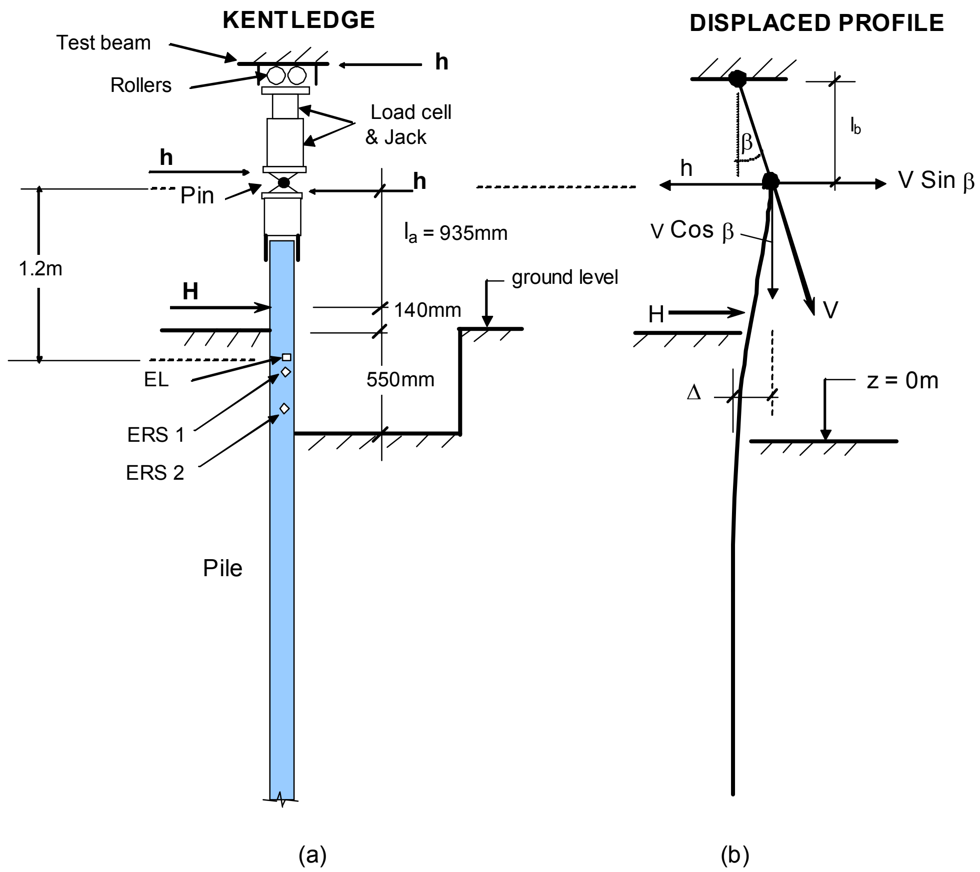

Fig. 7: Analysis of pile head restraint at pile AL1

The strain at any ERS gauge above the pit level (z=0m) can be used in conjunction with the measured moment-strain relationship (which was verified using non-linear 2-D finite element analysis for the reinforced concrete section; see Phillips, 2002) to determine the bending moment ($M_{measured}$) in the pile at that point. Therefore, having determined Δ and hence β (= $tan^{-1}\{\Delta/ l_b\}$), $M_{measured}$ at the ERS gauge level can be equated with $M_{applied}$ at the same level thus enabling h to be back calculated as follows:

$$h = [H(d) + V\sin\beta(e) + V\cos\beta(\Delta) - M_{measured}]/ e \quad (2)$$

where
- h = frictional force acting in the opposite direction to H
- H = applied lateral load
- V = the axial load measured by the load cell
- d = distance from point of lateral loading to $M_{measured}$
- $V \sin\beta$ = the horizontal component of the kentledge load due to pin joint rotation
- e = distance from pin joint to $M_{measured}$ (see Fig.8)
- Δ = displacement of pin joint relative to EL

The horizontal component of the angled vertical load, $V \sin \beta$ (Fig. 7b) was initially small but grew in magnitude as the joint rotation increased. The effect of the increasing joint rotation (β) on the magnitude of the restraining moment can be seen in Fig. 9. It is noteworthy that rotation at the pin joint causes the initial restraining (negative) moment to undergo a change in direction at $\beta \approx 0.5°$. This trend continues and the moment eventually becomes positive (i.e. an additional moment acting on the pile) once the pin joint rotation exceeds $\approx 2.6°$ (see also Fig. 11).

The accuracy of the frictional force given by (2) was checked using data from a second ERS gauge (ERS 2, see Fig. 8) also located above pit level. The results from this gauge predicted the frictional force, h within 5% of the value predicted by the first gauge thus confirming the validity of the structural model. The average value for h calculated from the two ERS gauges was used in the subsequent analysis of the data for pile AL1.

Therefore, having determined h, the resultant horizontal load, H_R applied to AL1 is given by the algebraic sum of the horizontal forces as shown in (3)

$$H_R = H - h + V \sin \beta \quad (3)$$

Hence, at ERS 1 for example, $M_{applied}$ can be calculated from the following:

$$M_{applied} = (H_R)(e - l_a) + (V \cos \beta)(\Delta) \quad (4)$$

H_R and $M_{applied}$ are used to fit the bending moments at the pile head with the measured bending moment profiles (inferred from the strain gauges) for pile AL1. The influence of the load test setup on the variation in H_R, h and $M_{applied}$ with the applied lateral load H is shown in Fig. 10 and Fig. 11 for CLT1 and CLT2 respectively.

Two points are worth noting from Fig. 11:

1. At loads above H = 47kN the rate of gain in H_R increases and was found to surpass the applied lateral load in the final load increment. The increasing horizontal component due to the rotation of the axial load is the reason for this occurrence.

2. The magnitude of $M_{applied}$ was significantly less than that measured during CLT1 (Fig. 10) at the same load levels. Moreover, above lateral loads of H = 47kN the value of $M_{applied}$ started to reduce for subsequent loads and ultimately changed sign over the final load increment. This change of sign coincides with the increase in H_R.

<u>Bending Moment Profiles</u>

Typical bending moment profiles derived for pile AL1 and L1 are shown in Fig. 12 and 13 respectively. The following observations are noteworthy:

<u>Pile AL1 (see Fig. 12).</u>

- The shape of the bending moment diagram for pile AL1 is similar to that for pile L1. However, due to the test setup, pile AL1 was subjected to a restraining moment and a reduced lateral shear force at the pile head.

- The overall magnitude of the ‘free’ bending moment (positive + negative) at the point of maximum moment is slightly less than the maximum moment measured in L1 at the same load. The difference is due to the smaller shear force applied to pile AL1.

- Much smaller negative bending moments at depth occur compared to L1, presumably because of the application of a restraining moment at the pile head. Moreover, the application of a restraining moment at the pile head results in a re-distribution of the ‘free’ moment between the pile head and the pile shaft, thus resulting in a more economic use of the pile section.

- The bending moment profiles shown in Fig. 12 indicate that the increase in bending moment is proportional to the applied load, suggesting that elastic conditions prevail in pile AL1 throughout CLT1.

Pile L1 (see Fig. 13).

- The depth to the maximum moment recorded by pile L1 did not appear to vary with load increment. The maximum moment occurred at a depth of ≈1.2m below the pit level (z=0). Matlock 1970, Reese and Welch 1975 and Briaud et al., 1984 have found that the depth to the maximum moment tends to increase, as the lateral load is incremented upwards. This was due to yielding in the soil close to ground level and the subsequent transfer of the excess stress to soil at greater depth. The results from the tests presented in this paper show that significant yielding of the upper stiff layer did not occur at the loads applied during LLT1.

- It can be seen that the maximum pile moment at a lateral load of 25.75kN is close to the calculated cracking moment M_{cr} (33kNm). This provides a marked contrast to the bending moments for pile AL1 (shown in Fig. 12) which remains uncracked at much higher lateral loads. The axial load and pile head restraint discussed above are the reasons for the difference in moment profiles between the piles

- The bending moment profiles illustrate that the behaviour of laterally loaded piles is dictated by the soil response within the top 6 pile diameters below ground level.

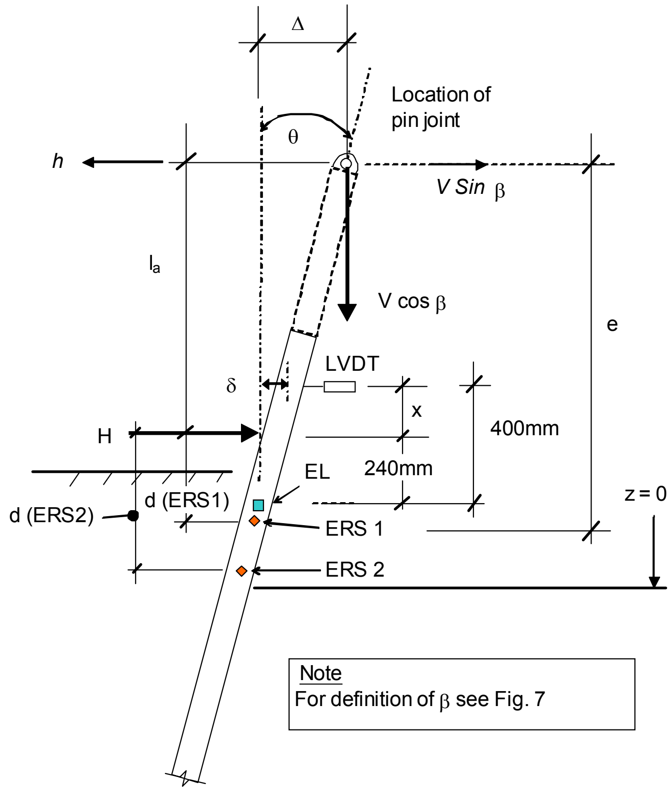

Fig. 8: Geometry used to calculate pin joint displacement Δ

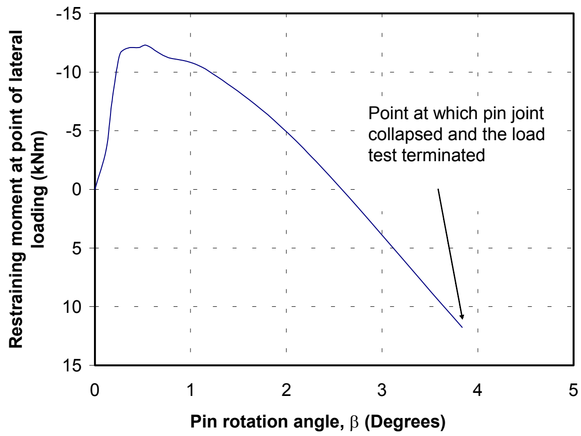

Fig 9: Variation in pile head restraining moment with pin joint rotation

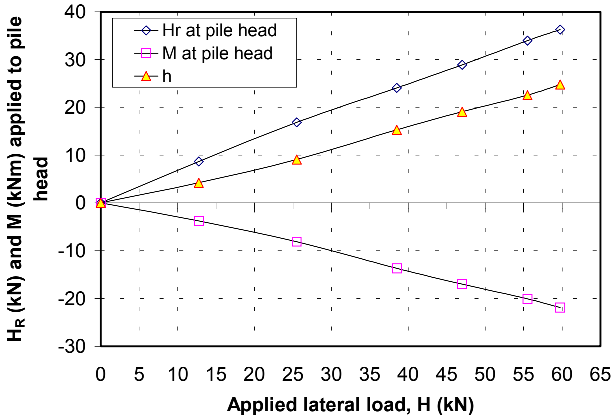

Fig 10: Resultant shear and moment at pile head of AL1 (CLT1)

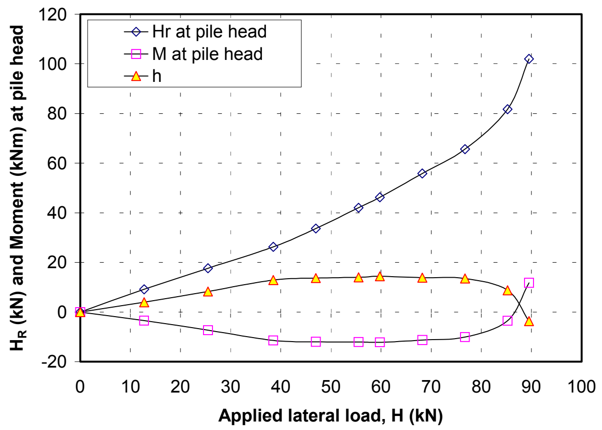

Fig 11: Resultant shear and moment at pile head of AL1 (CLT2)

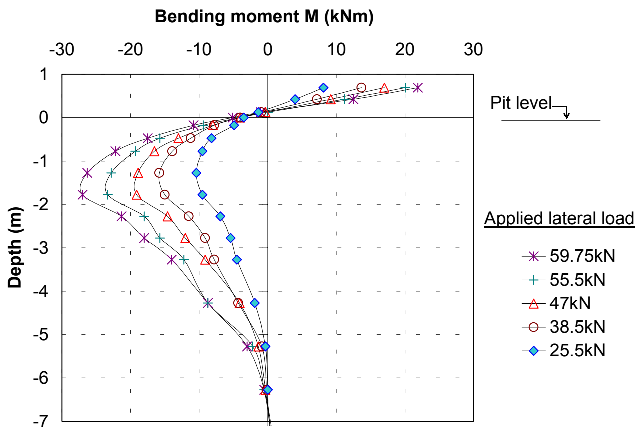

Fig. 12: Bending moment profile for AL1 during CLT1

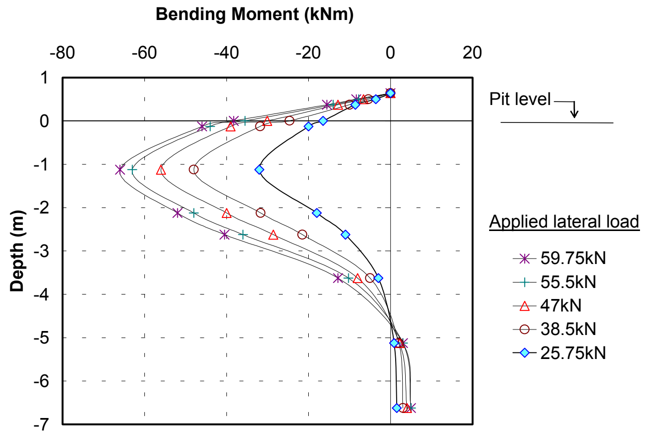

Fig. 13: Bending moment profile for L1 during LLT1

Displacement Profiles

The displacement profiles were determined from the EL data by fitting a fifth order polynomial combined with an exponential term to the measured slopes and integrating the equation to obtain the displaced profiles. Details of this procedure are provided in Phillips (2002). The accuracy of the results can be judged against the closeness of the EL displacement profile to the measured pile head displacement shown in Figs. 14 and 15 for pile L1 and AL1 respectively.

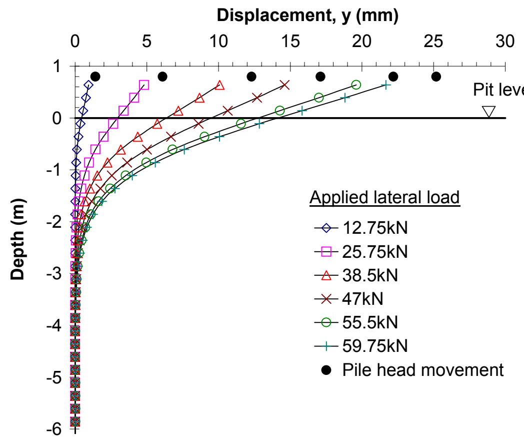

Fig. 14: Displacement profiles fitted to measured slopes for pile L1 (LLT 1)

The accuracy of the profiles derived by the ELs was verified by deducing displacement profiles from the bending moment profiles. Again, smooth curves were fitted to the inferred bending moments, M (see Phillips, 2002). The double integral of the M/EI profile (where EI is the pile's flexural rigidity) yielded the displacement profile. This exercise was performed for each load increment and a typical result is shown for pile L1 in Fig. 16.

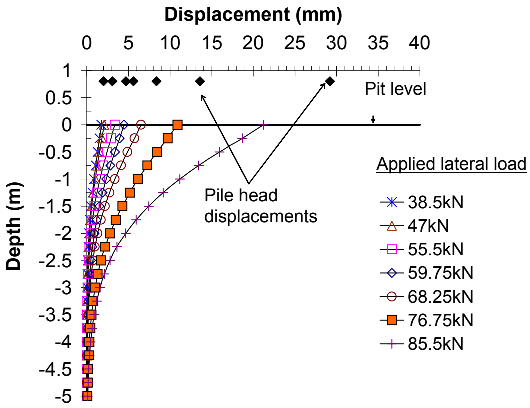

Fig. 15: Displacement profiles fitted to measured slopes for pile AL1 (CLT 2)

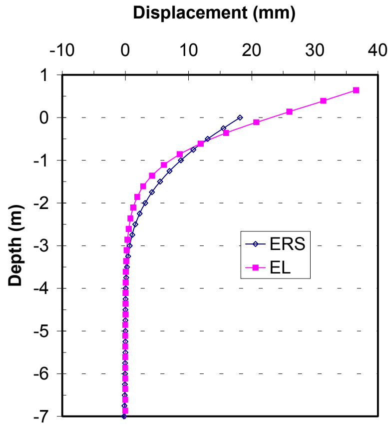

Fig. 16: Comparison of displacement profiles for pile L1 at 76.75kN (LLT2)

CONCLUSIONS

1. This case history involved the use of a mechanism to facilitate application of a combined vertical and lateral load to a pile. During design of the experiments, it had been the intent that this mechanism would provide negligible restraint to the pile head. Conclusions drawn from the experiments based on this assumption would have, however, been grossly in error. Fortunately, the relatively comprehensive instrumentation employed allowed the restraint to be quantified and for proper interpretation of the experiments.
2. The relative insensitivity of the 'free bending moments' on the pile head condition indicated that the soil resistance was not affected by the presence of a pile axial load or the pile head restraint condition.
3. The thin layer of stiff soil at shallow depth had a marked influence on the lateral pile response.

ACKNOWLEDGEMENTS

The authors would like to acknowledge Lowry Piling (Ireland) Ltd. for giving their time and expertise to make this research possible. The permission of the Waterworks Department of Belfast for the use of the site for research purposes is gratefully acknowledged.

REFERENCES

Ashour, M., Norris, G. and Pilling, P. (1998). *Lateral loading of a pile in layered soil using the strain wedge model*, J. of Geotechnical and Geoenvironmental Engineering, American Society of Civil Engineers, Vol. 124, No.4, pp. 303-315.

Briaud, J. L., Smith, T. and Meyer B. (1984). *Laterally loaded piles and the pressuremeter: comparison of existing methods*, American Society for Testing and Materials, West Conshohocken, Pa. pp 97-111.

Crooks, J.H.A. and Graham, J. (1972). *Stress-strain properties of Belfast estuarine clay.* Engineering Geology, 6, pp. 275-288.

Doran, I. G. (1992). *Soils of Northern Ireland.* The Structural Engineer, Vol. 70, No. 77, pp. 135-138.

Elson, W. K., (1984), *Design of laterally loaded piles*, CIRIA Report 103, London.

Matlock, H. (1970). *Correlations for the design of laterally loaded piles in soft clay*, Proc. 2nd Annual Offshore Technology Conference, V.1204, pp. 577-594.

Phillips, D. T. P. (2002). *Field tests on single piles subjected to lateral and combined axial and lateral loads,* Ph.D. Thesis, University of Dublin (Trinity College), Dublin.

Reese, L. C. and Welch, R. C. (1975). *Lateral loading of deep foundations in stiff clay*, J. Geotechnical Engineering Division, American Society of Civil Engineer, Vol. 101 No. GT7, pp. 633-649.