

THE INSTITUTION OF ENGINEERS OF IRELAND

DRIVEN PILES IN BOULDER CLAY-DESIGN AND PRACTICE

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Paper presented to a joint meeting of the Civil division and Geotechnical society of the Institution of Engineers of Ireland, on the 6 November 1995.




SYNOPSIS

This paper reviews current design practices for driven piles in boulder clays and presents the findings of an instrumented pile test programme carried out at the Croke Park site during its redevelopment in 1994. The findings of the pile test programme are discussed and are shown to give an improved understanding of the behaviour of piles in boulder clay; some commonly held design assumptions are also challenged. The paper also comments on the reliability of dynamic pile load testing.

1.0 INTRODUCTION


Driven piles are frequently the most economical foundations for large structures in the Dublin area. While these piles are used extensively in Dublin's Boulder Clay and have generally performed satisfactorily, recent experiences, such as those encountered during the Tallaght town centre development, have highlighted our lack of detailed knowledge regarding their behaviour. It is in the light of such difficulties that a research programme on the behaviour of driven piles in Boulder Clay was initiated at Trinity College. This programme, which was partly sponsored by Murphy International, included the driving of a comprehensively instrumented pile in Croke Park in 1994.

This Paper firstly reviews current design procedures for driven piles in Ireland and, in this context, also examines the capabilities of dynamic pile load testing. Current design practices are empirical and very much reliant on Contractors' experiences. Such a seemingly unsophisticated approach stems from the view of many engineers that the factors controlling the behaviour and capacity of a pile are too complex and consequently cannot be predicted reliably. The experimental investigations, which are subsequently described in this Paper, were aimed at improving our understanding of driven piles in Dublin Boulder Clay by physically measuring the distribution of axial load and water pressure along a pile's length. The measurements are shown to shed further light on some of the more important factors affecting axial pile capacity and also challenge some commonly held design assumptions.



Contractor	Pile Dimension (mm).	Typical Working Loads (kN)	Pile Impact Energy (kN m)	Set Criteria (Blows/ 25mm)
Murphy International Ltd.	250 x 250	600	14	20-25
	300 x 300	1000	14	
	350 x 350	1500	17.5	
Lowry McKinney Ltd.	250 x 250	600 - 800	45	10
	285 x 285	800 - 1000		
Taranto Ltd.	235 x 235	650	18	15
	275 x 275	1000	25	23
F-K Piling Ltd	275 x 275	1000	30	12
Pierse Bachy Ltd.	275 x 275	750-1000	25	25-30
	350 x 350	1200 - 1500		

Table 1 Pile dimensions, working loads and set criteria used by Irish contractors



2.0 TRADITIONAL PILE DESIGN

Precast concrete piles in Dublin are normally driven to a 'design set' to achieve the required working load. This set is derived using dynamic formulae with the benefit of experience from previous similar contracts. Pile load tests are generally, but not always, used to verify the selected design set. Experience has shown that the capacity of driven piles is generally in excess of that determined using traditional static design approaches based on the undrained shear strength of the clay. This section of the paper discusses the current pile driving practice in Ireland and collates some of the experience of pile load tests on driven piles in Dublin boulder clay. Comparisons with observed capacities are then compared with those estimated from traditional static design methods.

2.1 Pile driving practice

A survey of Irish piling contractors carried out for this paper showed that the dynamic driving 'Hiley' or 'Engineering News' formulae were most commonly used to determine the 'set' for a required pile

capacity. Approximately 80% of the piling contractors contacted indicated that their pile design was carried out using these dynamic driving formulae.

All dynamic driving formulae relate the ultimate load capacity to the pile set and are based on the principle that the resistance of piles to further penetration under working loads has a direct relationship to their resistance to the impact of the hammer when they are driven. These formulae are, however, sensitive to assumptions regarding the efficiency of the impact and the magnitude of elastic compression of the hammer, pile and soil system. Studies have shown (e.g. see England et al. 1994) that the driving formulae can be unreliable, although the experience with their use for driven precast piles in Dublin appears to be satisfactory. The success of such design approaches has very often depended on the subjective experience of the engineer using them. Table 1 presents typical sets used to achieve working loads on the piles currently available from local contractors in Ireland.

The typical stratigraphy in the Dublin area would be made ground over

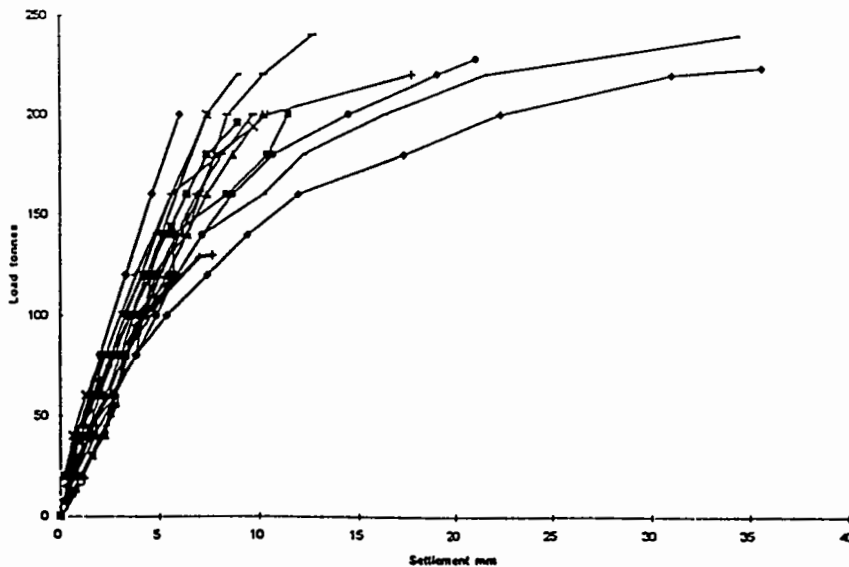


Figure 1 Load/ Settlement Curves for Tallaght and Croke Park.

brown boulder clay over black boulder clay. A review of driving records from two projects, the Tallaght Town Centre project and the Croke Park project, indicated that the required set is normally achieved with a penetration of 2m to 4m into the black boulder clay.

2.2 Traditional pile load tests

Farrell et al (1995) studied the results of traditional pile load tests carried out at the Tallaght Town Centre and Croke Park projects. A maintained loading schedule (to BS 8004) was generally used in these tests, although a modified version of this schedule was used on some piles at the Tallaght site (Armishaw et al 1992). The load/ settlement curves of all the piles tested fall within a narrow band as can be seen from Figure 1. (The results of the pile test which failed at the Tallaght site is excluded). This figure offers a useful reference for assessing the expected performance of driven piles in boulder clay in Dublin. These results

also show that the boulder clay is capable of supporting significant pile loads.

2.3 Traditional static design

The traditional static design approach for calculating the ultimate capacity of piles in clay is the total stress (c_u) method. In this method the shaft capacity is estimated using the average undrained shear strength c_u of the clay together with an adhesion factor ' α ', while the base capacity is predicted using the undrained strength c_u at the pile tip and the bearing capacity factor N_c . The following relationship is used (A_s and A_b being the shaft and base areas respectively):

$$Q_u = \alpha c_u A_s + N_c c_u A_b$$

The adhesion factor α is normally taken as 0.45 and Skempton's N_c value of 9 is usually used. The undrained shear strength of the black boulder clay at the Croke Park site was estimated to be about

450kPa. Consequently the ultimate load on a 300mm by 300mm pile driven 3m into this stratum would be about 1100 kN (giving a working load of about 440 kN using a FoS of 2.5). This ultimate load is considerably less than the capacity indicated by the pile load tests. The traditional method also suggests that 70% of the load would be taken by skin friction and that the skin friction is constant along the embedded length of the pile.

3.0 DYNAMIC LOAD TESTING

The search for rapid and low cost pile tests has led to the development of dynamic load testing. In such tests dynamic stress wave data obtained during pile driving or restrikes are used as a means of estimating the long term capacity of the pile. Once the data have been calibrated, wave equation analysis (such as the CASE (Goble 1987) analysis which uses a wave equation in conjunction with empirically determined correlation factors) can be employed to determine the ultimate static capacity of the pile from dynamic properties of the pile-soil system during driving. A CAPWAP (Goble 1987) analysis can be carried out to predict the distribution of skin friction and end bearing and to predict pile settlements at working loads.

The result of a dynamic pile test carried out on a 250mm square pile at the Tallaght site for Eolas (now Forbairt) is shown on Figure 2. Unfortunately the findings of the test must be qualified as the energy impacted to the pile was not sufficient to mobilise the ultimate soil resistance. The results do, however, show that the dynamic method predicted that most of the

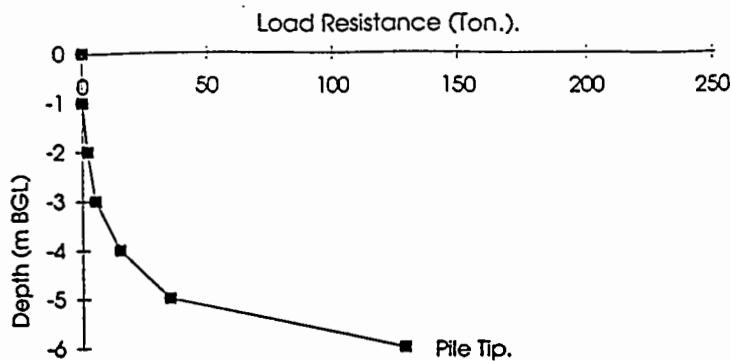


Figure 2 Resistance distribution as predicted by Dynamic test at Tallaght.

load capacity would be provided by the pile in end bearing. The predicted load settlement curve from this dynamic test also compares favourably with the test results shown on Figure 1.

The capability of dynamic testing to accurately predict the ultimate capacity of piles has received mixed reviews in the past, (Argiolas 1992, Goble 1987, Randolph 1987). Some authors observed good correlations between static and dynamic tests, while others found large discrepancies. At the 4th International Stresswave conference (1992), load predictions were made by engineers for five precast driven piles using dynamic testing techniques. Predictions for one pile, which had a static load capacity of 340 kN, varied from between as low as 90kN to up to 510kN. Better (but still not acceptable) predictions were made for the four other piles.

Both static and dynamic tests were carried out on the same piles as part of the Croke Park, TCD Residence and Georges Dock 2 piling contracts. A comparison of the measured static capacities with those interpreted from dynamic tests is shown on Table 2. The capacity, as predicted by the CASE

(Goble 1987) analysis, is very sensitive to the correlation factors (assumed damping factor etc.) employed in the analysis. For example, at Croke Park a change of the Damping factor (J_d) from 1 to 0.7 resulted in a 15% variation in the prediction. The agreement between the static capacities and dynamic predictions was generally quite good. It should be noted that in order for a dynamic pile load test to allow a prediction of the ultimate load capacity, the piling plant available on site must induce a permanent displacement great enough to mobilise all the available friction and base resistance. The low dynamic prediction in Croke park may therefore be partly attributed to insufficient energy being impacted to the pile.

4.0 CROKE PARK PILE TEST PROGRAMME

4.1 Test Objectives

The instrumented pile tests at Croke Park were designed to allow an assessment of current design practice for driven piles in Dublin Boulder Clay and had three primary objectives:

(i) to determine the distribution of shear stresses and pore pressures acting on the pile shaft at various levels of applied axial load and subsequently to compare these with presently assumed distributions.

(ii) to investigate the effect of set-up time (i.e. the time between installation and load testing) on the shaft and base capacities. This investigation was intended to clarify issues such as (a) why piles at Tallaght had apparently lost their set on re-driving and (b) what effect the time allowed between driving and a static load test would have on interpreted pile capacities.

(iii) to compare the pile behaviour under compression and tension loading (the distribution of shaft friction is currently assumed to be identical).

Piling Contract	Pile Dimension (mm)	Working Load (kN)	Static Test Ultim. Load Prediction (kN)	Dynam. Test Ultim. Load Prediction (kN)	Case, Damping Factor, J_d
Croke Park	300 x 300	800	=2900	1730 2030	1 0.7
TCD Resid.	300 x 300	900	=3100	2532	1
Geog. Dock 2	285 x 285	1000	=3400	2911	.7

Table 2

Comparison of Ultimate capacities predicted by dynamic and static pile tests

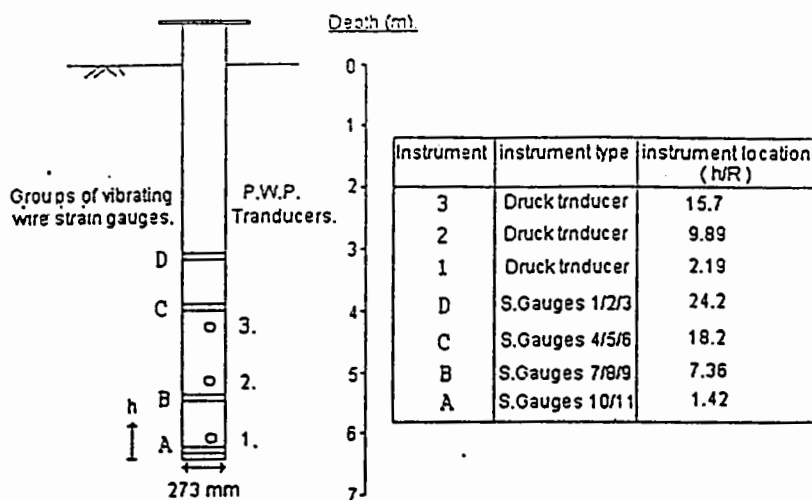


Figure 3 TCD Pile Configuration

4.2 Pile description

The Trinity College pile was a 273mm outside diameter steel tubular pile, welded together in segments to give a total length of 9m. The pile wall thickness was 10mm and a flat 20mm thick steel plate was welded to the pile base. The lower 3.5m of the pile was instrumented with both vibrating wire strain gauges and pore pressure probes. The pile configuration is illustrated in Figure 3. The position of instrumentation relative to the pile tip normalised by the pile radius is also shown. The lower 3.3m of the pile was cut into two 1.2m sections and a 0.9m section in order to instrument and calibrate the pile. These sections were subsequently re-welded together. The cables from the instrumentation passed up the centre of the pile and through a pre-drilled hole at the top of the pile for connection to a portable measuring unit.

4.2.1 Vibrating wire Instrumentation

Twelve vibrating (V.W.) wire strain gauges, three at four different levels, were installed on the inside surface of the pile. VW gauges were selected because they are robust, stable in the long term (i.e. would not suffer from drift) and are unaffected by lead length. The gauges were positioned at 120° from each other around the pile circumference at each of the four levels. This allowed both axial and bending stress to be monitored during static load testing. The gauges were installed by arc welding the side faces of the mounting block to the inside surface of the pile and were then encapsulated in resin to minimize damage due to the shock waves imported during pile driving.

4.2.2 Pile calibration

Calibration of the surface mounted vibrating wire strain gauges was carried out using a loading apparatus with a compressive capacity of 250kN. The pile was

initially loaded and unloaded to 225kN to seat the gauges and reduce the effect of hysteresis. The pile was then loaded and unloaded again to 225kN in intervals of 25kN. At each interval the period reading indicated on the strain measuring unit was recorded.

Mean curves were constructed for each of the four levels of strain gauges by calculating the mean of the three curves at each level. The percentage difference between the theoretical and calculated gauge factors ranged from 1.5% to 8.4%, on average being less than 3%. There was therefore good agreement between the estimated and the recommended gauge factor.

4.2.3 Performance of Vibrating wire Instrumentation

The fact that a zero load at the pile base was predicted by the three VW gauges near the pile base in the tension test indicated that they did not experience a significant zero shift during installation or any significant drift during the testing programme.

4.2.4 Pore water pressure probe

Pore water pressure probes, which were positioned at three levels over the lower 2.5m of the pile, took the form of specially milled plastic mounting blocks, housing resistance pressure transducers, with sintered stainless steel porous stones positioned at the front face of the block. The gap between the porous stone and the face of the transducer was saturated using de-aired silicone oil. To help prevent desaturation, these porous stones were stored in de-aired silicone oil up to minutes before driving commenced and only then were they

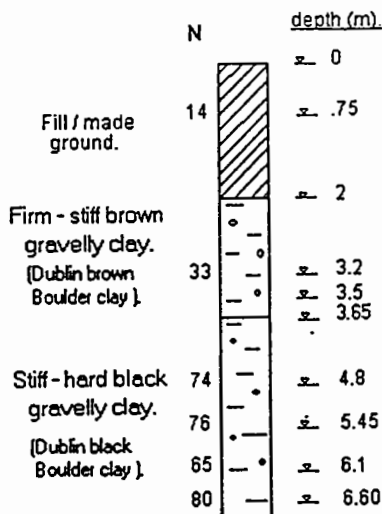


Figure 4 Soil Stratigraphy at Croke Park.

removed and placed in the mounting block in the pile wall.

It should be noted that silicone oil, which has a viscosity approximately twenty five times greater than that of water was employed to help avoid cavitation problems. The oil was de-aired by placing it in a vacuum chamber for a period of twenty four hours.

4.3 Site stratigraphy

In addition to the site investigation carried out for the Croke Park redevelopment, to aid the interpretation of results, one borehole and three dynamic probe tests were carried out within a radius of 2m of the test pile location,

The boring/probe tests from this investigation revealed a stratigraphy (See Figure 4) comprising 2m of fill overlying 1.65 m of stiff gravelly brown boulder clay with some clayey gravel underlain by hard black gravelly clay containing cobbles (black boulder clay). This layer was underlain by a dense angular gravel stratum which extended to limestone bedrock at a

depth of about 20m below ground level.

Standard Penetration Tests carried out in the TCD borehole gave N-values of 33 for the brown boulder clay, while N - values for the black boulder clay ranged between 68 and 80. No systematic variations with depth were recorded in the N - values in the black boulder clay. The properties of Dublin boulder clay are described in further detail by Farrell et al 1995.

4.4 Test programme

The pile was initially driven using a Banut four tonne hammer (fall height of 300mm) to a depth of 6m and after 1 hour was re-driven to a final depth of 6.4m with a set of 12 drops/25mm. This relatively low set (compared with Murphy International's standard value of 20-25 drops/25mm) was selected to ensure that it was possible, with the available kentledge of 1400 kN, to reach the ultimate shaft capacity of the pile during the subsequent load tests.

The load testing programme adopted, which is summarised in Table 3, was structured so that the pile was compression load tested before full dissipation of the pore water pressures generated during driving had taken place i.e. Tests 1C and 2C. Two compression tests (3C & 4C) and a tension test (5C)

were subsequently performed after all excess pore water pressures had effectively disappeared.

Note that T_w is the intervening time between pile driving and load testing.

5.0 RESULTS SUMMARY FROM CROKE PARK

5.1 Pore Pressures after driving

The measurements of the water pressures at the pile/soil interface during and shortly after driving revealed interesting trends. During driving, pore water pressures along the pile shaft were found to be quite low and often negative (i.e. below atmospheric pressure). This response is typical of overconsolidated clays which attempt to dilate during shear.

Once driving (and hence shearing) had ceased, pressures increased rapidly and reached values of up to 700kPa. As can be seen from Figure 5, these pressures subsequently dissipated at an ever decreasing rate and reached values within 5% of hydrostatic pressures (≈ 30 kPa) in 1 week.

5.2 Pore pressures during load testing

Pore pressures measured at the pile shaft showed a tendency to reduce as loading continued in tests 1C and

TEST NUMBER	LOAD TEST	T_w	MAX LOAD (kN)
1C	Compression	2.5 hours	700
2C	Compression	44 hours	950
3C	Compression	17 days	920
4C	Compression	17 days	1350
5T	Tension	24 days	430

Table 3 Summary of testing program

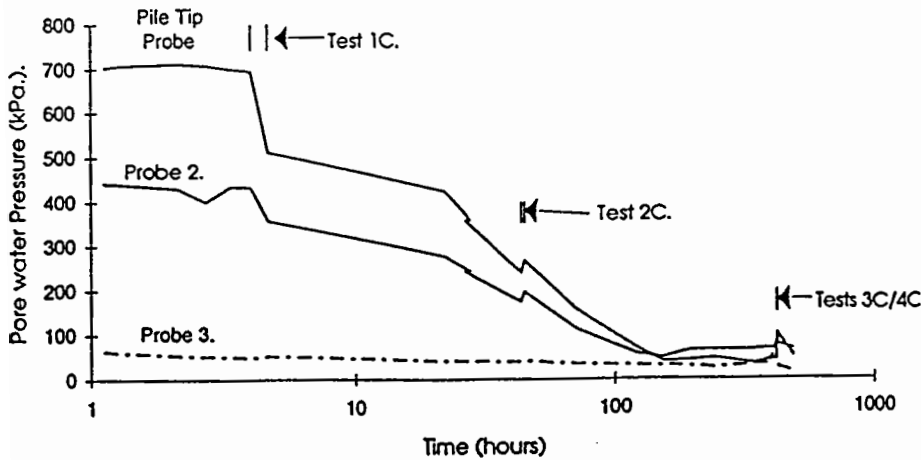


Figure 5 Pore water pressure dissipation After pile installation

in the final two compression tests. This response is attributed to the strong tendency of the black boulder clay to dilate when in a partially re-consolidated state.

5.3 Load distribution in compression tests

The load distribution curves measured during load test 1C are shown on Figure 6. Although this test was conducted when excess pore pressures in the ground surrounding the pile were very high, the curves shown are typical of the distributions measured in all the compression load tests. It is evident that:

(i) A residual base load of $\approx 50\text{kN}$ existed before application of the pile head load. This load is in equilibrium with shear stresses acting downwards on the pile shaft.

(ii) About 60% of the total pile capacity was provided by the pile base in this and all other compression tests. Table 4 summarises the relative contributions of the shaft and base to the ultimate capacity measured in each test.

(iii) The gradients of the load distribution curves indicate that load transfer from the shaft to the ground increased with depth.

(iv) There is negligible load transfer to the ground over the upper 3.7m length of the pile

indicating, perhaps surprisingly, the very low shearing resistance offered by the fill and brown boulder clay to the pile capacity. At these depths, such low resistance may also be partly attributed to the creation of an oversized hole during the driving process.

Observations (iii) and (iv) are in keeping with those of recent research (Lehane 1995) which found the contribution to shaft capacity of any soil layer surrounding the shaft decreases with increasing distance of that soil layer from the pile tip - and is not as is normally assumed a direct function of the depth to that particular soil layer.

5.4 Load distribution in tension tests

The ultimate tensile capacity of the

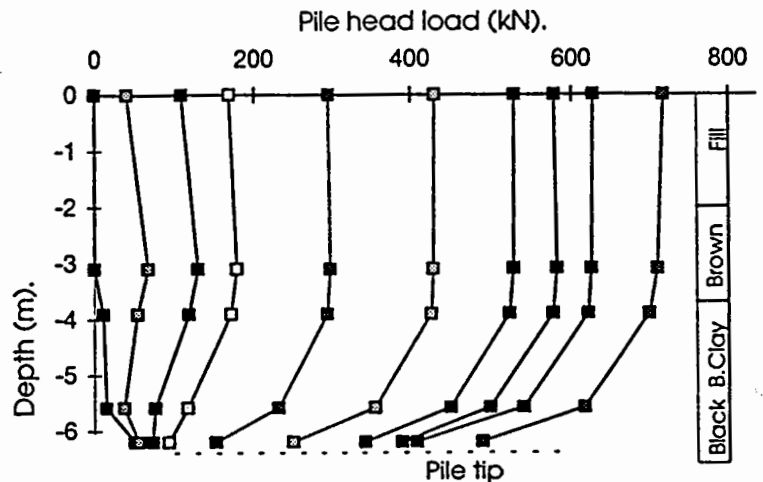


Figure 6 Load distribution in load test 1c.

TEST	1C.	2C.	3C.	4C	5T
Test type	Comp.	Comp.	Comp.	Comp.	Tension
Depth to pile tip (m)	6.4	6.4	6.4	6.4	6.4
Maximum head load (kN)	720	944	936	1350	450
Displacement at max. load (mm)	15	10	4.5	7.5	14
Max. shaft load (kN)	279	336	342	546	-450
Base load % of max. load	61.25 %	64.4 %	63.4 %	59.55 %	
α -value ($C_u = 450\text{kPa}$)	0.22	0.25		0.48	0.4

Table 4 Summary of load tests carried out at Croke park

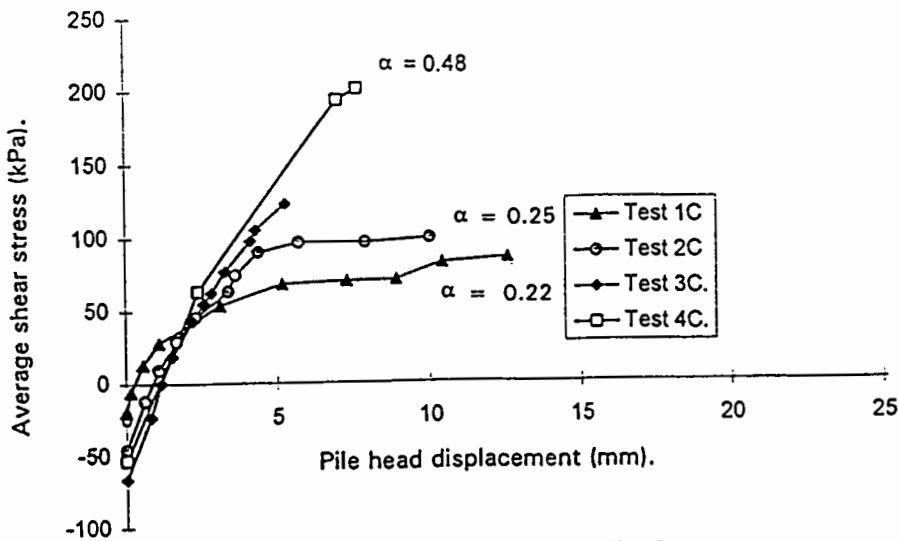


Figure 7 Average shaft friction mobilised in load tests 1c, 2c, 3c & 4c.

pile (410kN) was 70% of the ultimate shaft capacity of the pile measured in the final compression load test. Most notably, over 75% of this tensile capacity was supplied by the bottom metre of the pile shaft.

5.5 Average shear stresses on the pile shaft

The variations with pile head displacement of the average shaft friction mobilised over the lower 3.5 metre of pile shaft are shown on Figure 7 for the four compression tests performed. All curves are similar up to a shear stress of 50kPa, but their shapes at higher levels of applied load are constrained by the maximum shaft capacity that could be mobilised. Shaft capacities were evidently fully developed in tests 1C and 2C after a pile head displacement of about 3.5mm whereas displacements of over 5mm were required to mobilise the shaft capacity in tests 3C and 4C.

5.6 Increase in shaft capacity with time

One of the most striking features of the Croke Park data was the strong dependence of the measured ultimate shear stress on the time interval between installation and load testing. For example, the shaft capacity after 17 days is more than double the capacity after 2 hours. Previous research by Imperial

College and other institutes, which involved measurement of both radial total stresses (σ_r) in addition to pore pressure (u) confirmed the validity of the Coulomb equation relating the available shearing resistance (τ_r) to the radial effective stress (σ'_r) viz:

$$\begin{aligned} \tau_r &= (\sigma_r - u) \tan \delta \\ &= \sigma'_r \tan \delta \end{aligned}$$

where δ is the angle of interface friction. (Soil-steel laboratory shear box interface tests suggested that the δ value in operation at Croke Park was $\approx 30^\circ$)

The increase in shaft capacity with time at Croke Park can therefore be seen to be consistent with the reduction in pore pressures observed i.e. as pore pressures reduce, radial effective stresses acting on the pile shaft increase ($\sigma'_r = \sigma_r - u$) and hence shaft capacity increases.

The increase observed is shown on Figure 8 in terms of an increase in

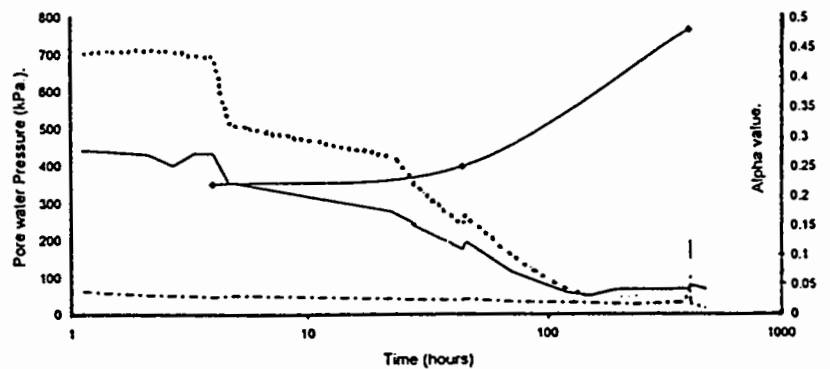


Figure 8 Increase in α -value with pore water pressure dissipation after pile installation

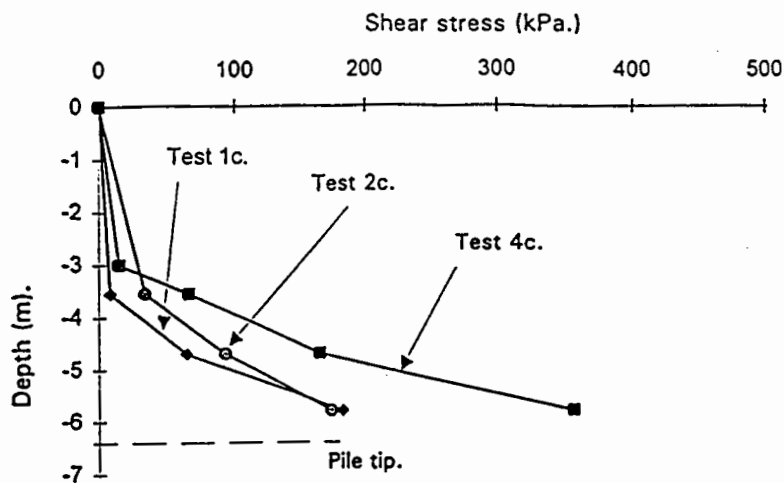


Figure 9 Shear stress distributions for maximum applied load in tests 1c, 2c & 4c.

the α -value (taking $c_u = 450\text{kPa}$) measured in compression tests; the corresponding pore pressures observed are also shown. The α -values can be seen to increase with dissipation of pore pressure starting from a value of 0.22 two hours after pile driving and reaching a maximum final value of 0.48. It is worthy of note that the increase in capacity is not directly proportional to the rate of reduction in pore pressure. Such a phenomenon arises due to the alteration in the properties of the clay adjacent to the shaft as reconsolidation takes place mirrored for example by the different pore pressure responses discussed in Section 5.2.

5.7 Local shear stresses

The distribution of local ultimate shear stresses (as interpreted from the gradients of load distributions) measured in all four compression load tests are shown on Figure 9. It is evident that, although increasing with depth, they showed no direct correlation with the undrained strength of the clay or SPT N-

values. This observation highlights the problems associated with prediction of shear stresses using simple empirical approaches such as the α method.

5.8 End bearing

The variation of end bearing with pile head base displacement measured during the four compression tests is shown in Figure 10. The measured unit ultimate resistance varied from 8MPa in load test 1C to well in excess of 14MPa in load test 4C. This implies that the bearing capacity factor, N_c , lies in the range

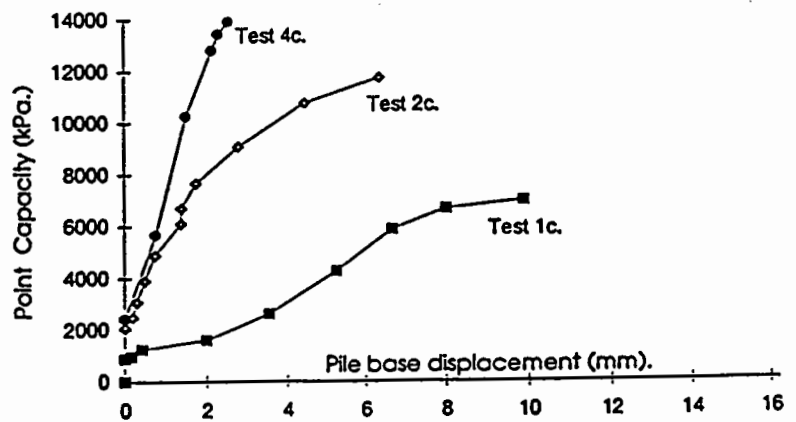


Figure 10 End Bearing recorded in load tests 1c, 2c & 4c.

of 17.5 to over 31 for an undrained strength of 450kPa; this range is significantly higher than the conventionally assumed value of 9.

As with the shaft friction, there was a large increase in base capacity as the excess pore pressures generated during driving dissipated.

6.0 CONCLUSIONS

Traditional design methods such as the dynamic pile driving formulae or static undrained strength approaches are incapable of accurately predicting the ultimate capacity of driven piles consistently. Pile design presently relies heavily on the experience of the engineer and static load testing of piles.

Dynamic testing is quick and relatively cheap to perform and, as a consequence a large portion of the piles in a foundation may be tested-providing increased confidence as a whole. The predictions of dynamic testing were found to correlate well with static load tests on the same pile. However past experience has shown that dynamic tests should not be used as a substitute for traditional static load tests.

The instrumented pile tests at Croke Park have significantly improved our understanding of the behaviour of driven piles in Dublin Boulder Clay and have challenged some of the commonly held assumptions regarding pile design in this material.

Some of the more important observations from this research are:

1. Pile driving induces large excess pore pressures which remain at high levels for some time after installation.
2. Both the shaft and base capacities measured in load tests are critically dependent on the excess pore pressures present. Both increase significantly with time as dissipation takes place.
3. Pile end bearing contributes to a major part of the pile resistance and, at Croke Park, was least four times higher than the conventional bearing capacity design method would predict.
4. Perhaps fortuitously, the average value of α measured at Croke Park of 0.48 is very close to the conventionally adopted value of 0.45. However, shaft frictions do not reflect undrained strength/SPT N profiles and indicate α values significantly higher than 0.45 near to the pile base.
5. Tensile ultimate shaft capacities are only $\approx 70\%$ of compression shaft capacities.
6. Further carefully planned instrumented pile tests would enhance over current knowledge and improve on confidence in design.

ACKNOWLEDGMENTS

The Authors are particularly grateful to Murphy International Ltd. and The Irish American Partnership for sponsoring the Croke Park Instrumented pile tests and also to the Department of Civil, Structural and Environmental Engineering, Trinity College. The authors also wish to acknowledge the assistance of Irish Geotechnical Services and John Sisks and Sons.

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