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Foundation engineering for the UK's new national stadium at Wembley

La construction des fondations du nouveau stade national de Wembley en Royaume-Uni

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ABSTRACT
This paper describes some of the geotechnical issues associated with the foundation engineering for the UK's new Wembley stadium. Predicted and observed vertical pile capacity and load-deformation behaviour of horizontally loaded piles are compared, together with observations and predictions of settlement with depth and time below a deep area of fill. Careful assessment of depth dependant variations of strength, Young's modulus and permeability, and the variation of Young's modulus with mobilised strain are necessary for realistic predictions to be made.

RÉSUMÉ
La note technique suivante décrit quelques aspects de la dimensionnement et de la construction des fondations du stade de Wembley en Royaume-Uni. Des comparaisons ont été fait entre les capacités verticales des pieux et leurs déformations sous chargement horizontale prévus et réels, ainsi que des études de la variation du tassement en dessous d'une couche épaisse du remblai. Des analyses détaillées de la variation avec profondeur de la résistance du sol, du module de Young et de la perméabilité, et de la variation du module de Young avec la contrainte mobilisé sont neccessaires pour faire des prévisions fiables.

1 INTRODUCTION

The UK's new national stadium at Wembley will be one of the largest football stadiums in the world. Geotechnically the design and construction of the stadium has provided several challenges, including:

- demolition of the old stadium and major earthworks for the new stadium, involving excavations up to about 7m deep (to create the new pitch) and fill (using excavation arisings) up to about 10m deep
- a variety of embedded retaining walls (cantilever, propped and anchored) up to 13m high, some backfilled with high plasticity clay fill
- construction of 4000 piles, pile diameter from 0.45m to 1.5m and pile lengths up to 40m. Some piles are subject to global ground movements associated with adjacent earthworks
- pile foundations subject to complex combinations of vertical, horizontal, moment and torsion loads

Due to limited space constraints for this paper the ground investigations will be briefly summarised, followed by a summary of the results from vertical and lateral pile load tests, and initial ground deformation monitoring data. This data will be compared with the design predictions.

2 SITE DESCRIPTION AND GROUND INVESTIGATIONS

The site is located in North London, UK, at the location of the old Wembley Stadium. The stadium is situated on a small hill. Across the stadium footprint the original ground surface levels vary between about 42m OD and 53m OD. There is a railway cutting about 13m deep situated to the south of the stadium. The geology is relatively simple comprising London Clay, beneath made ground of varying thickness, over the Lambeth Group and then Chalk. Overall the London Clay varies in thickness between about 30m and 40m, the Lambeth Group being encountered at between 10m and 15m OD, the Chalk at about -3m OD. The upper 8m to 10m is weathered "brown" London Clay, with the unweathered "blue" London Clay below.

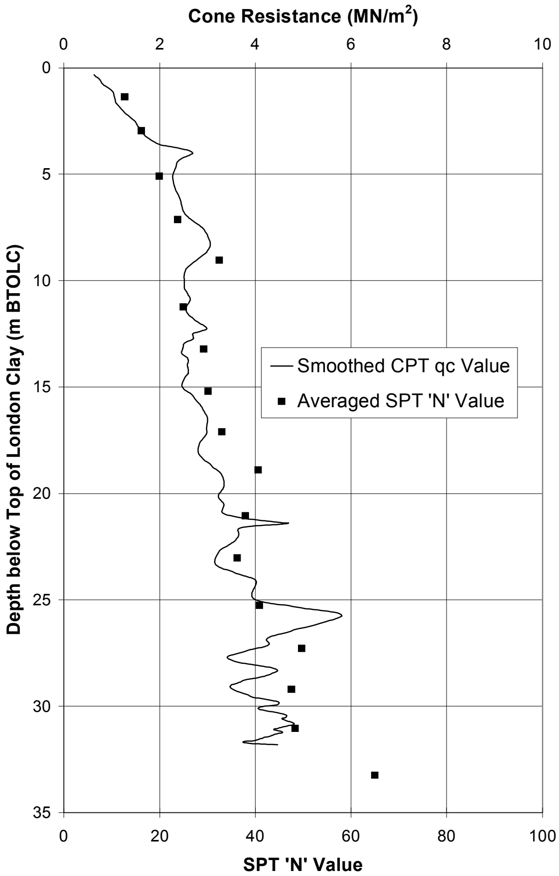

Figure 1: Cone Resistance and SPT 'N' Value vs Depth below Top of London Clay

At about 21m OD, the unweathered London Clay becomes siltier and sandier marking a change from the upper lithological unit B to the lower unit A (King, 1981). Figure 1 summarises the variation of SPT "N" with depth (averaged from numerous tests) and a typical CPT q_c profile, whilst Figure 2 summarises the profiles of shear modulus from the seismic cone (SCPT) and self-boring pressuremeter tests (SBP).
Piezometer monitoring indicated a water table level of about 2.5m below the London Clay (LC) surface, with a sub-hydrostatic increase of pore water pressure with depth to 200kN/m^2 at a level of about 20mOD; and remaining approximately constant below 20m OD.

The best estimate of undrained shear strength (based on in-situ and high quality laboratory tests) was:

$S_u = 40 + 7.5z$ kN/m^2 for z between 0 and 10m below LC surface

$S_u = 70 + 4.5z$ kN/m^2 for z between 10 and 32m below LC surface.

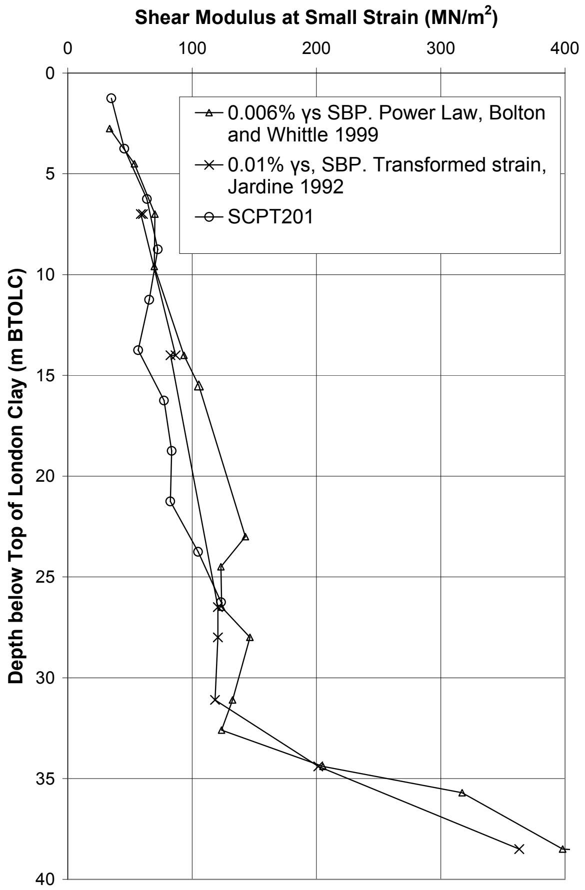

Figure 2: Shear Modulus at Small Strain vs Depth Below Top of London Clay (γ_s = shear strain)

Conventional quick undrained triaxial tests on 100mm diameter samples gave undrained strengths (S_{u100}) which were higher than the above "best estimate" profile, particularly within the near surface weathered London Clay. UK experience indicates that in central London, S_{u100} strengths with an α of 0.45 are usually appropriate to predict the ultimate capacity of bored piles (when compared with maintained load test data). However, at Wembley this conventional approach would have led to pile capacities being over-predicted by about 25% for 10m long piles, with the over prediction reducing for longer piles. It is possible that this discrepancy is due to the site's stress history being rather different to that in central London where the majority of the empirical data base has been obtained.

3 PILE LOAD TESTS

The results of 7 vertical maintained load tests are summarised in Table 1.

Table 1: Summary of Vertical Pile Tests

Pile Test No.	Pile Dia (m)	Ground Surface (mOD)	Pile Length (m)	Predicted Capacity (kN)	Ultimate Load (kN)
1	0.45	46.8	15.2	1408	1462
2	0.45	46.8	25.0	2811	2905
3	0.75	46.8	11.5	1852	2030
4	0.60	46.8	25.0	3864	3864
5	0.60	46.6	20.0	2839	2698
6	0.60	53.1	20.0	2839	3267
7	0.60	50.2	20.6	2974	3123

Note: 'Ultimate' load obtained at displacements of 7%-10% of pile diameter.

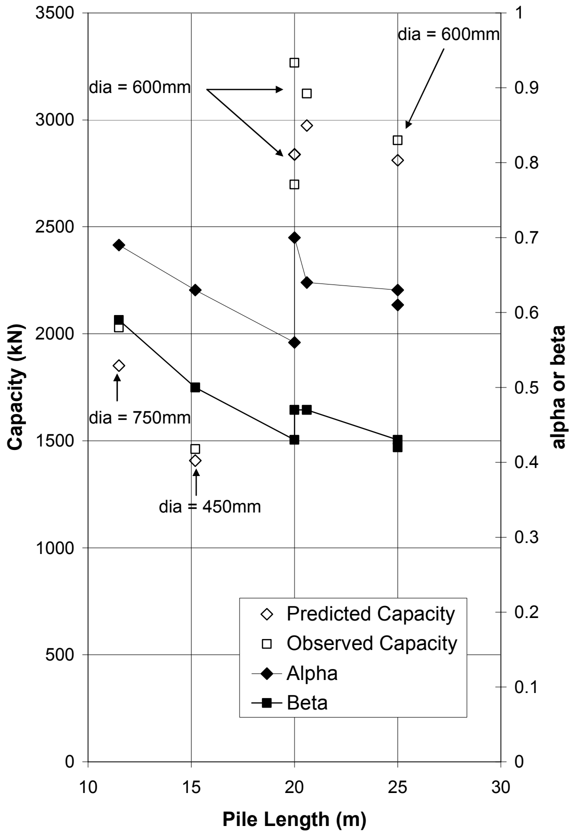

Figure 3: Comparison of Predicted and Observed Pile Capacities and Mobilised Values of α and β

Figure 3 shows a comparison of the predicted and observed ultimate pile capacities and the mobilised values of α (τ / S_u) and β (τ / σ_v'). Pile tests 1 to 3 were carried out within 10m of one another, whereas the other pile tests were distributed around the old stadium footprint. The ground surface elevation for test piles 4 and 5 was similar to that at test piles 1 to 3, whereas at test pile 6 the ground surface was about 6.5m higher than that at test piles 1 to 5. The measured ultimate pile loads were about 5% higher on average than those predicted by the best estimate undrained strength profile together with an α factor of 0.6. Values of β (= τ/σ_v'), Burland 1973, follow a consistent pattern with β reducing from about 0.6 for short piles to about 0.4 for long piles. The higher capacity of pile test 6 compared with pile test 5 is mainly explained by the higher average effective stress along the pile shaft.

Figure 4 shows the measured load displacement behaviour for one of the lateral load tests. The load displacement response is strongly non-linear, and non-linear elastic behaviour prior to local soil failure had to be assumed for realistic predictions to be made. The parameters used for the non-linear analysis are summarised on Table 2, the small strain shear moduli derived from the SBP tests (refer to Figure 2) were used as the initial tangent horizontal Young's moduli.

Table 2: Vertical (E_v) and Horizontal (E_h) Young's Moduli used in Repute Analysis

Depth (m)	E_v (MN/m^2)	Variation of E_v with Depth	E_h (MN/m^2)	Variation of E_h with Depth
0-5	100.0	6.8	100.0	15.0
5-10	134.0	9.0	175.0	11.0
10-25	179.0	14.0	230.0	14.6
25-35	390.0	3.0	450.0	0
35-40	420.0	0	450.0	0
> 40	Rigid Layer			

Note: E_v and E_h quoted is value at top of a layer

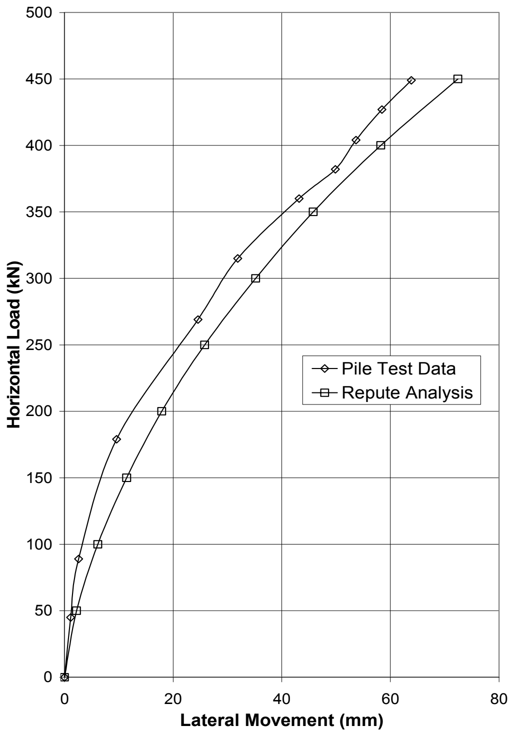

Figure 4: Comparison of Results from Lateral Load Test on 750mm Diameter Pile with Repute Analysis

The following hyperbolic curve fitting constants (R_f) were used to model the non-linear response of the pile between the small strain elastic load-displacement region and local soil failure at large strain:

R_f = 0.5 (Shaft)
R_f = 0.99 (Base)
R_f = 0.9 (Lateral Load)

The hyperbolic non-linear model used in Repute (2002) is based on the model proposed by Duncan & Chang (1970).

4 SETTLEMENT BELOW DEEP FILL

A deep area of fill (about 9m at the extensometer location) was constructed between April and August 2003. The behaviour of this area of fill over time and with depth has major implications for the surrounding structures such as retaining walls and bearing piles. Several different analytical methods were used to predict the behaviour of the fill and extensometers were installed to allow comparisons to be made with the real behaviour. Figures 5 and 6 summarise the observations of settlement with depth and time, respectively observed to date. Also shown are the predictions of settlement based on a non-linear elastic method (NLS) described by O'Brien and Sharp (2001) and from a linear elastic perfectly plastic finite element analysis (Plaxis, 2002).

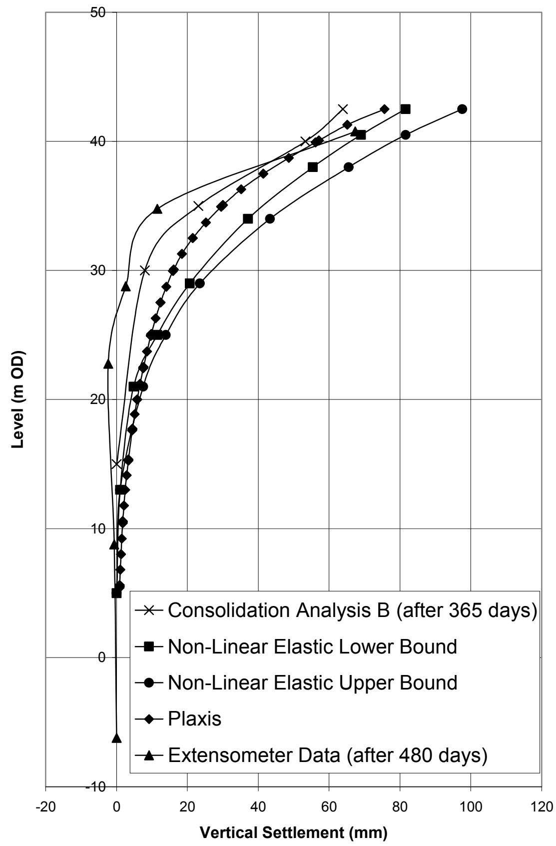

Figure 5: Predicted and Observed Settlement Behaviour, with Depth

Based on observations made to date it is likely that the final total settlement may be about 100mm. The stiffness parameters used for the analyses are summarized on Table 3 and Figure 7; these were derived from insitu and laboratory testing. Laboratory tests included a series of stress path triaxial tests with local instrumentation for small strain measurement, (Clayton and Heymann, 2001).

Figure 6 shows the predictions from two sets of consolidation analyses, analysis A, with a constant permeability of 1×10^{-10} m/s, and analysis B which assumed permeability reducing with depth from 1×10^{-9} m/s at the London clay surface to less than 4×10^{-11} m/s at depths greater than 22m below the LC surface (which is consistent with observed permeability variations in the London Clay, Dixon and Bromhead 1999). In addition, mobilsed Young's modulus was derived from the upper bound non-linear NLS analysis. The observed behaviour is similar to the consolidation analysis B. Currently available data, approximately 15 months after placement of the fill, indicates that only relatively shallow settlement has occurred to date. Deeper seated settlement is likely to develop in the future. However, since the LC at depth is considerably stiffer total settlement is only expected to increase by about 50%, from the current measured values of about 65 to 70mm.

Table 3: Stiffness Parameters used in Settlement Analyses

Elevation (mOD)	Young Modulus (E') MN/m^2	
	NLS ($E'_{v0.1}$)	PLAXIS
42.5	29	20
33.5	54	
23.5	88	170
13.5	207	390
5.0	479	1000
<0	Rigid	Rigid

Notes:
(1) NLS, $E' \alpha (p'_c)^n$, n=1.0 lower bound, n=0.8 upper bound
(2) NLS, Degree of stiffness non-linearity shown in Figure 7
(3) NLS, E'_v = drained secant Young's Modulus at 0.1% volumetric strain at p'_0.
(4) Young's modulus varies linearly with depth between elevations indicated.
(5) p'_0 is initial mean effective stress, p'_c is average mean effective stress during consolidation
(6) E' for PAXIS is linear elastic.

5 CONCLUSIONS

The construction of the new Wembley Stadium has raised several geotechnical challenges, including assessing pile behaviour under complex load combinations and large time dependent global ground movements. Based on observations from field tests and currently available data, the main conclusions are:

(i) pile behaviour under vertical and lateral loads was able to be predicted with reasonable accuracy (for practical purposes), but both undrained strength and deformation properties had to be carefully assessed from both insitu and laboratory tests on high quality samples
(ii) empirical methods for predicting vertical pile capacity although well established in London Clay proved to be unreliable at this site, possibly due to its topography and associated changes in its stress history
(iii) an effective stress approach generally provided a more realistic approach to predicting vertical pile capacity however β appears to be depth dependent for this heavily over consolidated clay.
(iv) load-deformation behaviour of laterally loaded piles could only be satisfactorily predicted if non-linear small strain stiffness behaviour was assumed.
(v) the variation of settlement with depth and with time, is sensitive to depth dependent permeability variation, non-linear small strain stiffness and the variation of drained stiffness with mean effective stress during consolidation.

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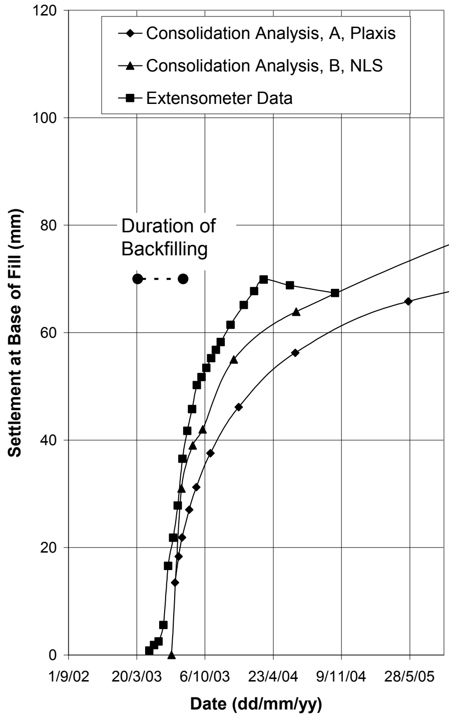

Figure 6: Predicted and Observed Settlement Behaviour, with time

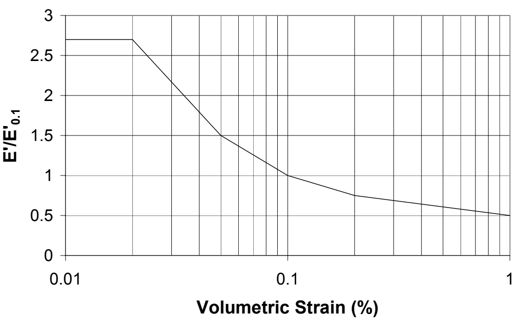

Figure 7: NLS, variation of mobilized Young's modulus with Strain