PILE GROUP DESIGN FOR MAJOR STRUCTURES

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This paper uses case histories and a parametric study to compare the influence of differing geologies and analytical methodologies in the design of large pile groups for major structures. Using both linear and non-linear stiffness parameters the paper analyses the performance of two large pile groups at London's New Wembley Stadium and the Emirates Twin Towers in Dubai. Current practice, code requirements and project risk and sustainability issues are considered. The paper concludes that designers need to consider a factor of safety for the whole group, structural capacity and deformation criteria rather than concentrating on the geotechnical capacity of individual piles within the group.

INTRODUCTION

For the majority of major structures a pile group will form the foundation for the superstructure, say a large span bridge or large tower. For the pile group foundation the designer should consider the overall performance of the foundation system, and answer a series of questions regarding the soil-structure interaction of this system, Figure 1. However, many designers and contractors focus their efforts on the assessment and verification of the ultimate geotechnical capacity of a single pile. In many instances, designers attempt to obtain a code compliant factor of safety for each pile within a pile group. This practice, in combination with routine linear elastic analysis, will lead to uneconomic foundation design, e.g. Burland 2006, Mandolini, 2005.

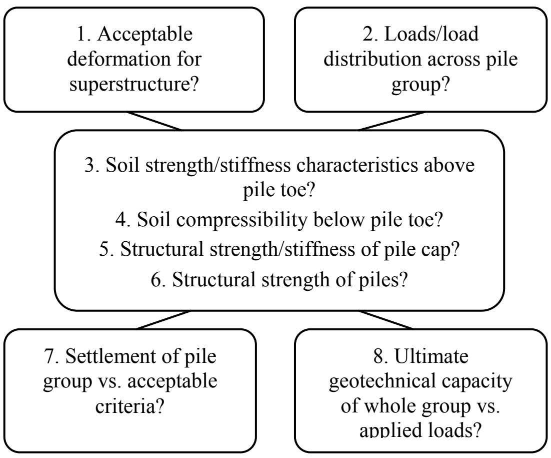


Figure 1: Design Considerations for Pile Groups, Vertical Loading

This paper describes the performance of two large pile groups firstly for the western base of the iconic Arch for the New Wembley Stadium, and secondly for the Emirates Twin Towers in Dubai. For these large pile groups (in very different ground conditions) the following issues are considered:

(i) Non-uniform axial load distributions in piles across the pile group
(ii) Group versus individual pile factors of safety
(iii) Pile group deformation

For (i) to (iii) above, the influence of different analytical methods and different soil parameters are considered. Finally the paper considers the impact of different design strategies in the context of current code requirements, project risk management and sustainability. The paper will focus on pile group behaviour under vertical loads. For the purpose of this paper, a "large" pile group comprises 25 piles or more in the group.

WESTERN BASE FOR THE NEW WEMBLEY STADIUM ARCH

The new Wembley Stadium's architectural feature is an iconic 133m high triumphal arch. The arch has a 315m span and weighs 1,750 tonnes. Each end of the arch is supported on a pile group. The western arch base also forms the foundation for an adjacent shear core, and the pile group, comprising 60 no. 1.5m diameter piles, is the largest at the new stadium. Firstly, the site and geology will be described and the preliminary pile tests, before discussing the pile group design.

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.

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). The London Clay is a heavily overconsolidated high plasticity clay, at this site the plasticity index tended to reduce with depth, from about 55% to 60% in the upper weathered London Clay to about 30% to 40% in the deeper, siltier, unit A London Clay. The underlying Lambeth Group is a more complex stratum, comprising interbedded layers of very stiff clays, very dense sands and silts. Figure 2(a) summarises the variation of SPT "N" with depth (averaged from numerous tests) and a typical CPT q_c profile, Figure 2(b) summarises undrained shear strengths derived from a wide variety of methods (both insitu and laboratory), whilst Figure 2(c) summarises the profiles of small strain 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 insitu and high quality laboratory tests) was:

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

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

It should be noted that the "best estimate" undrained strength profile (termed "characteristic" design profile on Figure 2(b), consistent with EC7 terminology) was derived from a wide range of insitu and laboratory data (such as CPT correlations, and high quality sampling, using push-in thin wall and rotary core sampling methods together with triaxial testing at slow rates of strain). The majority of the undrained strength data indicated that (compared with routine quick undrained triaxial tests, 100mm diameter specimens, obtained from U100 sampling): undrained strengths were relatively low in the near surface weathered zone; and undrained strength increased rapidly with depth in the deeper unweathered London Clay, (refer to Figure 2(b)). Hence, for predicting pile capacity there were two options:

(i) Use a conventional approach, based on U100 quick undrained triaxial tests and an α factor of 0.5. This approach is commonly used, and in general, is relatively successful in Central London, when compared with maintained load test data, Patel (1992).

(ii) Use the best estimate strength profile with a "modified α" factor. Based on comparisons with effective stress methods a "modified α" value of 0.6 was judged to be appropriate, (for routine application a total stress method was preferred, since it was simpler than an effective method to use).

Both (i) and (ii) above, gave similar pile capacities for a pile length of about 20m. However, at Wembley the piles which were to be constructed (in total about 4000 piles) varied significantly in length from about 10m to nearly 40m. The concern with a conventional approach (option (i)) is it would over-predict the capacity of short piles and under-predict the capacity of long piles. Hence option (ii) was used for predicting ultimate pile capacity. Seven preliminary pile tests were subsequently carried out under compressive loading to failure, Table 1. The measured ultimate geotechnical capacity of the test piles was about 5% higher on average than those predicted using the method outlined in option (ii) above, importantly there was no bias in predicted capacity with respect to pile length. The reason for the poor reliability of a conventional approach is unclear. However, at Wembley the depth of weathering and the site's stress history is rather different to that in central London where the majority of the empirical data base has been obtained.

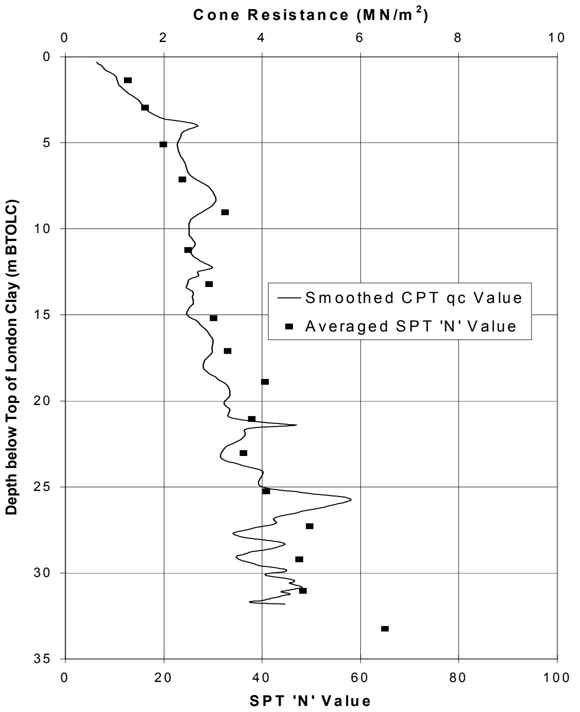
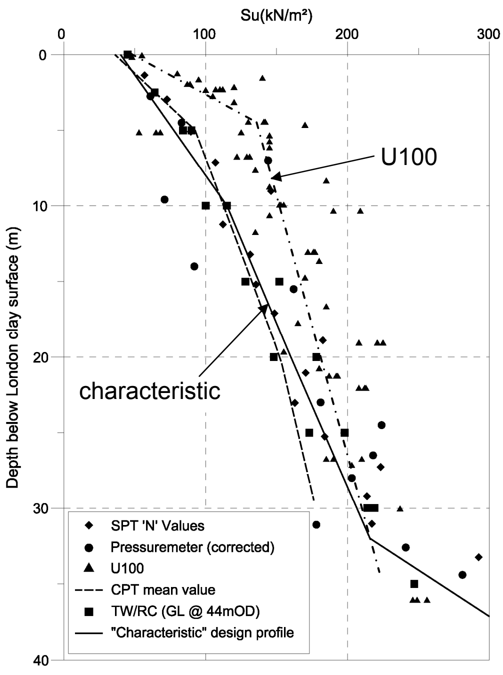


Figure 2(a): Wembley, Cone Resistance and SPT "N" Values vs Depth



Note TW/RC – thin wall push in or rotary core samples sheared at 4.5%/day strain rate

Figure 2(b): Wembley, Undrained Shear Strength

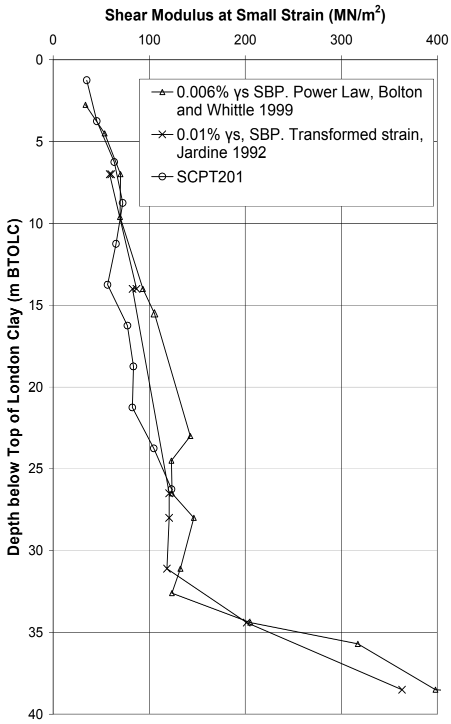


Figure 2(c): Wembley Shear Modulus at Small Strain vs Depth (γ_s = shear strain)

Figure 3(a) shows the axial load-settlement response of 0.45m diameter, 15m long and 0.6m diameter, 25m long test piles, together with the predicted load-settlement curves from Fleming's (1992) method, based on the input parameters given in Table 2. Figure 3(b) illustrates the predicted load-settlement curve for a 1.5m diameter, 33m long, pile compared with a working pile test loaded to 10MN. It can be seen that for this site the settlement characteristics and ultimate capacity of individual piles could be accurately predicted.

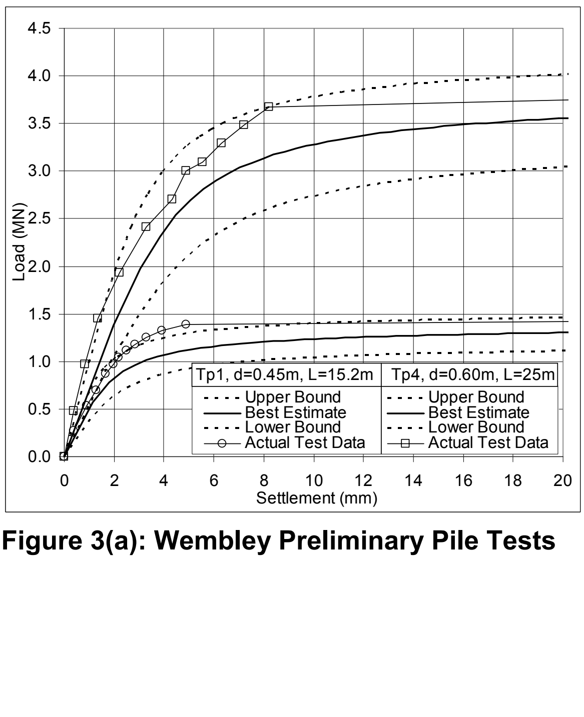


Figure 3(a): Wembley Preliminary Pile Tests

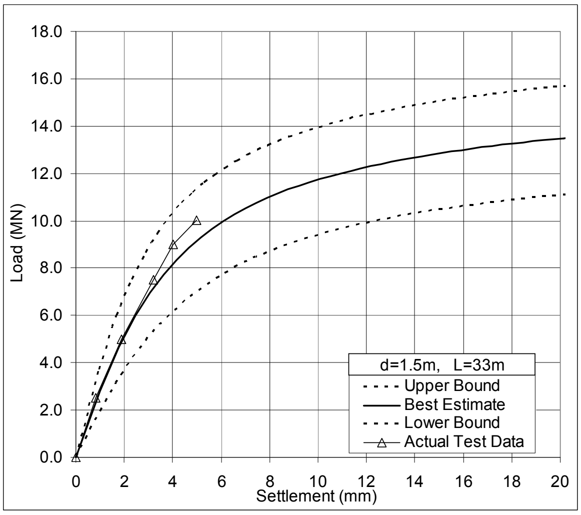


Figure 3(b): Wembley, 1.5m Dia Pile Test

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

Notes:
1."ultimate" load obtained at displacement of 7%-10% of pile diameter.
2. maintained load tests ICE Specification (1996)

Table 1: Summary of Vertical Pile Tests

Parameter	"Characteristic" Values	UB	LB
Shaft Capacity, U_s	$= \alpha S_u$, $\alpha = 0.6$	+10%	-10%
Base Capacity, U_b	$q_b = 9 S_u$	+10%	-30%
Shaft Length through Made Ground, Lo (Zero Friction Transfer	Best Estimate	+5%	-5%
Shaft Length through London Clay, L_f (Friction Transfer)	Best Estimate	+5%	-5%
Shaft Flexibility, M_s	0.001	0.0008	0.0015
Base Deformation Modulus, E	250 S_u	500 S_u	100 S_u
Concrete Young's Modulus, E_c (GN/m²)	40	60	30

Notes:
1. Friction Transfer Coefficient, Ke equal to 0.45 for all analyses.
2. Refer to Fleming (1992) for description of parameters
3. UB = upper bound, LB = lower bound

Table 2: Wembley Fleming's Analyses (CEMSET), Input Parameters

For the Western arch base the vertical load applied to the pile group was about 325MN, which is equivalent to an average load per pile of about 5.4MN. A 1.5m diameter pile 27m long was estimated to have an ultimate geotechnical capacity of between 12MN (based on undrained strength, and an α of 0.6) and 13MN (based on an alternative effective stress approach, Bown and O'Brien, 2008). Hence, the overall group Factor of Safety was expected to be between 2.2 and 2.4. In terms of practical construction, the 27m long pile was believed to be optimal since it ensured that the pile toe was kept within the London Clay, being about 2 to 3m above the surface of the Lambeth Group. Hence, the pile bore could be bored "dry" with conventional rotary techniques in a rapid cost-effective manner. Initially linear elastic analyses of pile group behaviour were carried out using the software MPILE. The shear moduli profiles, Table 3, were carefully selected to reflect both the influence of near surface weathering and the effects of decreasing strain amplitude (and therefore increasing mobilised stiffness) at depth. For example, the ratio of vertical shear modulus to undrained shear strength varies from about 200 at 5m depth to about 350 at 25m depth. The predicted pile group settlement was about 28mm under serviceability load conditions. However, the principal concern was the distribution of axial loads across the group, Figure 4(a). A maximum axial load of 15MN was predicted and numerous piles around the pile group perimeter had predicted axial loads in excess of 10MN. Given the ultimate axial capacity of 12MN, these analyses indicated that the minimum individual pile factor of safety would be less than 1.0. Some parts of the design team wished to utilise a design criterion which required every pile within the pile group to meet a minimum factor of safety of 2.0. This would have had a major impact on pile construction, since the bored piles would need to be much longer and would need to penetrate the water bearing Lambeth Group. Hence to maintain pile bore stability, the use of bentonite slurry support would have been required. There would have been several adverse impacts both on the project and the local urban community:

(i) Increased construction cost
(ii) Increased construction programme
(iii) Increased construction traffic through adjacent urban areas; importing extra materials (steel, concrete) into site, and exporting additional waste from the site
(iv) Increased risks of variable pile performance (due to greater difficulty of constructing piles), potential pile

bore instability at depth and associated concerns about pile integrity and long term durability.

Depth	Linear Elastic (Gv in MN/m²)	Non Linear (Evo in MN/m²)
0	4 + 2.4Z ν' = 0.15	100 + 6.8Z
5		134 + 9Z
10		179 + 14Z
25		390 + 3Z
35+		420

Notes:

1. Vertical loading hyperbolic parameters for non-linear analysis: Rf = 0.5 for shaft; Rf = 0.99 for base. Poisson's ratio, ν' = 0.1
2. Interface strength for non-linear analyses based on α = 0.6 and best estimate undrained strength
3. Z is depth below London Clay surface
4. Gv is vertical shear modulus
5. Evo is tangent Young's modulus at small strain for non-linear analysis, derived from seismic cone test.

Table 3: Wembley. Input Parameters for Linear and Non-Linear (Repute) Analyses

The senior author considered that lengthening the piles was wasteful and also counterproductive in terms of project risk management. Based on published research (e.g. Cooke, 1986) it was considered likely that:

(i) The linear elastic analyses were over-predicting axial loads around the perimeter of the pile group.
(ii) Even if locally piles did reach their ultimate geotechnical capacity, load could be safety redistributed to adjacent piles provided that they had sufficient reserve capacity.

To verify (i) above, more sophisticated non-linear analyses were carried out using bespoke MM software (NLPILE) for final design of the pile group. NLPILE assumes non-linear pile load-settlement behaviour, but pile to pile interaction factors are derived from linear-elasticity. NLPILE was validated both theoretically (when piles assumed to behave elastically) against published solutions (Poulos and Davis, 1980) and against limited available case history data (e.g. Cooke et al, 1981). The principal advantage of NLPILE, compared with linear elastic software such as MPILE, is that the consequences of local yielding at, say, corner piles, in terms of pile group settlement, rotation, and axial load redistribution can be assessed. Prior to raising the arch (O'Brien, 2007) non-linear boundary element analysis (REPUTE) was carried out to assess pile group deformation.

Assumption (ii) critically depended on the pile load-settlement behaviour being ductile and ensuring that the pile cap was sufficiently strong and stiff to transfer loads between piles across the group. Data presented by Burland et al (1986) indicates that the load-settlement behaviour of friction piles in overconsolidated clays is ductile.

Based on the above considerations the design criterion were:

(i) An overall factor of safety on group capacity of 2.0 or more against geotechnical failure (sum of individual pile capacities or block failure, whichever critical);
(ii) Pile group settlement of less than 50mm;
(iii) Acceptable structural capacity for both piles and pile cap under the full range of anticipated loads;
(iv) Individual pile factors of safety against geotechnical failure were not considered.

The REPUTE input parameters are summarised in Table 3. The input parameters for NLPILE assumed a non-linear load-settlement curve for an individual pile consistent with the "characteristic" value from CEMSET analysis (Table 2), together with group interaction parameters, k, of 0.216, 0.091 and 0.068 for the outer, middle and inner zones of the pile group (Figure 4(b)). The interaction parameters "soften" the load-settlement response of the piles to allow for group interaction effects, equation (1), these interaction parameters were derived from linear elasticity.

$$\delta_p = {\delta_i}/{k} \qquad - (1)$$

δ_p = pile settlement within group

δ_i = pile settlement of individual pile

k = group interaction parameter for relevant zone

The NLPILE predictions of axial load distribution are summarised in Figure 4(b). Comparing Figures 4(a) and 4(b) it is clear that the non-linear analysis gives a more uniform distribution of axial forces, with a peak axial load of 8MN. Based on this analysis it was decided that the pile group layout of 60, 1.5m diameter, 27m long piles was adequate. Because of the large horizontal and moment loads applied to the group under various ultimate limit state load cases during arch raising, the structural capacity of the piles (criterion (iii) above) proved to be the critical design criterion.

Prior to the Arch raising a full non-linear analysis was carried out using REPUTE, this gave similar distributions of axial load across the pile group as NLPILE. However, the predicted pile group settlement of 15mm was about half of that previously predicted from either the MPILE or NLPILE analyses. These pile group settlements were based on the serviceability load case, which is dominated by the arch and substructure weights, which could be calculated quite accurately. The non-linear load-settlement characteristics for an individual pile simulated by both NLPILE and REPUTE were practically the same. Hence, it is considered that the main reason for the difference in settlement prediction is that the pile to pile interaction simulated in NLPILE (based on elastic interaction factors, and similar to MPILE) was over-conservative. The observed pile group settlement, about two years after arch loading was applied was about 8 to 10mm.

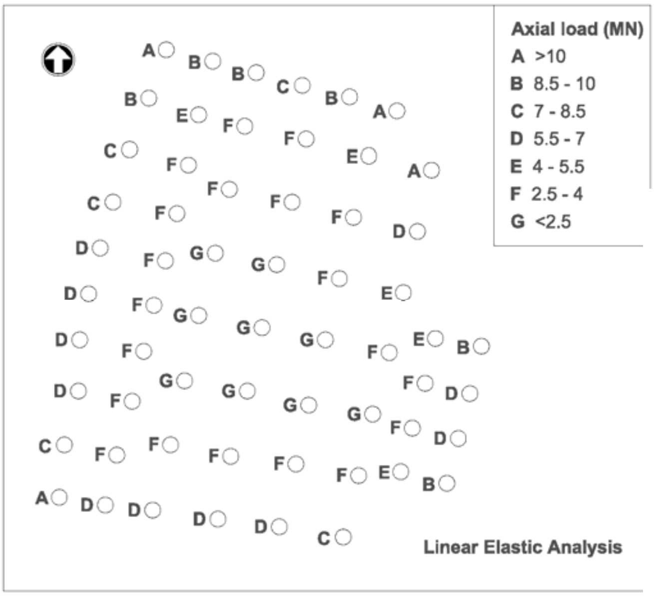


Figure 4(a): Wembley Western Arch Base, Axial Load Distribution, Linear Elastic Analysis

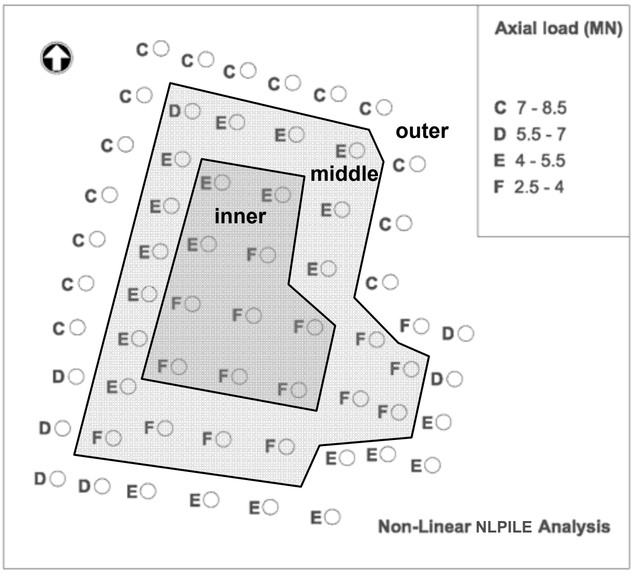


Figure 4(b): Wembley Western Arch Base, Axial Load Distribution, Non-linear Analysis

EMIRATES TWIN TOWERS, DUBAI

The pile group design for the Emirates Twin Towers provides an interesting case history, due to the different ground conditions compared with Wembley Stadium. The ground conditions and foundation design are described by Poulos and Davids (2005). These are briefly summarised below, before the revised analyses carried out by the authors of this paper are described.

Strata	Linear elastic	Non-linear	
	Secant Young's Modulus (MN/m^2)	Tangent Young's Modulus (small strain) (MN/m^2)	Interface strength τ (kN/m^2)
Silty Sand	15	20 + 8Z	20 + 5.5Z
Sand	50		
Calcareous Sandstone	250	1500 + 40Z	150 +12Z
Cemented Sand (?)	50		
Calcareous Siltstone + Conglomerate	200	3000 +50Z	500

Notes:
1. Linear elastic, pile base stiffness, $E' = 40MN/m^2$
2. Non-linear, hyperbolic constants, R shaft = 0.65, Rbase = 0.99
   Maximum shaft friction = $500kN/m^2$,
   maximum base pressure = $2700kN/m^2$,
   pile base modulus E'_0 = 750MN/m2 (E'_0 = small strain tangent stiffness)
3. Z is depth below ground surface
4. Fleming (1992), CEMSET, Parameters: $L_p = 40m$;Ms = 0.0015; $E_b = 750MN/m^2$ $q_b = 2.7MN/m^2$
   $E_r = 40GN/m^2$; Ke = 0.45

Table 4: Emirates Twin Towers, Pile Group Analyses, Input Parameters

The ground conditions mainly comprise interbedded calcareous siltstone and sandstone. The strength of these materials are typically between five and eight times larger and the small strain stiffness (from insitu seismic tests) are often an order of magnitude larger than the London Clay. The tower was founded on two triangular shaped pile groups comprising 92 and 102, 1.2 diameter piles, 40m long. The estimated ultimate geotechnical capacity of a single pile was about 42MN. The parameters assumed for the revised analyses for this paper are given in Table 4. These analyses included both linear elastic and non-linear analyses. Firstly the load-settlement behaviour of a single pile was modelled and calibrated against preliminary pile test data for a 0.9m dia, 40m long pile, Figure 5, prior to modelling the entire pile group. The revised analysis predictions of axial load distribution across the pile group are summarised in Figure 6.

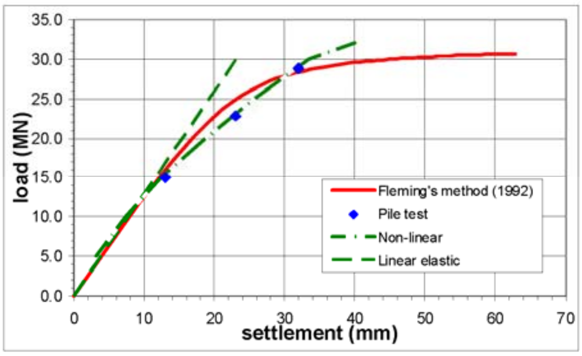


Figure 5: Emirates Twin Towers, Preliminary Pile Test and Analysis Simulations

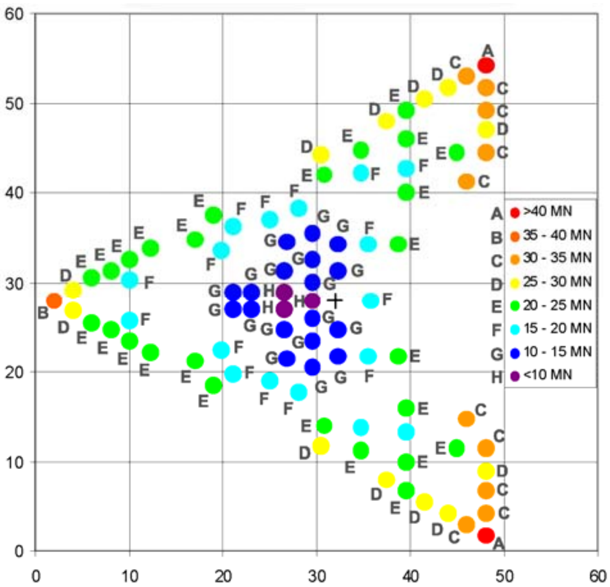


Figure 6(a): Emirates Twin Towers. Axial Load Distribution – Linear Elastic

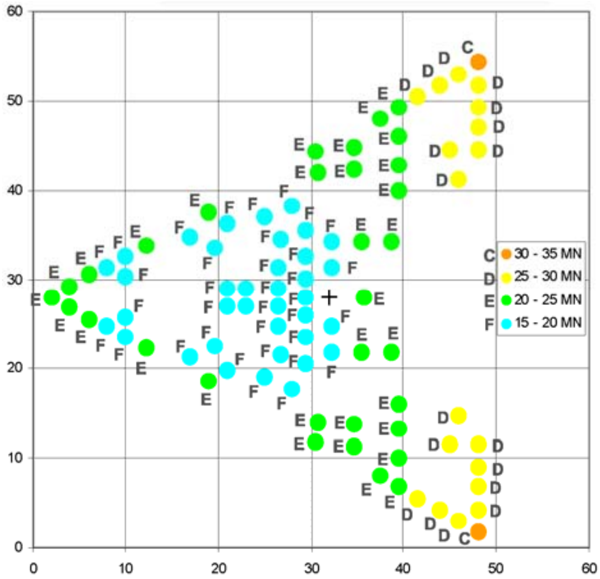


Figure 6(b): Emirates Twin Towers. Axial Load Distribution – Non-linear

Similar to the Wembley analyses the linear elastic analysis predicts a highly non-uniform distribution of axial load, in particular axial loads in corners are especially large. The linear elastic analysis predicts maximum axial loads of 43MN compared with a maximum of 31MN from the non-linear analysis. Hence, similar to Wembley, the local factor of safety for a pile in the group, (based on linear elastic analysis) is less than 1.0, although the overall factor of safety for the pile group is about 2.0. The pile group settlement was observed to be about 8 to 10mm after 70% of the load had been applied, and the estimated final settlement was 20 to 40mm, Poulos and Davids (2005). The original prediction, based on linear elastic analysis, was for pile group settlement to be between 90 and 140mm. The pile group settlement predicted from the revised analyses were:

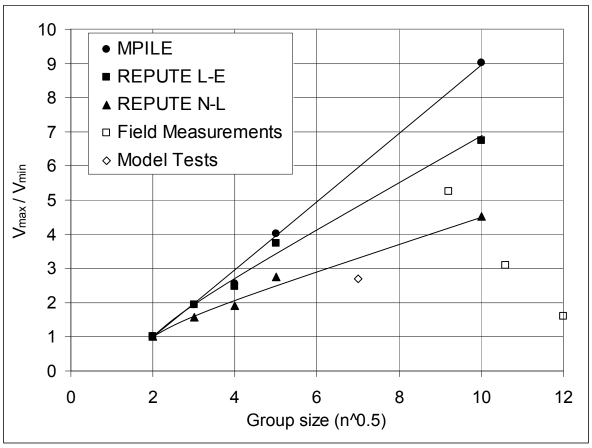
(i) Linear elastic : 55mm
(ii) Non-linear elastic : 25mm

DISCUSSION

The case histories described above can be compared with observations of pile group behaviour reported in the literature and the results of some parametric studies, prior to drawing some general conclusions.

The axial load distribution across pile groups is a common and significant concern for designers. Axial load distributions within pile groups have been monitored for a number of pile groups (e.g. Hooper, 1979; Cooke et al, 1981; Russo, 1996; Katzenbach et al 2000). These field observations consistently show that perimeter, and particularly corner, pile loads are higher than those recorded in piles within the central area of a group. To gain some further insight into the predictions of different analytical methods a parametric study was carried out for square pile groups, with a pile spacing varying between 2D and 10D (D is diameter), and a group size varying between 4 and 100 piles. The pile length and diameter is constant at 20m and 0.6m respectively. The soil strength and stiffness properties are representative of a firm to stiff overconsolidated clay. Figure 7 summarises the results of this study with the ratio of corner to centre axial loads plotted against pile group size. Vmax is the maximum predicted axial load, which for square pile groups occurs in the corner piles. Vmin is the minimum predicted axial load, which occurs in the piles close to the centre of the pile group. This shows that the linear elastic analyses over-estimate the load redistribution across the group when compared with the non-linear analyses, and that this discrepancy increases with increasing pile group size.

Compared with the available case history data the prediction given by the linear elastic methods appear to be over-conservative. The measured ratio between maximum and minimum axial loads reported by Mandolini et al (2005) tends to vary between 1.5 and 3.1 (for pile spacing between 2.5 and 4 diameters). Whereas the linear elastic analyses indicate maximum axial loads up to nine times larger than the minimum axial load. Nevertheless, it is important to note that for large pile groups, the local factor of safety for individual piles around a pile group will inevitably be relatively low, assuming a factor of safety of between 2.0 and 2.5 is targeted for the overall pile group.



Note: 1. Cooke et al (1981), for n=351 V_{max}/V_{min}=2.2
2. Field measurements for s/d = 3 to 3.5
3. MPILE and REPUTE analyses s/d = 3

Figure 7: Ratio of Peak Corner Load, Vmax, to Min Internal Load, Vmin, versus Pile group Size

For pile group settlement, Mandolini et al (2005) has suggested an empirical correlation between pile group reduction factor, R_g, and pile group aspect ratio, R, equation (2), refer to Figure 8, based on an analysis of 63 case histories (for pile groups with varying pile diameter, length, geology, etc):

$$\frac{w}{nW_s} = \frac{a}{(R)^b} = R_g \qquad - (2)$$

w = settlement of pile group
a and b are empirical coefficients, a=0.29, b=1.35
$R = (ns/_L)^{0.5}$, group aspect ratio (Randolph and Clancy, 1993)
s = pile spacing
L = pile length
W_s = single pile settlement, under average working load of piles within group (Q/n)
Q = pile group vertical load
n = number of piles in group
R_s = group settlement ratio
R_g = group reduction factor

Group Settlement ratio, R_s, is the ratio of pile group settlement to settlement of a single pile under an equivalent axial load, equation (3):

$$R_s = \frac{w}{W_s} \qquad - (3)$$

Group reduction factor, R_g, (as indicated in equation (2) above) is inversely proportional to the number of piles in the group and proportional to the group settlement ratio, R_s, equation (4):

$$R_s = n\,R_g \qquad - (4)$$

Both R_g and R_s represent the effects of interaction between piles within a group. Figure 8 compares the case history data from the study by Mandolini et al (2005) against the results of the parametric study. From this comparison it can be seen that:

(i) The linear elastic models usually overpredict group settlement, and the overprediction increases with increasing pile group size (for the conditions considered and a $G/_{Su}$ value of 200). To improve the match between linear elastic prediction and observed settlement, the appropriate $G/_{Su}$ value would need to vary with pile group size.
(ii) The different software, when using the same input parameters (i.e. Repute linear elastic and MPILE), give different predicted settlements (since the different software make different mathematical approximations). Hence, the appropriate input parameters are software dependent. This emphasises the

importance of software calibration against case histories.

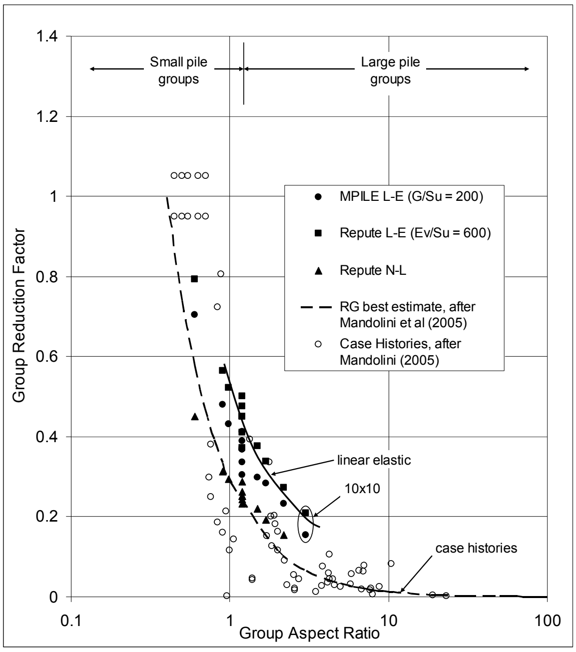


Figure 8: Group Reduction factor, R_g, versus Group Aspect Ratio. R Field Measurements and Parametric Study

The observed and calculated values for the group settlement ratio, R_s are summarised in Table 5 for the Wembley and Emirates case histories. The empirical values of R_s were calculated from Mandolini's empirical relationship, equation (2), and from equation (4), based upon the pile group aspect ratio for the sites under consideration and the single pile settlement under the average working load for piles within the group (based upon CEMSET analyses, refer to Figures 3 and 5, and Tables 2 and 4). The observed value of R_s is lower for the weak rock underlying the Emirates Twin Towers than the deep layer of over-consolidated clay at Wembley. The empirical relationship by Mandolini seems to be quite accurate for Wembley, and rather conservative for the different ground conditions in Dubai (nevertheless it would form a useful design check). As expected the linear elastic analyses give R_s values which are too high, whilst the non-linear analyses are more realistic.

Site	Group Settlement Ratio, R_S			
	LE	NL	empirical	observed
Wembley	8	4.3	3.6	2.9 – 3.7
Emirates	4.8	2.3	5.8	1.8 – 3.6

Notes:
1. LE is linear elastic 2. NL is non-linear 3. Empirical: refer to text

Table 5: Group Settlement Ratio, R_s, for case histories

In general, the Eurocodes represent a step forward, compared with British Standards, for carrying out design where soil-structure interaction is important. However they are unhelpful for the design of large pile groups in the following respects:

(i) Although the importance of serviceability limit state, SLS, analysis is emphasised, there is no guidance on how SLS analysis should be carried out. The overall impression is that ULS checks remain the dominant design criterion;
(ii) The significant differences in behaviour and safety (in terms of redundancy) between a single pile and large pile groups are not properly dealt with.
(iii) As shown in this paper the type of stress-strain model selected for analysis can have a significant impact on the output, and potentially be more significant than the code factors used to ensure “safety”.

Randolph (2003) has outlined the advantages of moving towards displacement based design criteria for large pile groups, rather than relying on criteria which use the capacity of individual piles. The ultimate geotechnical capacity of a single pile, depends critically on local conditions at the pile-soil interface (which are often controlled by pile installation methods and can be highly variable), whereas the deformation response of a pile group is largely controlled by the soil-stiffness behaviour away from the pile, Figure 9. This principle has been well demonstrated by pile tests analysed by Mandolini et al (2005), Figure 10. These tests showed that despite large variations in ultimate capacity (due to construction induced variations) the initial pile settlement characteristics were similar. Pile test data reviewed by Fleming (1992) and England (1999), also indicate that the initial pile deformation characteristics show less variability than ultimate capacity. Hence pile group deformation can be more reliably predicted than the ultimate capacity of a single pile. This is an important consideration for project risk management.

Burland (2006) has highlighted the profound importance of understanding the mode of foundation failure: ductile or brittle. Routine design practice is largely based on the assumption of ductile behaviour.

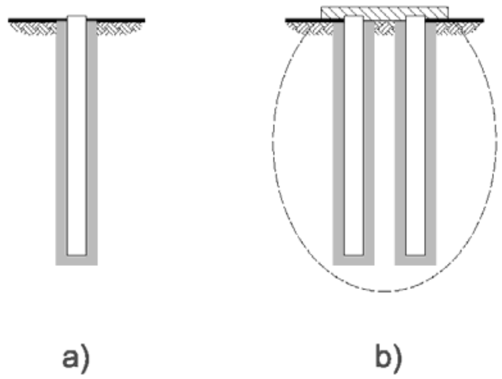


Figure 9: Pile group capacity and stiffness: a) capacity dependent on conditions at pile-soil interface b) stiffness determined primarily by far-field conditions (after Randolph, 2003)

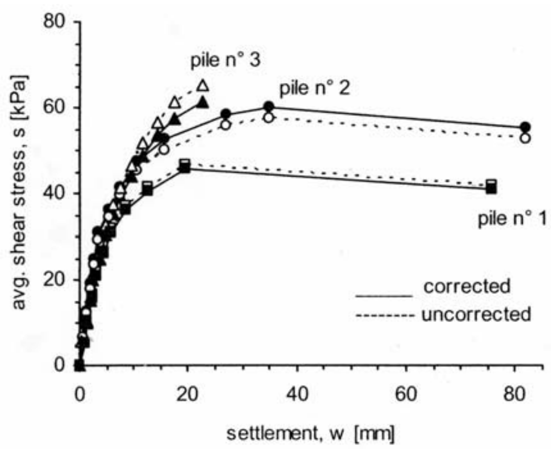


Figure 10: Mobilised Shaft Stress versus settlement, after Mandolini et al (2005)

Usually pile foundations behave in a ductile manner, the exception is for rock socketed piles in some rock types or if "punch-through" failure into underlying weaker ground can occur. Studies carried out in the USA (NCHRP report 507) provide interesting examples of the factor of safety (FoS) needed to achieve the same level of reliability (i.e. the same statistical probability of failure) for a single pile compared with small pile groups of between 2 and 5 piles, e.g. a single pile FoS needs to be 44% higher than a 5 pile group, to achieve the same reliability. By comparison large pile groups will have a much greater level of redundancy.

CONCLUSIONS

The paper has reviewed the performance of two large pile groups, firstly in overconsolidated clay at Wembley and secondly in a weak rock in Dubai. Different methods of analysis have been compared with case history data. The main conclusions are:

(i) Locally factors of safety for piles around the pile group perimeter were low (much less than typical code requirements), nevertheless the overall performance of the pile groups has been good with small settlements observed to date;

(ii) Current codes mainly focus on the ultimate geotechnical capacity of individual piles and associated factors of safety. However, for large pile groups (with significant redundancy) the principal design issues will be acceptable deformation and ensuring the structural strength and stiffness of the piles and pile cap are adequate. It is illogical to continue to utilise current code factors of safety for ultimate geotechnical capacity, for large pile group design, given that the codes were mainly developed for single piles or small pile groups.

(iii) In terms of factor of safety against geotechnical failure, for pile groups subject to vertical loading only, it is only the group capacity which should generally be considered rather than individual pile capacities within a group. For pile groups subject to large overturning moments, then a small partial factor of safety on piles along a perimeter row (say 1.2 to 1.3) may be appropriate to guard against a rotational (toppling) failure. However, for complex loading scenarios, which may involve vertical, horizontal and moment loading, the limitations of linear elastic analyses are usually more significant than for vertical loading only, and non-linear analyses would usually be appropriate.

(iv) Linear elastic analyses will overpredict pile to pile interaction and therefore maximum axial loads around the group perimeter and pile group settlement will tend to be overpredicted. Non-linear analyses are capable of improved predictions, however good quality data on ground stiffness at small stains is required together with careful calibration of the model.

(v) Deformation based design criteria would be beneficial in terms of more economic foundations, improved project risk management (since generally this would lead to shorter, fewer piles which are easier to construct) and better sustainability (less waste and less impact on adjacent areas).

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