Advanced testing and modelling delivers cost effective piled raft foundation solution

Comment des essais avancés, associés à la modélisation, permettent d'obtenir une solution éconoomique de fondation sur pieux et radier

Bourne-Webb P.
Instituto Superior Técnico, Lisbon, Portugal

Cunningham M.
Shell Global Solutions Intl. BV, Den Haag, Netherlands

Card G.
GB Card & Partners, London, United Kingdom

ABSTRACT: A piled raft solution was proposed as an alternative to a conventional fully piled foundation for a new shopping development in Cambridge, UK. This paper demonstrates how the use of precedent knowledge, appropriately targeted investigation and modelling can provide cost effective and resource efficient foundation solutions.

RÉSUMÉ : Une solution radeau empilé a été proposé comme une alternative à une base conventionnelle entièrement empilés pour un développement nouveau point de vente à Cambridge, Royaume-Uni. Cet article montre comment l'utilisation des connaissances préalables, l'enquête ciblée et modélisation peut fournir rentables et efficaces des ressources des solutions de fondation.

KEYWORDS: Gault Clay, piled raft foundation, nonlinear stiffness, site characterisation.

1 INTRODUCTION.

1.1 *Piled rafts and settlement reducing piles*

There is a wide literature on the various methods of analysis and use of raft foundations generally (Cooke 1986, Price & Wardle 1986, Poulos 2001, Reul & Randolph 2003) and the use of piles as settlement reducers (Burland 1995, Love 2003).

In essence, where the soil underlying a structure is sufficiently stiff it is often the case that the use of a plain or piled raft solution will lead to economies when compared to the costs associated with a fully piled foundation system.

Effectively, load from the superstructure is first distributed through a plain raft to the subsoil and if the analysis predicts settlement in excess of that deemed to be acceptable, settlement reducing piles can be introduced at strategic points in order to stiffen the support to the raft and bring the expected settlements down to an acceptable level.

However, this solution is not often examined due to the sophisticated nature of the soil-structure interaction that needs to be analysed – the work involved in delivering the solution is too 'complicated' and the structural engineer prefers the ease, risk transfer and robustness of a fully piled option.

1.2 *Gault Clay and geological setting*

The Gault Clay is an over-consolidated clay that was laid down towards the end of the Lower Cretaceous period (c. 100 Ma), and its engineering behaviour lies between that of a soil and weak rock (Marsh & Greenwood 1995). In Cambridge, UK the Upper Gault sub-division predominates

In the Cambridge area, the strata over-lying the Gault, most significantly the Chalk of the Upper Cretaceous, have been largely removed by erosion, and the area was also subjected to a number of glaciations.

The removal of an estimated 150 m of overburden (Samuels 1975), in addition to the ice cover, has subjected the clay to significant stress relief with associated intense fissuring and softening and in addition, the stratum has experienced moderate levels of tectonic activity.

In engineering terms, the Upper Gault is a high plasticity clay (LL ~ 70 - 80%; PI ~ 45 - 55%) with moisture contents close to its Plastic Limit and a significant calcite content (30 – 40%).

At the site, the soil profile (Table 1), especially the distribution of the superficial soils and the upper surface of the Gault Clay, has been modified by historic construction that had reduced the original ground level.

Groundwater was found to be perched within the superficial soils at a level between 0.5 m and 1 m above the surface of the clay (noting that the surface of the clay undulated significantly), and water levels in the Lower Greensand were found to be in hydraulic continuity (Nash et al. 1996).

Table 1. Site specific soil profile

Top of layer: m OD	Thickness: m	Soil description
+5.8 to +11	0.3 to 3.3	Made Ground
	0.3 to 1.7	Brickearth
	0.1 to 2.4	Sand & Gravel
+3.2 to +8.5	0 to 3.6	Gault Clay (weathered)
	>34.7	Gault Clay
-30 approx.	Not proven	Lower Greensand

1.3 *Development at adjacent sites, Grand Arcade and foundation options*

Located in central Cambridge (Fig. 1), the Grand Arcade site is immediately adjacent to the Lion Yard shopping centre and the Crown Plaza Hotel (Lings et al. 1991, Ng & Nash 1995). The development covers a total area of about 1.4 hectares.

The excellent field and laboratory research work undertaken in relation to the deep basement excavation at the latter site, informed the decisions made during the development of the ground model for this project.

In addition to these relatively modern buildings, other structures of significance to the project were the Post Office (PO) and Telephone Exchange (BT) building(s) in the northeast corner of the site, and retained facades along the eastern boundary, facing St. Andrews Street; all of which had to be protected from damage during the works.

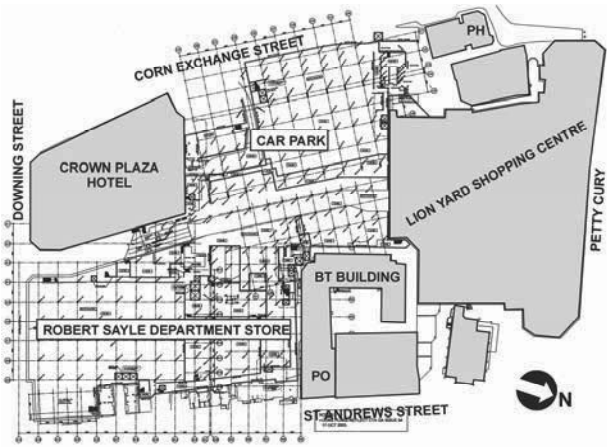

Figure 1. Plan of area surrounding Grand Arcade.

The re-development at the site involved the demolition of a number of existing buildings and because of the long history of occupation in the area, archaeological investigation was undertaken prior to the main construction works starting.

To form the basement substructure, zero sheet-pile and secant bored pile perimeter retaining walls were installed where needed, and ground levels were reduced generally by between 4 m and 7 m across the site.

The gross weight of the new structure was equivalent to a uniform loading of about 120 kPa which with an average unloading due to demolition and excavation of 80 kPa (ranging between 30 and 100 kPa), equates to a net loading of 40 kPa. However, this varied somewhat across the site and as a result in some areas, e.g. under core structures and some columns, net contact pressures locally were as high as 220 kPa.

During tendering, a value engineering exercise was undertaken and the option of replacing the conforming fully piled foundation with a piled raft was investigated, and this preliminary assessment suggested that significant savings were possible using this alternative.

2 SOIL CHARACTERISATION

2.1 *Site specific investigation and testing*

In order to develop the ground model for the proposed raft foundation it was necessary to supplement the site investigation and laboratory testing previously completed with further, high quality sampling and testing. Therefore, three additional 30 m deep boreholes were completed using rotary coring techniques and sub-samples were taken from the cores for stress path testing.

Prior to the stress path testing, the suction in each sample was measured using the IC suction probe, in order to be able to understand whether the samples may have been unduly disturbed when recovered or when in transit.

These measurements proved to be very interesting (Fig. 2) and while generally consistent they were significantly lower than what would be expected based on experience in for example the London Clay Formation, and imply much lower values of “at-rest” earth pressure coefficient, K_0 than might be expected based on a simple one-dimensional depositional and erosion environment.

It is thought that the post-depositional processes alluded to earlier and especially the lateral changes resulting from historic tectonics may have led to lateral stress relief and generally lower K_0 values in the Gault Clay. There is also the possibility that the clay simply cannot maintain suctions high enough to reliably represent the in situ stress conditions, although this is considered less likely - similar results in terms of suction measurement from large diameter (300 mm) samples of Gault Clay recovered from a site near Cambridge were seen by the Authors.

Stress path testing was undertaken on six samples to examine the changes in soil stiffness, during stress paths representative of the expected unloading during demolition and excavation, and reloading as the raft is constructed and loaded.

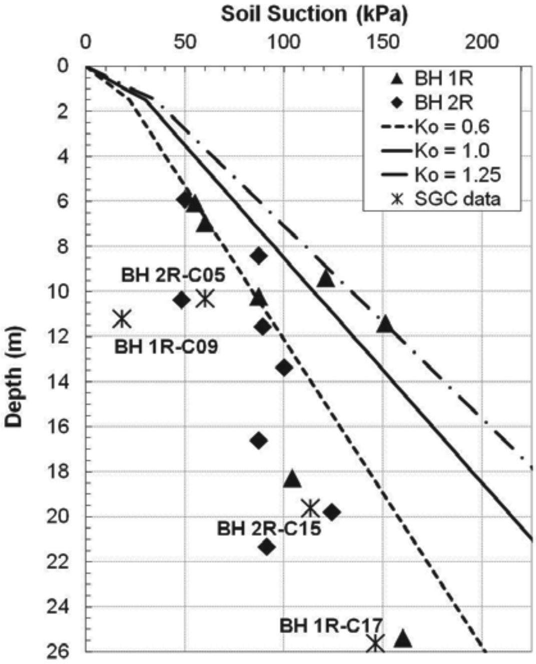

Figure 2. Sample suction profile and implied K_0 values

2.2 *Ground response at adjacent site*

When developing the ground model for design of the piled raft, it was recognised that next door there was effectively a full scale element test available in the form of the instrumented 10 m deep basement excavation created for the Crown Plaza Hotel (Fig. 3).

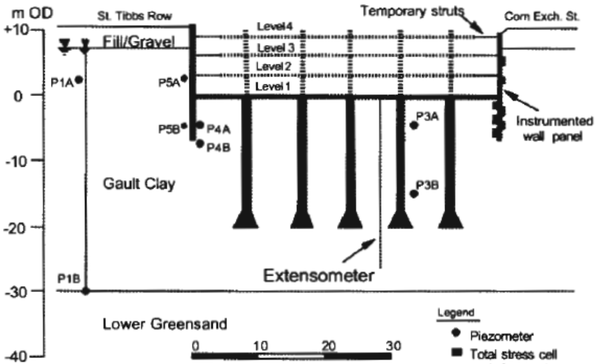

Figure 3. Basement layout and instrument locations, Nash et al (1996)

The instrumented basement construction provides very useful information in terms of the vertical ground movements associated with unloading during excavation and in the long-term (Nash et al. 1996, Lings et al. 1991). And while acknowledging that the piled foundations will have influenced the observations to a degree, the data (Fig. 4) was able to be used to make an independent assessment of the non-linear stiffness of the soil mass with depth and strain level. In particular the excavation to Level 2 was of interest as this represented the level of excavation in the new development.

During the basement excavation, block samples of the clay were recovered for laboratory characterisation of the soil. These studies examined the strength and deformation characteristics of the clay (Ng & Nash, 1995), Table 2 and its stiffness anisotropy (Pennington et al. 1997, Lings et al. 200). As with the field observations, this data informed the decisions made during the development of the ground model for this project as described in the following.

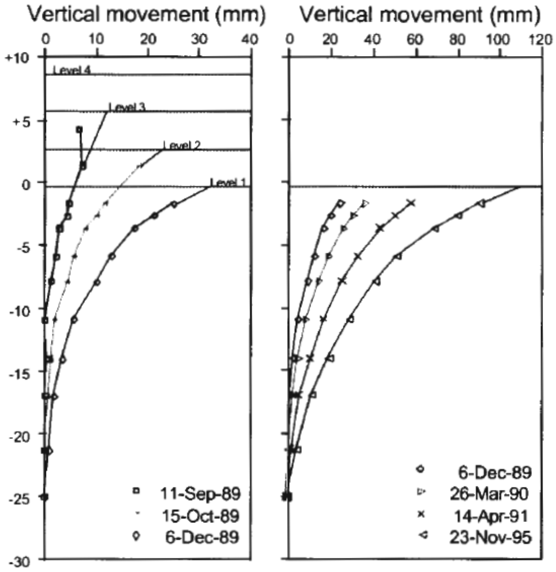

Figure 4. Observations of ground heave inside basement during and post-construction, Nash et al (1996)

Table 2. Typical Gault Clay geotechnical parameters

Parameter	Value
Natural moisture content: %	25 - 32
Liquid Limit, LL: %	70 - 80
Plasticity Index, PI: %	40 - 50
Liquidity Index, LI: %	0 ± 0.1
Undrained shear strength, c_u: kPa	75+5z $^{(1)}$
Critical State angle of resistance, φ'_{CS}: deg	24° - 28°
Peak apparent cohesion, c'_{PK}: kPa	2 - 3
Peak angle of shearing resistance, φ'_{PK}: deg	32° - 34°
Initial shear modulus, G_0: MPa	80 - 120
Young's modulus at 0.2% strain, $E'_{0.2\%}$: MPa	c. 10

(1) z = depth below top of clay

2.3 *Integrated ground model*

In order to undertake the geotechnical calculations for the design of the piled raft, a ground model in terms of vertical stiffness, E was developed based on back-analysis of the observed heave response in the adjacent basement, Fig. 4 & 5 and stress path testing undertaken on high quality core samples from the site, Fig. 6. Data from the latter are summarised in Table 3.

The stiffness data from these two sources are compared in Fig. 6 – the comparison is remarkably good given the quite different sources and gave confidence in the use of the ground model for the design of the foundation system.

Table 3. Summary of stress path testing

Sample	1R-C09	1R-C17	2R-C05	2R-C17
Depth: m	11.2	25.6	10.3	19.6
$G_{max,i}$: MPa	37.5	180	45.5	104
$E_{u,0.01}/c_u$: -	1000	2200	1400	1200
$E_{u,0.1}/c_u$: -	475	860	300	560
$E_{u,0.5}/c_u$: -	190	220	100	280

The stiffness profile chosen for the design calculations is shown in right-hand side of Fig. 5 where it is compared to the 'average' line from the field data (dashed line in both sides of Fig. 5) and various values of constant E_u/c_u ratio ranging from 250 to 1500.

In the preliminary design at tender stage, anticipating strains in the region of 0.2% to 0.3%, a uniform modulus ratio E_u/c_u = 300 was used. It is clear from the figures below that this assumption quickly becomes unrepresentative of the response of the clay at depth and therefore any calculations are likely to overstate the movements that might be expected.

In the design calculations, use of the linear model yielded settlement predictions about one-third larger than that of the pseudo-nonlinear model. Thus peak settlements were reduced from 90 mm to 60 mm, and the area where settlements were considered excessive (i.e. greater than 40 mm), was greatly reduced.

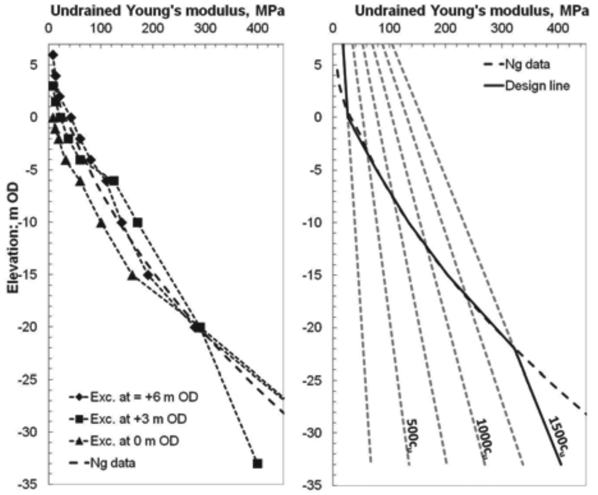

Figure 5. Back-analysis of ground stiffness model

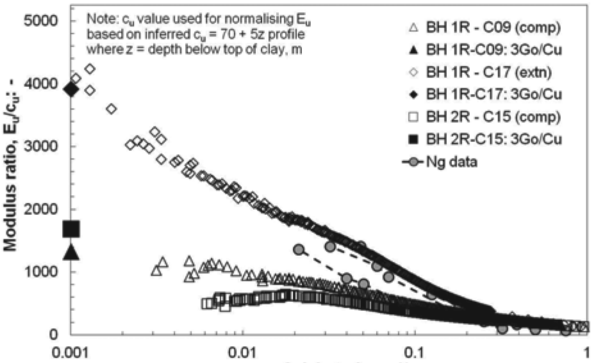

Figure 6. Nonlinear stiffness response from laboratory and back-analysed field data

3 FOUNDATION DESIGN

3.1 *Calculation method*

A number of simplified methods have been proposed for the evaluation of the load-settlement response of piled rafts but often these are difficult to apply to cases where the raft shape and/or load patterns are complex – as was the case here.

For this project, a plate-on-springs type structural analysis was undertaken in parallel with geotechnical analysis of ground movement using the pseudo-nonlinear elastic ground model described above. The analyses were iterated to achieve comparable movement predictions, the latter calculation providing subgrade stiffness values for use in the former that were consistent with the expected load changes and associated ground response across the raft footprint.

When including settlement reducing piles (SRP) in the calculation, rather than modelling them as a spring, it was assumed that they acted as a constant load. This was deemed acceptable on the basis that they would be settling 25 mm to 40 mm, at which point the shaft resistance would be fully mobilised.

Calculations were undertaken for three stages of loading / soil response:

1. Undrained excavation using the undrained stiffness profile in Fig. 5; this analysis suggested heave up to 30 mm might occur.
2. Semi-drained net loading using a stiffness profile based on $E' = 0.75E_u$ which was considered to be a reasonable estimate for the situation at the end of construction when it was estimated that 40±10% consolidation might have been achieved.
3. Drained net loading including uplift due to groundwater using a stiffness profile based on $E' = 0.50E_u$ which was assessed using elastic theory and the degree of anisotropy in stiffness suggested by Pennington et al. (1997), i.e. $G_{0(hh)}/G_{0(hv)} \approx 2$.

3.2 *Settlement control and role of SRP*

In the design, an overall factor-of-safety with respect to bearing capacity failure in excess of three was demonstrated for the raft. However, mitigation measures were required in order to:

- Reduce excessive contact pressures and limit raft settlements to less than 40 mm, as a plain raft was expected to settle 40 – 60 mm with local maxima of 60 – 90 mm under heavily loaded columns and building cores.

- Limit raft settlements along sensitive boundaries to less than 25 mm; in the absence of SRP where a similar range of settlement values as indicated above were expected.

- Minimise net load changes adjacent to the diaphragm wall on the hotel boundary which was achieved by introducing SRP to limit the contact pressures on this boundary to approximately the same values present prior to the redevelopment.

- Minimise movements of bearing piles supporting the hotel which with the measures introduced were estimated to be less than 1.5 mm.

This was achieved by the introduction of SRP at the required locations. Their use in the first instance is well documented however it is thought that this is the first such application in terms of mitigating potential impacts outside the site boundary.

4 CONCLUSIONS

Use of precedent knowledge of the ground's response to load change has allowed a calculation model to be derived that is better conditioned to predict ground movements in Gault Clay.

In this case, the use of a piled raft, though more complex to design, provided a clear cost and time advantage to a fully piled solution. Furthermore, the use of a well-conditioned pseudo-nonlinear elastic soil model allowed further savings by reducing the number of SRPs needed to achieve the performance criteria.

SRP have been employed in a novel way in order to limit vertical ground movements off-site by constraining those within the site boundary, and thus protect neighbouring buildings from potentially damaging movement.

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