From Minor to Major – The Evolution of Deep Foundations for Transportation Projects

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ABSTRACT – This paper will outline opportunities for more economic design of pile groups; a case history will illustrate the benefits of considering non-linear soil stiffness characteristics together with overall group capacity, rather than local factors of safety for piles within a group. A piled-raft can offer many advantages compared with a conventional pile group, however piled-rafts are currently under-used by practitioners. Some simple concepts and definitions will be described which have been found to be of practical benefit when considering the use of piled-rafts. The paper will also outline recent challenges associated with the design of deep foundations for "over-site developments (OSDs)" above underground metro stations. The paper will give examples of OSD foundation design, the first is for a compensated piled-raft in stiff clay, the second is for an innovative solution which combined deep ground improvement with piles for a site underlain by very soft clays.

1. INTRODUCTION

This paper will outline the evolution of deep foundations for transportation projects from the conventional requirements for bridge foundations, for highway and railway projects, through to the more complex foundation requirements for large "over-site developments (OSDs)" above deep underground metro stations. Figure 1 illustrates a rail bridge which will be discussed in greater detail later in this paper. The pile group foundations for this bridge are conventional, however at the current state of practice there is still confusion amongst practitioners about how code requirements should be applied for pile group design. This often leads to over-conservative design and associated buildability problems. The case history outlined in this paper will outline how a more economic and buildable pile group design was delivered.

Fig 1. Rail Bridge founded on conventional pile groups.

Figure 2 illustrates an OSD for a metro project in London. The OSD is often a crucial component of the business case for modern metro schemes being built beneath major urban areas. Property values in cities such as London and Singapore are extremely high, hence there is a high financial return if the surface space above underground metro stations can be maximized, typically for large towers. This creates some novel and challenging geotechnical design issues. First there is the need to build deep retaining walls to create the underground space for the metro station, whilst minimizing ground movements beneath adjacent infrastructure. These underground structures are usually buoyant (since they are usually well below the water table) so tension piles are required to restrain the upward movement of the metro box. Second the OSD will add a large vertical load and this will need to be resisted by the retaining walls, piles and base slab of the metro box. The design of these "compensated piled-rafts" (O'Brien et al, 2012) is complicated by the need to minimize differential movements across the connecting tunnels which enter the metro box. The design of this type of piled-raft needs to take account of a greater range of factors than is the case for a piled-raft for an isolated building. In particular, the assessment and control of differential movement and the wide range of applied loads, from large tensile to large compressive forces, are important considerations. A recent OSD foundation design will be discussed and key issues highlighted.

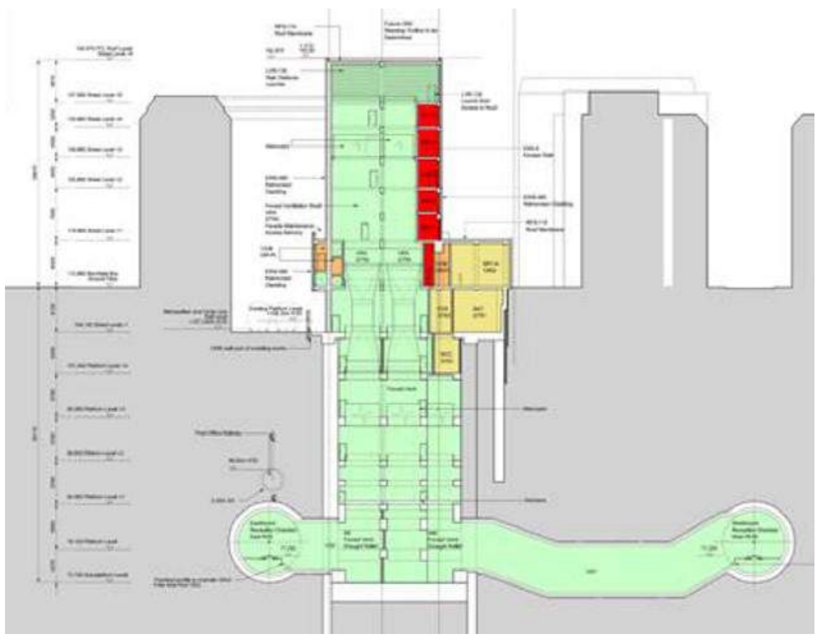


Fig.2. OSD and underground Metro station founded on compensated piled-raft.

Large coastal cities, such as Singapore, are creating additional land for development from coastal reclamation schemes. Singapore is underlain by deep deposits of very soft clay which undergoes large consolidation settlement under the loads applied by the coastal reclamation fill. This settlement can take decades to be completed. Design and construction of underground metro projects though this consolidating soft ground needs to limit the time dependent movements to avoid damage to tunnels and OSDs built within these transportation corridors. An innovative design which combined deep ground improvement and OSD pile foundations will be described. This innovative design created significant time/cost and carbon savings.

This evolution from conventional bridge foundations to foundations for OSDs above metro stations represents a significant increase in complexity, from "minor to major". Although, as discussed, there is a still a need for improvements in industry practice for the apparently simple requirements of pile groups for bridge foundations.

2. CURRENT INDUSTRY PRACTICE AND INDUSTRY DRIVERS

The foundation engineering industry is seeing a demand for deeper and larger foundations and retaining walls. For example, in London, the Crossrail project required several underground structures in excess of 40m deep. Shaft diameters up to 60m are required for the Thames Tideway project. Piled foundations deeper than 60m are becoming more common, to support large towers and OSD structures. These large underground structures are often being built close to existing, often fragile, existing infrastructure. In urban areas the "underground space" is becoming as congested as the areas above ground surface. Potential interactions between new and existing underground construction are often a key issue for designers. Previous underground construction will affect a site's stress history and lead to significant changes of in-situ effective stress. These considerations mean that current empirically based design methods often need to be seriously questioned. However, the industry, both in the UK and elsewhere, is constrained by:

i) Fragmentation – consultants, specialist and main contractors are often working in isolation. This "silo" working inhibits knowledge sharing;
ii) Design and construction are often carried out under aggressive commercial contracts, with unfair risk sharing;
iii) Design codes and specialist piling contractors focus too heavily on the geotechnical ULS capacity (or Factor of Safety) of a single, isolated, pile (this is often exacerbated by the performance requirements usually stipulated for piling contracts with the emphasis on proving the capacity of a single pile under a test load);

Because of the above constraints there is often limited opportunity or motivation for innovation. Excessively long piles, with congested reinforcement, are difficult to build and can create health and safety risks for piling operatives and concerns about foundation durability. Delays during piling construction have severe consequences for the whole project, because foundation construction is invariably on the critical path for any civil engineering project. Hence, innovative design has more significant value than simply saving material costs for the foundation elements of a project.

The UK Government Strategy document "Construction 2025" (2013) is aiming to stimulate a step change improvement in the construction industry through improvements in planning, procurement, training and embracing digital technology. The drivers for change include:

a) Increasing urbanization, globally there is anticipated to be a 50% increase in the urban population by 2050;
b) Climate change and increasing environmental concerns, and the need for low carbon, sustainable construction;
c) Opportunities created through the revolution in digital technology;

d) In many mature economies, professionals with specialist skills (such as geotechnics) are becoming scarce relative to future demands.

Construction 2025 has the following ambitious targets: a 50% reduction in greenhouse gas emissions; a 33% reduction in costs; a 50% reduction in time for project delivery. Foundation design/construction is a small part of the whole construction industry, nevertheless it can make a considerable contribution towards achieving these targets because its overall impact (either positive or negative) is disproportionate to its size (Clayton, 2001).

3. OVERVIEW OF PILE GROUP DESIGN

The responsibility for deep foundation design is often split, between the main design consultant (who is often responsible for the superstructure and sub-structure, including a pile cap) and the piling contractor (who is often responsible for pile design, typically the performance criterion is to demonstrate that the pile capacity and settlement of a single pile meets a notional design load specified by the main consultant), Figure 3. This division of design responsibility often leads to poor design of the overall foundation system. Ground-structure interaction (between piles, pile cap and the ground) is often not properly considered within this type of contractual arrangement. Current codes (eg Eurocode 7) offer limited guidance on pile group design which often leads to over-conservative design.

If the pile cap is relatively "stiff" then pile axial loads will be non-uniform, with high axial loads in piles around the perimeter of a pile group, especially at corner piles, Figure 4 (as discussed in text books such as Fleming et al 2009, Viggiani et al 2012). Figure 5 (discussed by O'Brien 2012) compares calculations of peak axial loads, for pile groups of varying size up to 100 piles (using pile group analysis software with either linear elastic or non-linear soil models and assuming the pile cap is rigid), with field observations. Figure 5 (a) indicates that analyses assuming linear elasticity become increasingly over-conservative as pile group size increases. The analyses assuming non-linear ground behavior (assuming a hyperbolic model for pile load-deformation behavior) lead to a closer match between observations and calculations. Nevertheless, Figure 5 (b) indicates that the peak axial loads in corner piles (even if assuming non-linear behavior) will inevitably approach the ultimate pile capacity as the size of the pile group increases. This non-uniform distribution of axial loads often raises concerns and can be mis-interpreted by designers, who may increase pile lengths in an attempt to ensure that each and every pile in a group has a code-compliant "factor of safety" (against geotechnical capacity). This particular pitfall has been highlighted by several eminent geotechnical engineers (eg Burland, 2006; Randolph, 2003); nevertheless, this wasteful design strategy remains common. The correct strategy is to assess the overall capacity of the pile group and check that the pile cap has sufficient strength and stiffness to redistribute axial loads. If the pile cap has adequate structural strength and stiffness, then the designer should not be concerned about "local" factors of safety for individual piles within a group. In the author's experience, the key pile group design issues are:

1. Checking the structural strength and stiffness of the pile cap and structural capacity of the piles;
2. Checking that pile group deformation is acceptably small relative to the serviceability limits for the superstructure.

The overall geotechnical capacity of a pile group is rarely a critical issue. It is also interesting to note that Viggiani et al, 2012, (discussing long term observations of axial loads in pile groups), indicate that creep effects (of the piles and/or of the reinforced concrete pile cap) can lead to a substantial reduction of the initial non-uniform distribution of axial load across large pile groups.

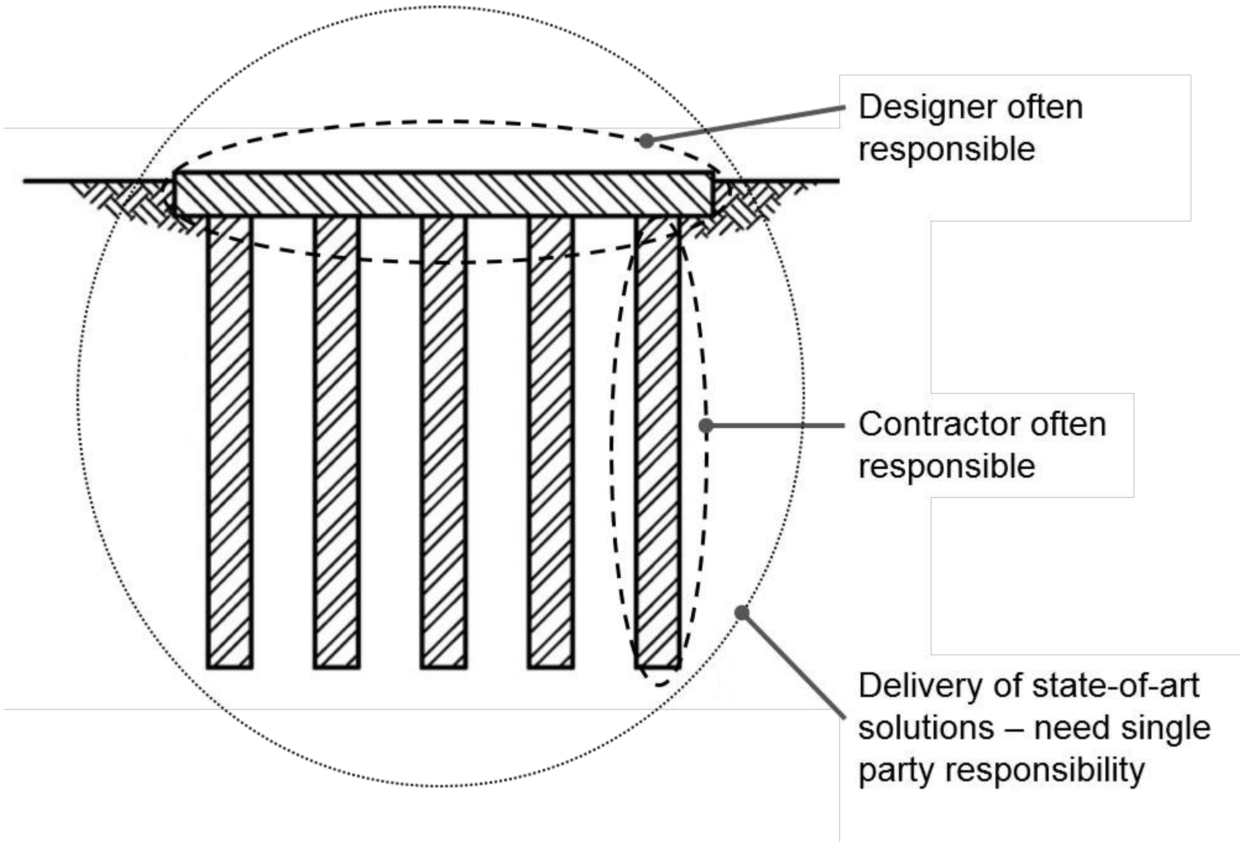


Fig. 3. Potential fragmentation of responsibility for pile group design.

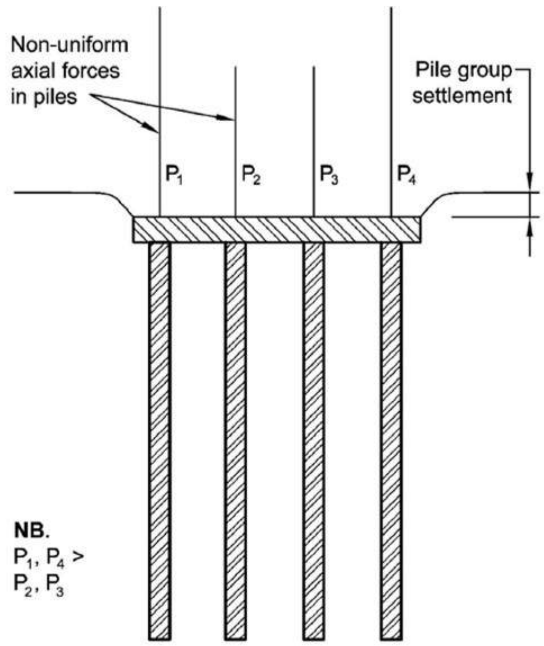


Fig. 4. Non-uniform axial loads across pile group with "stiff" pile cap.

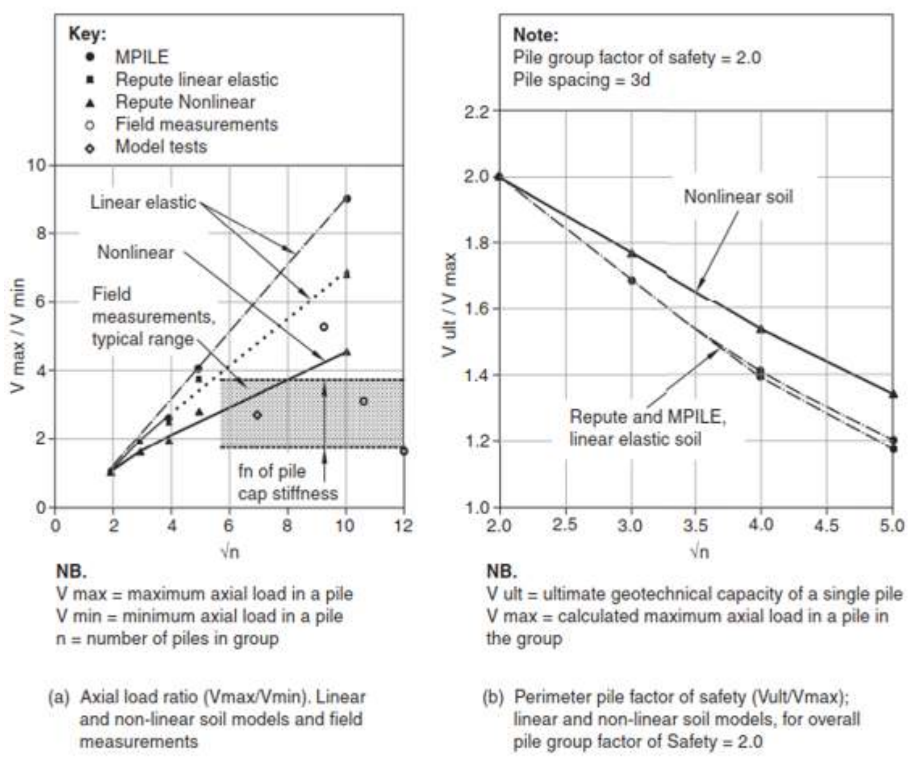


Fig. 5. Axial load distributions for linear and non-linear ground models versus pile group size.

Concerning the deformation of pile groups at working loads, studies by Mandolini and Viggiani, 1997, and Mandolini et al, 2005, have confirmed that, when using linear elasticity, the most appropriate soil stiffness is the shear modulus at very small strain, Go (or Gmax). The shear strains induced in the bulk of the ground mass adjacent to piles are very small (typically << 0.01% strain), Figure 6, as described by Jardine et al 1986. However, the empirical stiffness correlations often used in practice are based on relatively large mobilized strain amplitudes (in excess of 0.1%), which can lead to pile group deformation being substantially over-estimated (by factors of 2 to 3 or more).

Bridge foundations usually impose large lateral and moment loads onto pile groups and then the choice of ground stiffness can have a significant impact on calculated pile bending moment, as well as foundation deformation. Figure 7, from Hardy and O'Brien 2006, shows that the calculated bending moments in piles within a group are highly sensitive (varying by a factor of about 3) to the assumed stress-strain model (hyperbolic or linear elastic) and, if linear elastic, to the assumed secant moduli (ie choice of modulus at small, intermediate or large strain amplitude). For pile groups subject to large horizontal/moment loads non-linear stress-strain models (such as the hyperbolic model in software such as REPUTE) are likely to be most appropriate (as discussed by O'Brien 2007).

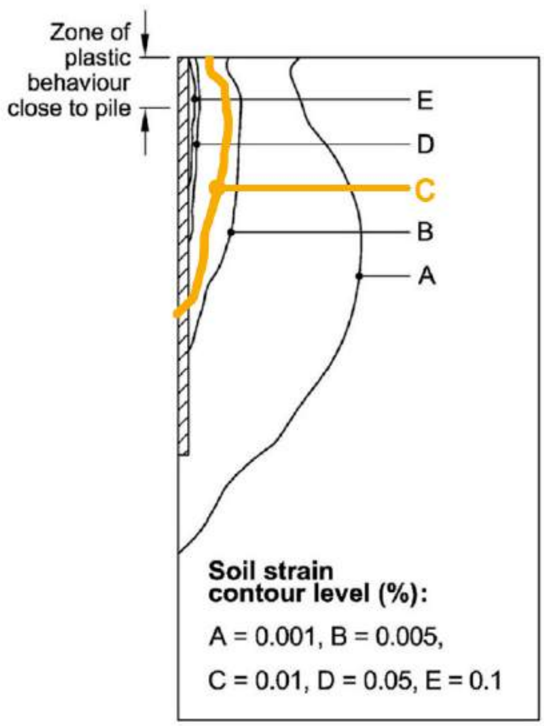


Fig. 6. Contours of shear strain around a pile at working load.

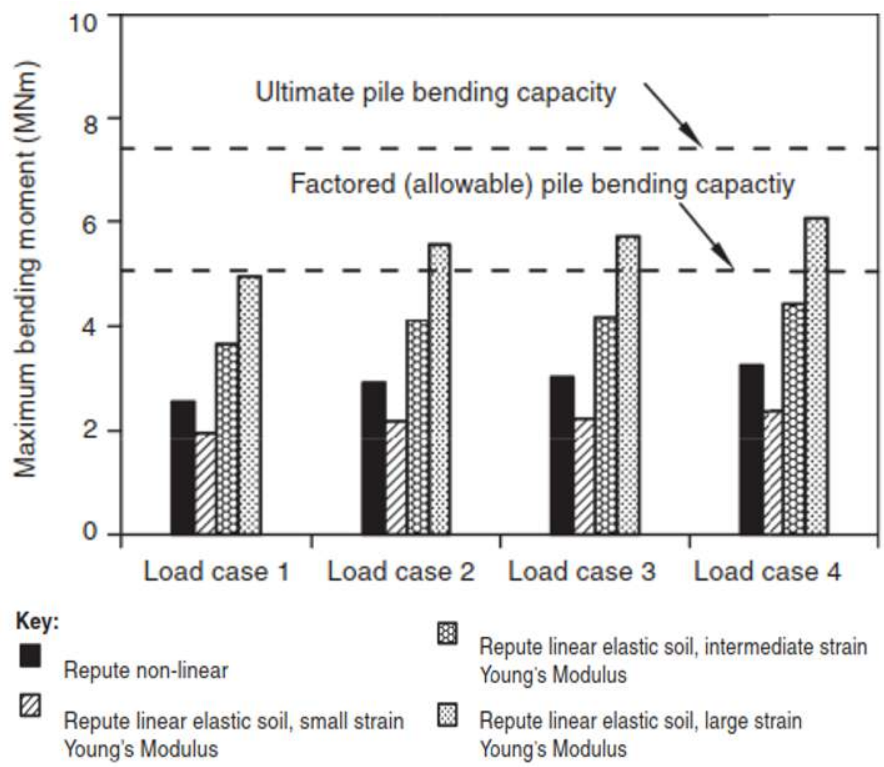


Fig. 7. Influence of soil model and selected Young's modulus on peak bending moment under horizontal loading.

4. PILE GROUP DESIGN - CASE HISTORY

A new highway scheme crosses beneath an existing railway, Figure 1. The railway was on an embankment and had to remain operational except for a short (4 day) "track possession" period to allow the new bridge deck to be jacked into position. The track possession period was fixed with the rail authorities several months in advance and everything had to be ready at this time for the deck installation. Delays to foundation construction could not be tolerated. Hence, pile installation had to be carried out adjacent to the existing embankment and operational railway. The rail bridge was at about 60 degrees to the highway alignment and this together with the physical constraints of the rail embankment meant that the four pile groups, which would act as foundations to the bridge, would experience large moment and horizontal loads (the bridge deck bearings were offset from the centroid of the pile groups), Figure 8.

The site geology comprised alluvium over glacial deposits, and the groundwater table was close to the ground surface. The original ground investigation (based on routine sampling, lab tests and SPTs) indicated an interbedded sequence of sands and clays (with a thick clay layer at proposed pile toe level), Figure 9(a). Based on the ground and groundwater conditions, CFA piles were preferred, being quicker and cheaper to install than conventional bored piles (which would require temporary support using casings and/or slurry). However, for CFA piles there were concerns that the reinforcement cage (to cope with the large applied moments) would be too heavy to install in a CFA pile. A preliminary design was prepared assuming 23m deep 900mm dia CFA piles in a 6 by 3 pile group. Based on the original GI data, non-linear pile group analyses were carried out (using the hyperbolic model in REPUTE). These analyses highlighted the importance of the stiffness of the deep glacial clay layer, below pile toe level, in controlling the overall rotation of the pile groups under high moment loading. There was also a serious concern that installation of the 23m deep CFA piles would be too time consuming, especially if the ground was stronger than the original GI indicated.

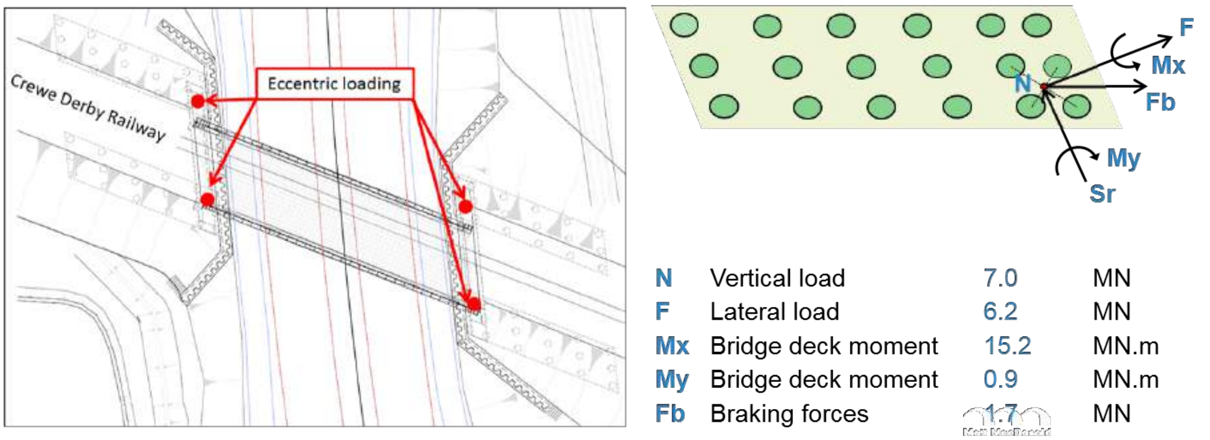


Fig. 8. Case history-applied vertical, moment and horizontal loads.

Additional investigations and preliminary pile tests were therefore planned to confirm design parameters. CPT, Piezocone and seismic cone testing was carried out, and specialist laboratory tests included bender element tests (Clayton, 2001). The CPT and Piezocone tests indicated a more complex soil layering than the original GI, especially at depth (below 9.5m bgl) were laminated clays and silts were identified, Figure 9 (b). The seismic cone and bender element tests gave a Go profile which was similar to the original "best estimate" profile at shallow depths (estimated from several different empirical correlations) but indicated a more rapid increase of Go at depths in excess of about 15m, Figure 10 (a) and 10 (b). The seismic cone Go values were reasonably consistent and provided confidence in selection of a design profile.

The preliminary pile tests indicated a similar initial stiffness (at displacements up to about 0.5% of the pile diameter), Figure 11 and was also similar to that estimated on the basis of the original GI data (shown as "CEMSET best estimate). However, at larger pile displacements the differences between the 2 pile tests and with the original estimated behavior increased. The ultimate capacity derived from the 2 tests was different (3.9MN and 5.3MN at a displacement of 10% of the pile diameter). Back analysis of the tests (based on a hyperbolic curve fitting method described by Fleming, 1992, and England, 1999) suggested that the main difference in capacity between the two tests was due to differences in "drained" (or partially-drained) end-bearing in the silty layers at 23m depth (test pile 1 mobilized undrained end bearing, with an undrained shear strength of about 100 kPa, whereas test pile 2 mobilized drained end bearing with a friction angle of about 26 to 27 degrees.) However, the piles were basically acting as friction piles (with both piles mobilizing a shaft friction of about 3.3MN) and the shaft friction response was better characterized by effective stress rather than total stress methods, (assuming ks = 1.0, and δ = 25 degrees).

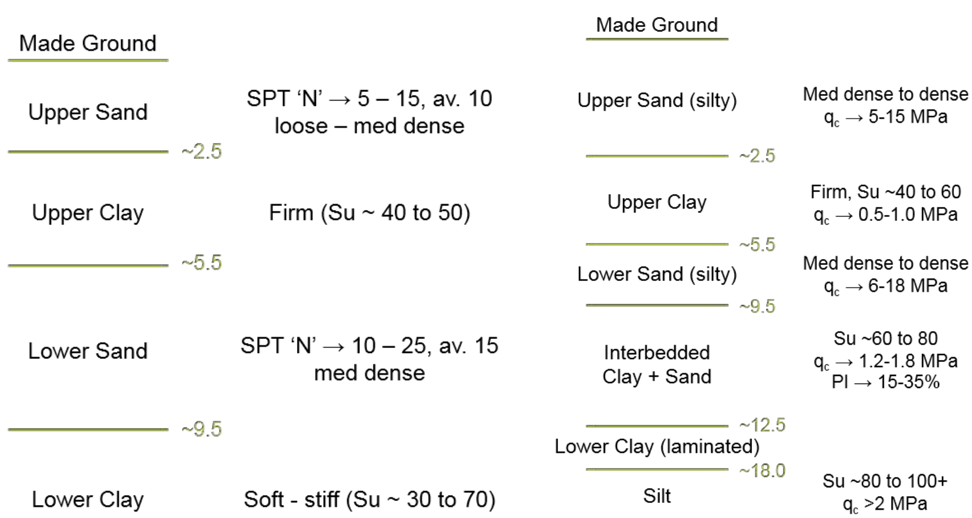


Fig. 9(a). Original ground profile. Fig.9(b). Revised Ground Profile.
Fig 9. Ground profiles derived from original and additional ground investigations.

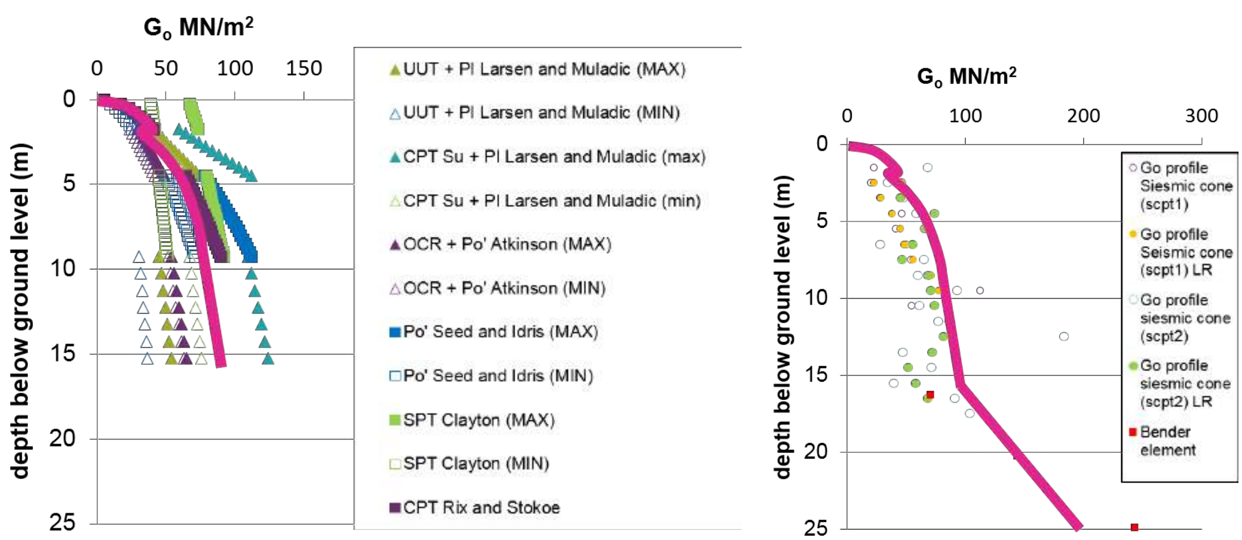


Fig. 10(a). Original profile (empirical). Fig. 10(b). Revised profile (direct measurement).
Fig 10. Shear moduli profiles derived from original and additional ground investigations.

Fleming's method was used to simulate the load-settlement response of a single pile, and the hyperbolic model in REPUTE was also calibrated against the pile test data, Figure 12. This calibrated model of single pile behaviour, together with the Go profile derived from the additional ground investigation, was incorporated into a REPUTE model of the pile group which was then used to assess the pile group deformation behavior under vertical, moment and horizontal loads. This revised model indicated that the pile groups were likely to be much stiffer than originally assumed, Figure 13. This then facilitated a re-design of the pile groups – the front 2 rows of piles were kept at 23m long, but the remaining rows of piles were shortened to 15m. Pile reinforcement was also able to be reduced, which reduced construction risks. This enabled a significant overall saving in pile quantities (about 25%) and more critically a saving in the foundation construction schedule.

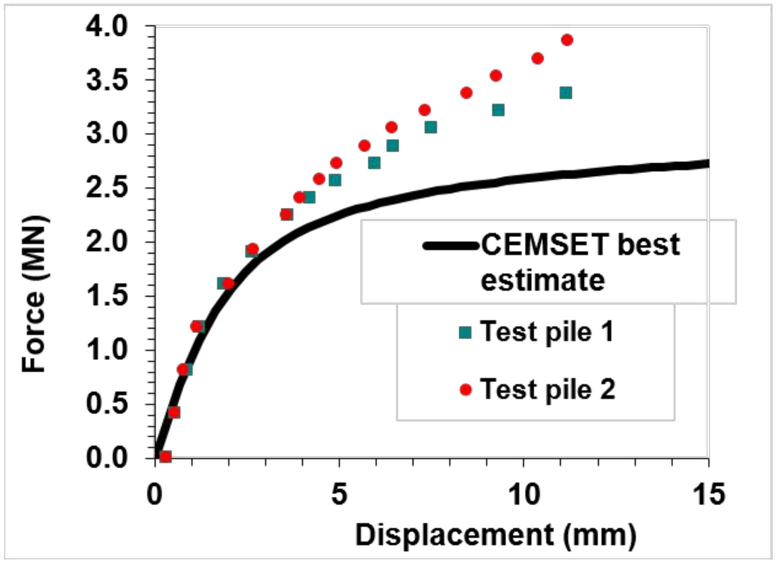


Fig 11. Pile load test data versus estimated (original GI).

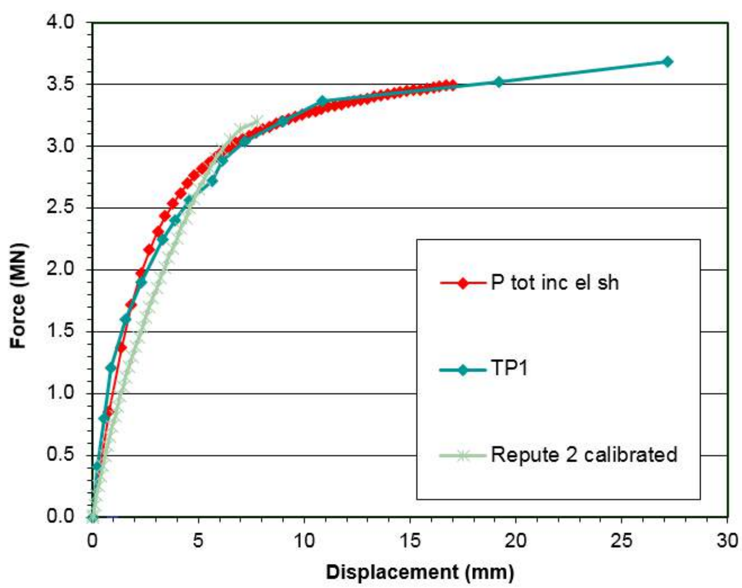


Fig. 12. Pile load test data and calibrated models.

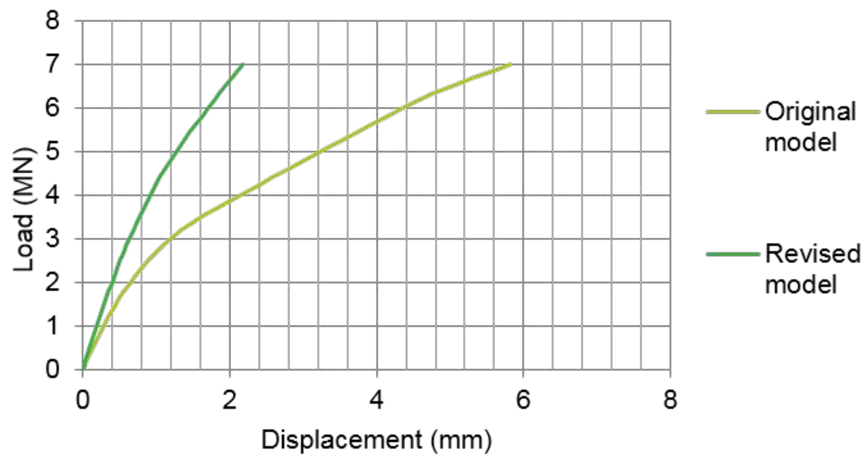


Fig. 13. Pile group settlement, original and revised REPUTE models.

However, the biggest challenge was to get the necessary design approvals from the technical advisers to the rail authority and the independent checker (required under this contract). There were serious concerns because some of the piles in the pile group were mobilizing most of their geotechnical capacity, Figure 14, with about 7 out of 18 mobilizing axial loads in excess of their code factored ultimate capacity. However, the remaining 11 piles had a "reserve" capacity which was more than adequate to deal with the applied bridge loads. The pile cap and bridge sub-structure were very stiff structurally (pile cap stiffness, Krs > 5, as defined by Horikoshi and Randolph 1997) and the pile load-settlement behavior was ductile, Figure 12. Using an approach similar to that outlined by Randolph 2003, it was possible to demonstrate that the overall foundation system was robust and had a more than adequate geotechnical capacity. Following prolonged discussions, the pile group design was approved. Foundation construction was successfully completed. Following bridge deck installation, the observed pile group deformation was smaller than estimated, Figure 15.

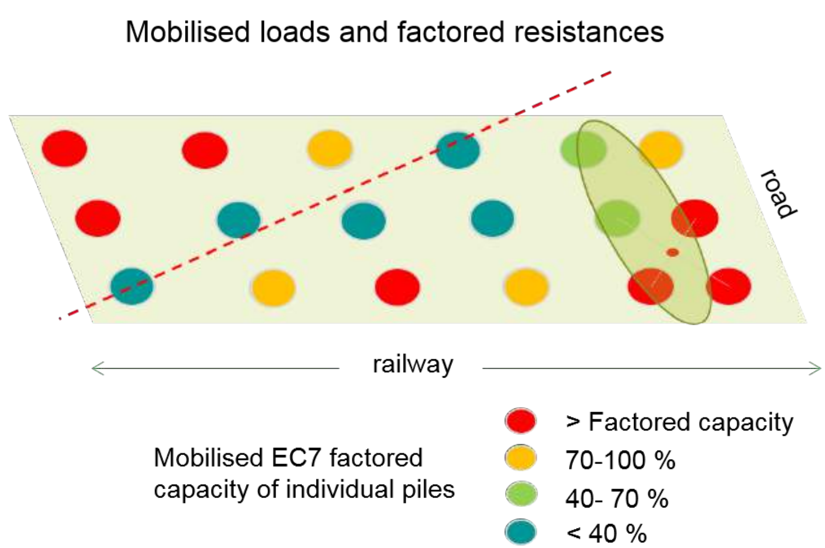


Fig. 14. Mobilized axial load as percentage of code factored resistance.

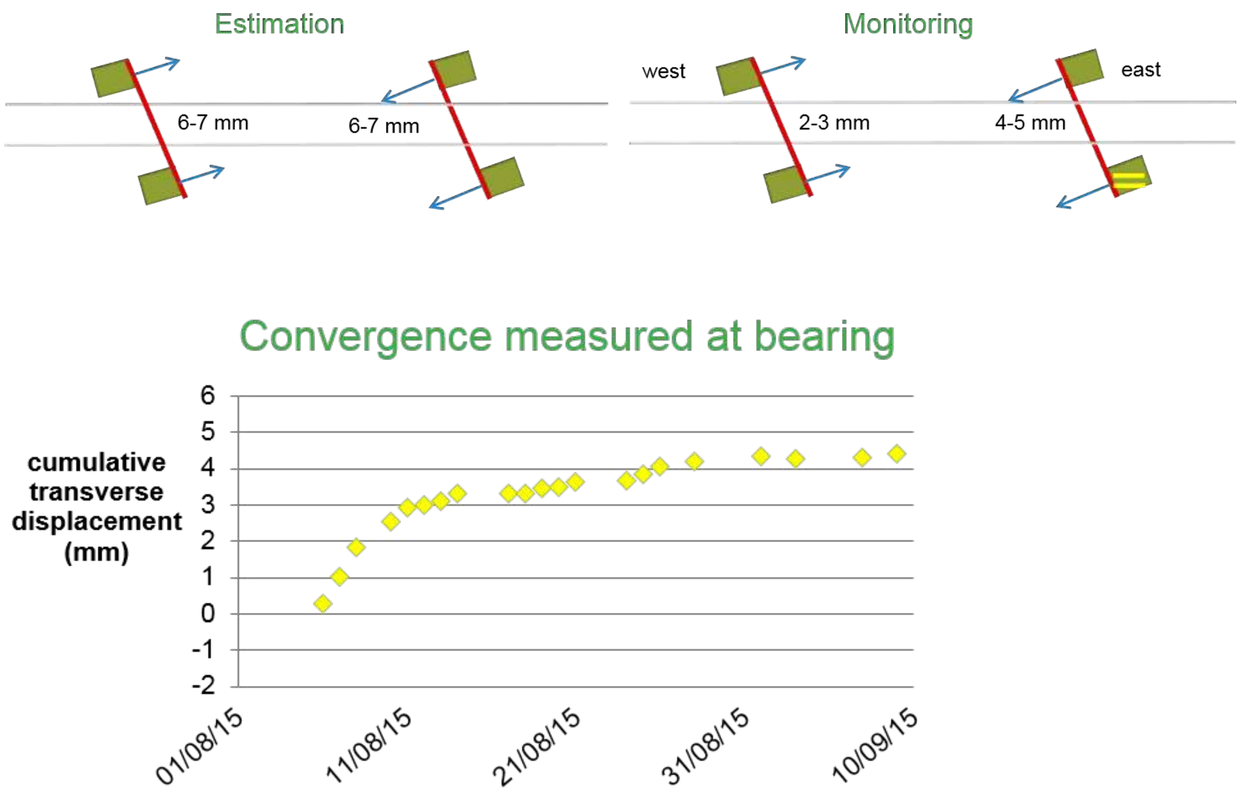


Fig. 15. Predicted (revised calibrated model) versus observed pile group deformation.

5. PRACTITIONER'S VIEW OF PILED-RAFT DESIGN – A SIMPLE CONCEPTUAL FRAMEWORK

Conventional design of pile groups ignores the resistance provided by the pile cap, with all load assumed to be resisted by the piles. Several research studies (eg Randolph 1994, Mandolini et al 2005) have shown that when pile caps are in contact with competent soils (such as stiff clays and dense sands) that this assumption is highly conservative, and the applied loads are shared between the pile cap (acting as a raft) and the piles. The ground-structure interaction behavior of piled-rafts is potentially highly complicated (eg Katzenbach et al, 1998), however, it is helpful for practical design purposes to have clarity on some of the main features of behavior prior to detailed analysis. In the UK, piled-raft design guidance has been provided in the ICE Manual of Geotechnical engineering (O'Brien et al, 2012). The main objective was to provide a simple conceptual framework for piled-raft design to encourage practitioners to make more use of this type of foundation. This simple framework is briefly summarized below. It is helpful to consider two types of piled-raft, Figure 16:

(i) A "raft-enhanced" pile group, Figure 16(a) – the extra capacity provided by the pile cap or raft allows fewer piles to be used to provide the same margin of safety against failure and similar (typically small) settlement as a conventionally designed pile group.

The piles will typically be at a wider spacing than a conventionally designed pile group, which enables the pile group to work more efficiently;

(ii) A "pile-enhanced" raft, Figure 16(b) – a small number of piles are used to reduce peaks in shear and bending moment in the raft and control differential settlement, typically piles are located beneath heavily loaded columns. The ultimate geotechnical capacity of all the piles are fully mobilized.

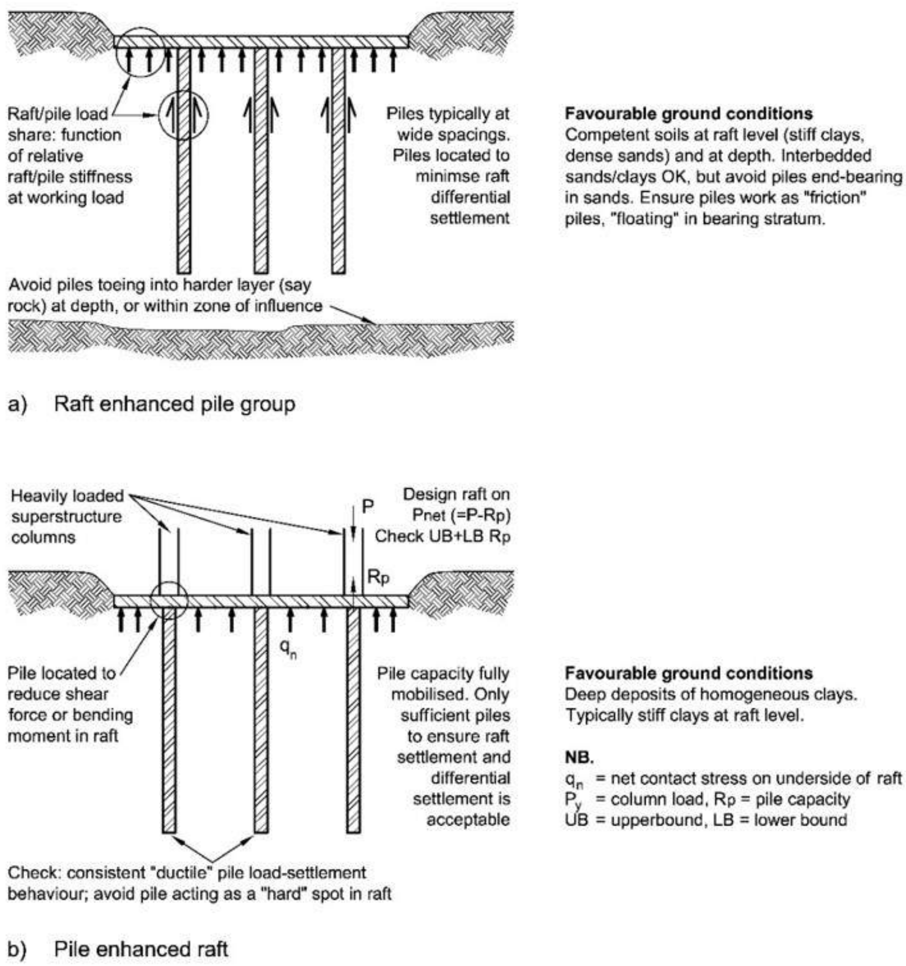


Fig. 16. Raft-enhanced pile group versus pile-enhanced raft.

The difference in design philosophy is illustrated in Figure 17 which shows a sketch of foundation settlement versus number of piles. Conventionally designed pile groups and rafts are shown at the extremes of the diagram. A raft-enhanced pile group plots close to a conventionally designed pile group, with relatively low foundation settlement achieved from more efficient use of a pile group. In contrast the pile-enhanced raft exhibits greater settlement than a raft-enhanced pile group, but only a small number of piles are required. As indicated in Figure 17 there is an intermediate zone, between the two types of piled-raft which should be avoided. Typically, in this intermediate zone there is a risk that, for the wide range of different load cases which typically need to be considered, that one part of a raft may experience relatively "soft" support from underlying piles (as their capacity is fully mobilized) whereas adjacent parts of a raft experience relatively "stiff" support from underlying piles which are still operating within their pseudo-elastic range. Poulos, 2001, suggested a simplified tri-linear load settlement curve for preliminary design of piled-rafts, Figure 18. The initial settlement behavior is pseudo-elastic up to a load P1when the ultimate capacity of the **pile group** is fully mobilized. For loads beyond P1 the piled-raft stiffness is controlled by the raft stiffness only. As indicated in Figure 18, raft-enhanced pile groups operate within the initial "elastic" zone, whereas pile-enhanced rafts operate within the second zone. If this conceptual framework is followed then the design process can be simplified.

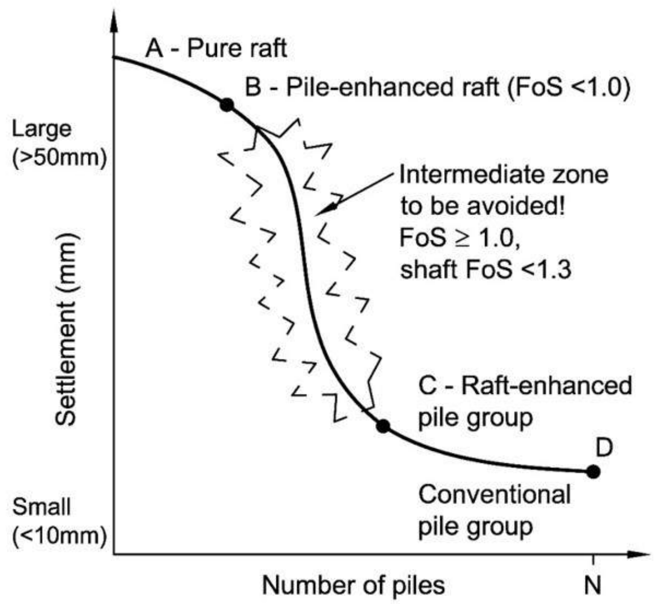


Fig. 17. Relative stiffness of raft-enhanced pile groups versus pile-enhanced rafts.

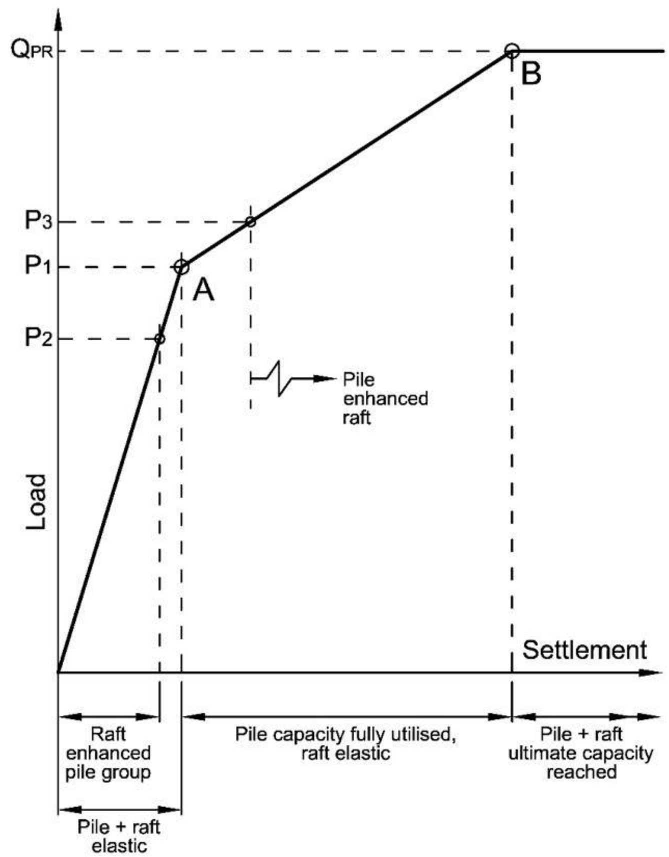


Fig. 18. Simplified load-settlement curve for piled-rafts.

For conceptual and preliminary design purposes, there are a number of simple empirical and theoretical relationships which can be used, these are discussed in the ICE Manual (O'Brien et al, 2012). An example is shown in Figure 19, which summarizes case history data collated by Mandolini et al, 2005, on the load sharing observed between raft and piles. These types of empirical relationships are extremely valuable for designers to refer to before embarking on detailed site-specific analysis.

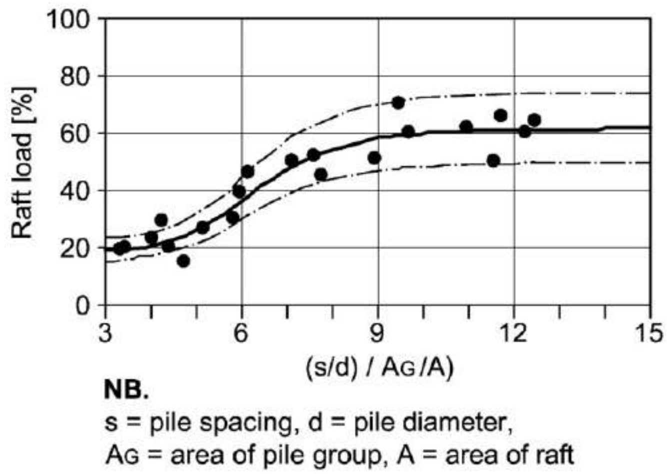


Fig. 19. Piled-rafts, raft load share (case history data).

Burland, 1995, and Katzenbach et al 2000, describe how the presence of a raft modifies the behavior of piles below a raft compared with free standing piles. Mandolini et al, 2005, discuss the performance of a raft-enhanced pile group for steel tanks in the Port of Napoli. Burland and Kalra, 1986, and Love, 2003, discuss examples of pile-enhanced rafts for building foundation projects in London. Pile-enhanced rafts typically comprise "large" rafts (Viggiani et al, 2012), with the raft width, B, larger than the pile length, L. The pile spacing is typically > 8d (d is pile diameter), hence pile-pile interaction can be assumed to be negligible. In contrast, raft-enhanced pile groups can encompass a wide range of pile and raft geometries, typically with pile spacings greater than 4d to enhance the efficiency of the pile group.

6. DEEP FOUNDATIONS FOR OSD'S, ABOVE UNDERGROUND METRO STATIONS

A special type of piled-raft is a "compensated piled-raft" – this piled-raft is located at the base of a deep excavation, usually well below the water table, Figure 20. Because of the large reduction in over-burden stress and the buoyancy force applied by water pressure, there are several additional factors which need to be carefully considered during design (eg Sales et al, 2010), including:

(i) Pile behavior and load sharing between the raft and piles is affected by the excavation and pile installation sequence;

(ii) The excavation is usually supported by deep embedded retaining walls, which will also pick up loads (hence, loads will be shared between piles, retaining walls and raft);

(iii) The presence and magnitude of the buoyant force (ie groundwater pressure) will have a significant effect on overall behavior and load sharing. The mass permeability of the ground will control the timing of the application of the full buoyant force (ie "long term" groundwater pressure), hence at intermediate time periods load sharing may be different to that in the long term. Any future changes in the groundwater regime will also influence overall behavior and load sharing;

(iv) Piles are usually subject to uplift forces and heave-induced tension at some point during and after construction, depending on the sequence of construction and the timing of excavation, pile installation and application of OSD loads (above the "box" excavation);

(v) OSD construction is often part of a separate development (metro owner will install the box and foundations and the OSD owner will build the OSD afterwards) and therefore the timing of OSD construction can be uncertain, which generates several additional load case scenarios which must be considered in the piled-raft design;

(vi) Large changes in effective stress (both negative and positive) may occur at different stages during the design life of the piled-raft. These will control the capacity and stiffness of the different components of the piled-raft, hence empirically based formula (in particular, total stress methods for clays) are often inappropriate for compensated piled-raft design. More fundamental methods, based on effective stress (eg Burland and Twine 1988) need to be used;

(vii) The embedded retaining walls potentially provide a high stiffness/capacity component of the overall foundation system. The capacity/stiffness of the walls will be influenced by the means/methods for installation and subsequent excavation support (eg bracing or ground anchors, top-down or bottom-up construction sequences, etc) - since this will influence the horizontal effective stress and interface friction mobilized along the retaining walls.

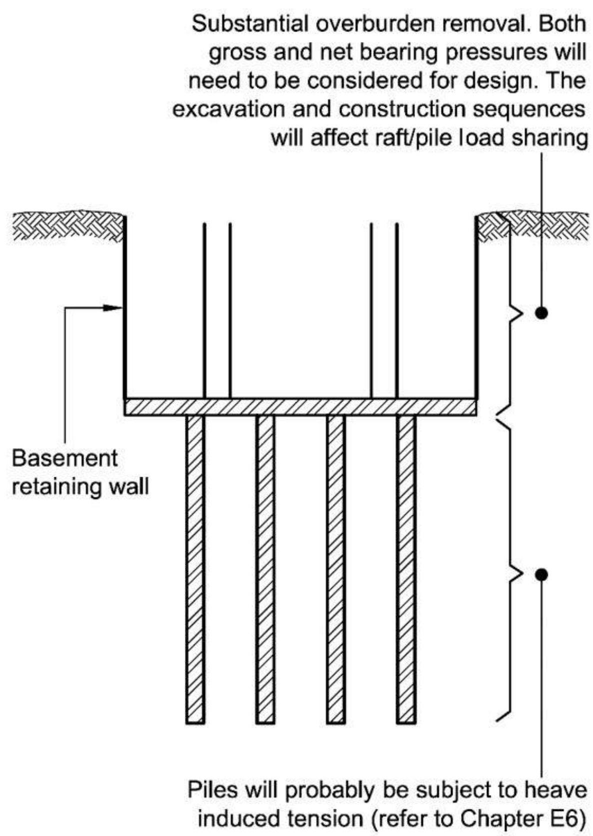
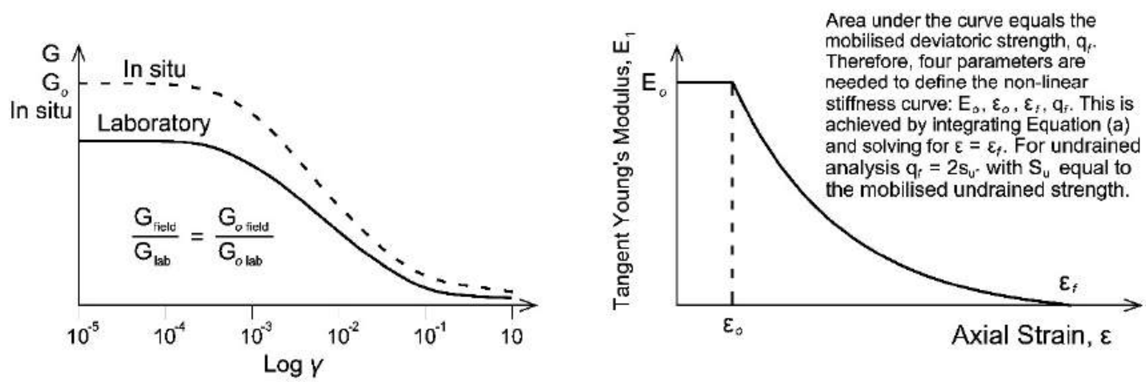


Fig. 20. Compensated Piled-raft.

Simplified analytical methods, based on linear elasticity are often sufficient for design of conventional pile groups, however for compensated piled-rafts full non-linear stress-strain models are usually necessary. It is important to have a reliable methodology for assessing the non-linear stiffness characteristics of the ground. This is a challenging topic, beyond the scope of this paper, nevertheless, given its importance a few matters need to be highlighted:

(a) Commercially available finite element software - a range of different advanced non-linear stress-strain models are now available. However, the specific type of model, the scope of GI necessary to obtain appropriate input parameters, how the stress-strain model can be checked and calibrated (eg, for different stress paths around/beneath the excavation) all need careful thought and review by experienced specialists;
(b) Some non-linear stress-strain models rely upon input parameters which either cannot be directly measured or rely upon relatively esoteric laboratory testing – reliance upon these types of models needs to be questioned. For example, even when high quality sampling methods are used, sample disturbance is problematic and typically becomes more severe as depth increases.

To address these issues Mott MacDonald have developed a parameter selection framework for characterizing non-linear ground stiffness, known as A*, based on research in Japan and the UK (Ishihara 1996, and Atkinson 2000) and is described by Eadington and O'Brien, 2011. The key advantages of the A* model is that it uses both in-situ and laboratory testing and the key input parameters are directly measurable (and can be cross-checked by several methods). Figure 21 summarizes the A* model, and this model has been shown to give reliable non-linear stiffness parameters for a range of geotechnical ground-structure interaction problems (including tunnels, foundations and deep retaining walls), O'Brien and Liew, 2018.



Notes: $$\frac{E_t}{E_0} = \frac{1-\left(\frac{\varepsilon_f}{\varepsilon}\right)r}{1-\left(\frac{\varepsilon_0}{\varepsilon}\right)r}$$ equation. (a)

Fig 21. Non-linear ground stiffness, A* model.

Figure 22 gives the cross-section through a metro station box being constructed in London. The compensated piled-raft is located at a depth of 22m below ground level, founded on very stiff London Clay. London Clay is underlain by the Lambeth Group (interbedded dense sands and very stiff clays) and Thanet Beds (predominantly very dense silty sands). The retaining walls for the Metro station box are formed by 1.2m thick diaphragm walls (D-walls) and the piles were 1.8m and 2.1m dia bored piles. The D-Wall is about 30m deep, but alternate D-Wall panels are extended as deep barrettes to provide OSD load carrying capacity, Figure 23 shows the 3D PLAXIS finite element mesh used for detailed analysis.

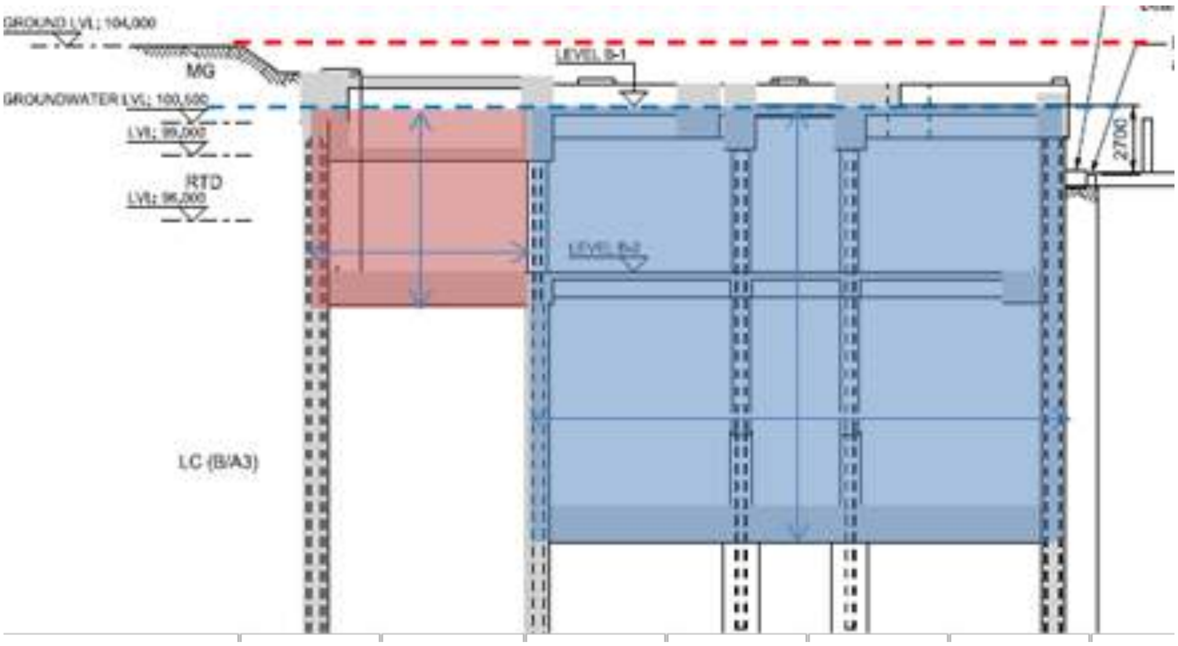


Fig. 22. Case history, OSD Compensated Piled-raft.

A 12m long representative section was used as the basis for the coupled effective stress analysis, which was used to simulate several different construction scenarios. Because of uncertainty about the timing of OSD construction the barrettes and piles also needed to act as tension elements to resist buoyancy forces (assuming OSD construction was delayed for several years). The key design question was the differential settlement across the Metro box and the connecting Metro tunnels. Figure 24 illustrates the predicted load share between the raft and the deep foundation elements (piles and barrettes) if the OSD construction followed immediately after completion of the underground station (with groundwater pressures equilibrating to long term values after OSD construction). It can be seen that load share changes markedly once the buoyancy force develops (note the “remaining” load share elements are the retaining walls for the metro box). An important benefit of the numerical modelling is to assess several “what-if” scenarios and assessing the impact of various design changes.

Figure 25 shows the impact of shortening the deep foundation elements from 62m to 41m, which led to an increase in differential settlement across the box from 1 in 1700 to 1 in 900. The shorter foundation elements had significant benefits, including: avoiding penetration of sand layers at depth (and associated use of bentonite slurry support), health/safety and buildability challenges (mainly related to handling large heavy reinforcement cages within a constrained site). These benefits need to be balanced against the increase in differential settlement, hence consideration of what is deemed to be an allowable maximum differential settlement is often a critical issue. This needs effective communication between structural and geotechnical engineers and realistic limits set for allowable differential settlement (these issues are also discussed by Burland 2006).

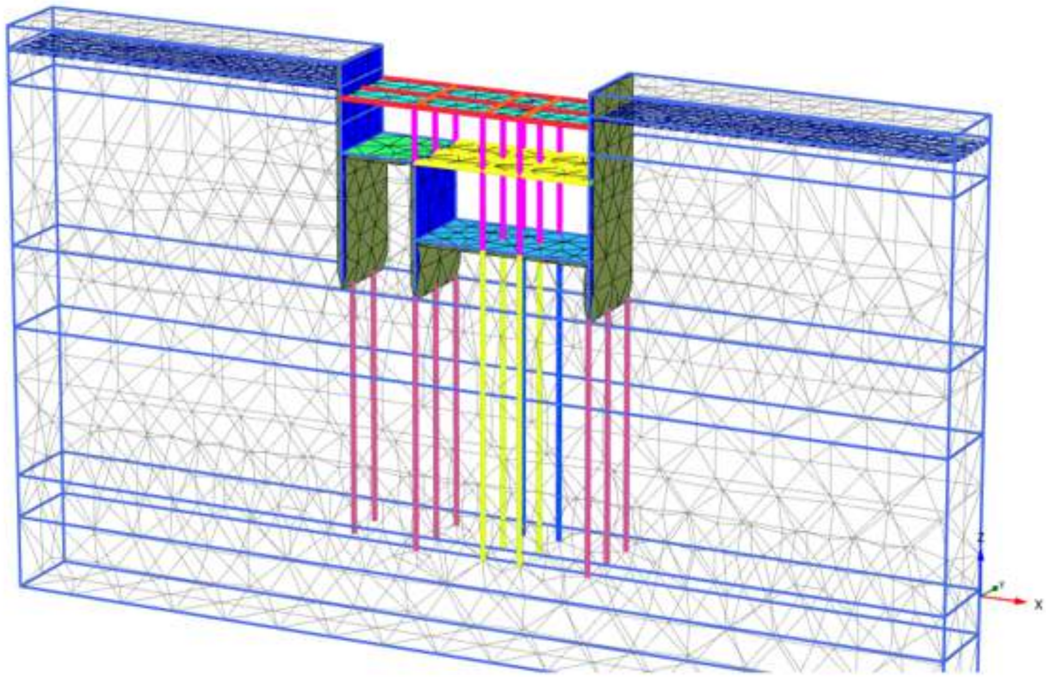


Fig. 23. 3D Numerical model mesh for OSD Compensated piled-raft.

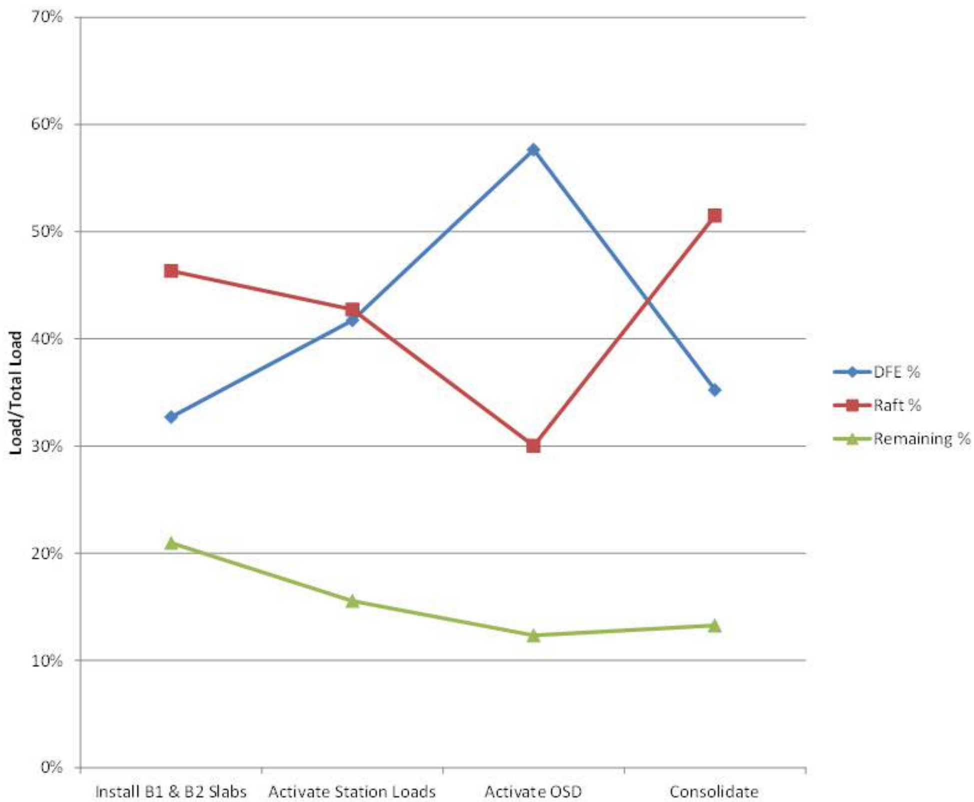
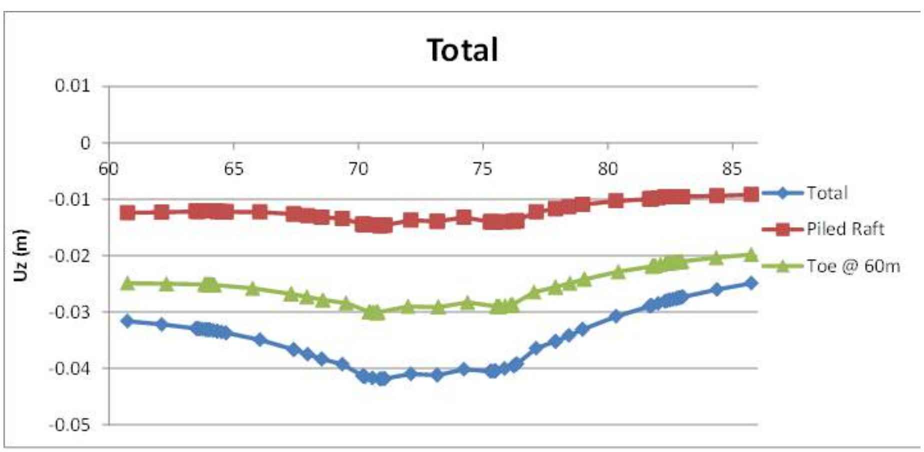


Fig 24. Variation of load sharing for a design scenario.



Note: "total"- raft only; "piled-raft"-deep foundations 62m deep; "toe@60m"- deep foundations 41m long.

Fig. 25. Settlement across piled-raft

7. INNOVATION – COMBINING DEEP GROUND IMPROVEMENT WITH PILES, A SINGAPORE CASE HISTORY

In Singapore, a part of a new metro line passes through recently reclaimed land. The deep deposits of very soft Marine clay are consolidating under the weight of the reclamation fill. Figure 26 shows CPT profiles compared against the vertical effective stress profiles (and assuming that the CPT derived undrained strength is equivalent to 0.22 times the vertical effective stress) for two different areas – Figure 26 (a) is for an area of old reclamation fill where the Marine clay has fully consolidated under the reclamation (built nearly 50 years ago), whereas Figure 26 (b) is for a more recent area of reclamation which is still actively consolidating. It is known from past monitoring that the overall magnitude of consolidation settlement can be circa 1.5m or more. Based on the age of the reclamation, the consolidation settlement which may affect the metro and OSD was expected to be more than 0.5m. Hence, a major challenge in the design of tunnels and OSD foundations is the drag force imposed by the consolidating clay. To mitigate these effects a deep ground improvement block was planned (using deep soil mixing, DSM) along the metro corridor, to provide a permanent load transfer system to the more competent "old alluvium" (at a depth of about 40m below ground surface) which underlies the soft Marine clay. DSM produces an improved material known as soilcrete, by mixing cement with the insitu soil. The DSM process is discussed in detail by Kitazume and Terashi, 2017. The future OSD which is planned along the metro corridor would conventionally be founded on deep piles embedded into the old alluvium (for the proposed scheme these would be 78m long, 1.8m dia piles spaced at 12m c/c).

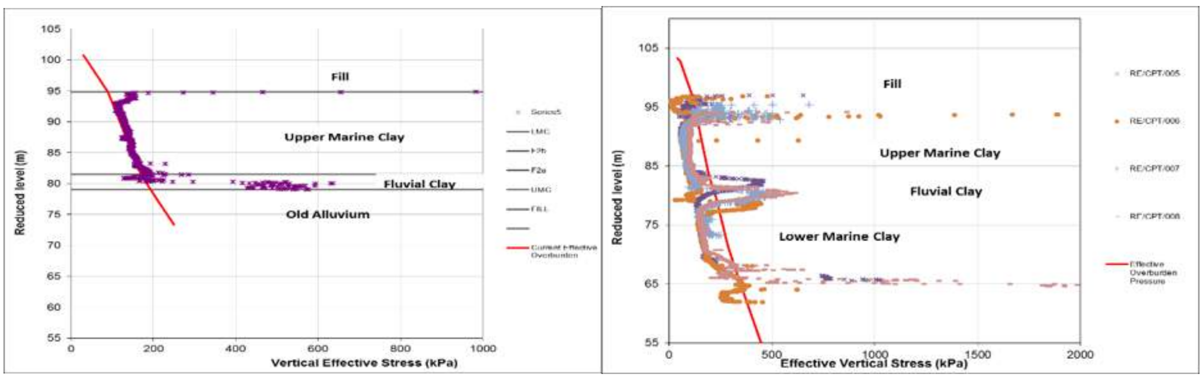


Fig. 26(a) Reclaimed land, full consolidation. Fig. 26(b) Reclaimed land, under-consolidated.
Fig 26. Reclaimed land - fully consolidated versus under-consolidated areas.

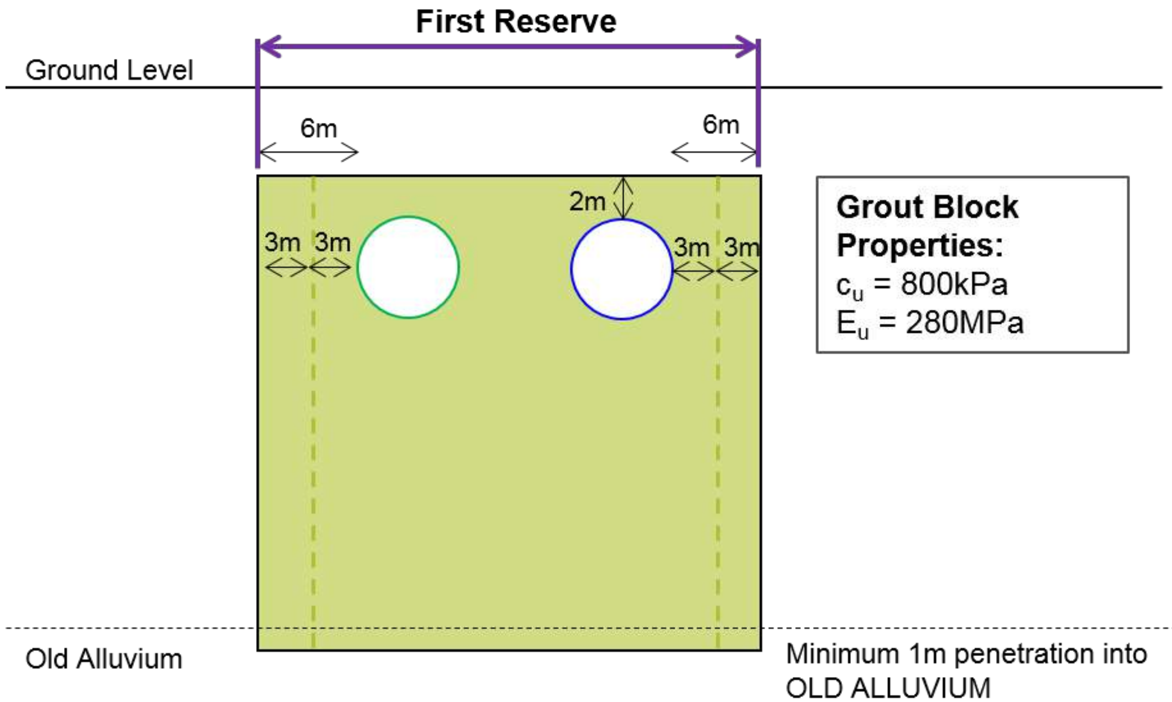


Fig. 27. Original design for ground improvement (DSM) to limit tunnel settlement.

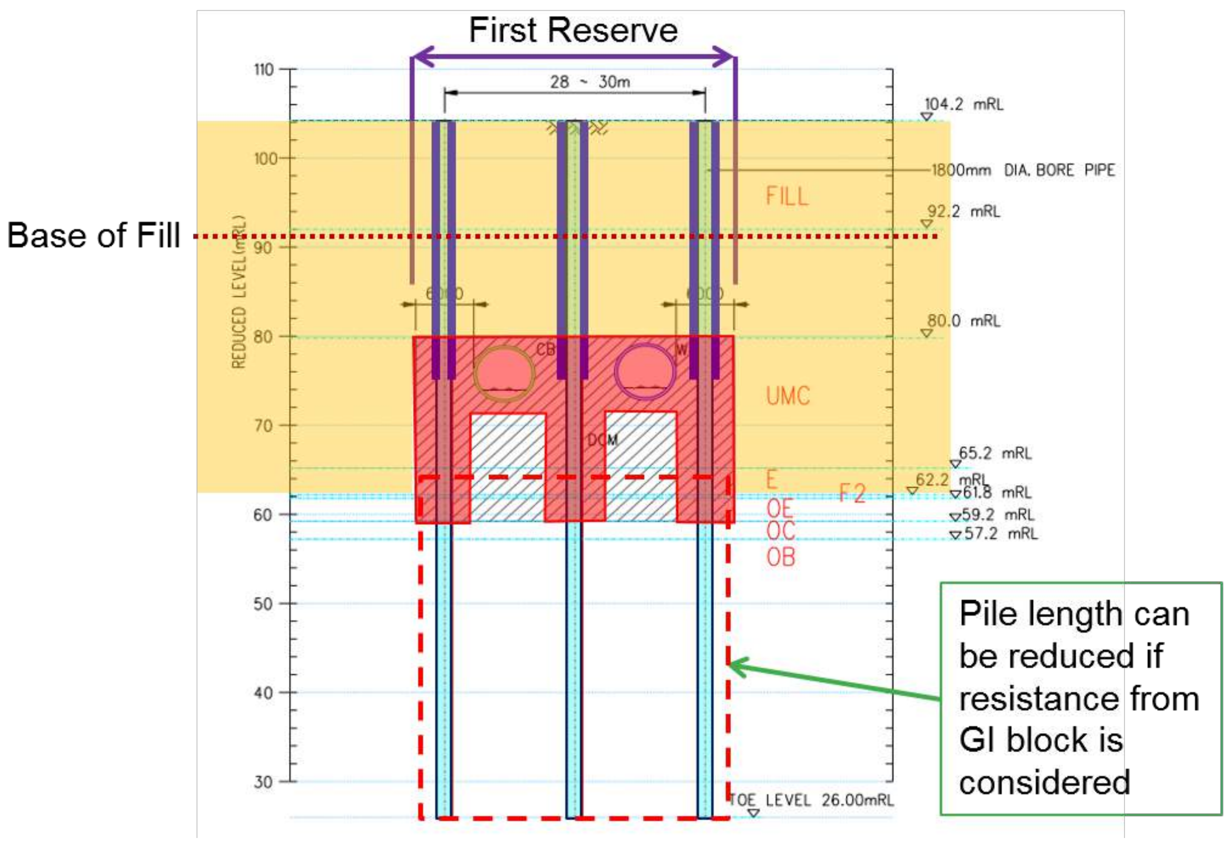


Fig. 28. Innovative design, modified DSM geometry and option to found piles in DSM.

Following value engineering studies MM proposed a combined solution for the Metro and OSD which optimized the geometry of the deep ground improvement into a “trouser leg” configuration and founded the OSD piles in the deep ground improvement block. This is the first solution of this kind in Singapore and therefore a preliminary trial and load test of the combined ground improvement and pile design was developed, Figure 29. The plan area of the ground improvement block is 6m by 10m and 45m deep. The pile is 1.8m dia and 24m deep, founded about 8m below the surface of the ground improvement. Instrumentation includes rod extensometers and strain gauges along the pile to monitor pile movement and load transfer along the pile shaft. Inclinometers and extensometers were installed in the ground improvement block to monitor its integrity and movement. The test pile was loaded up to 2.5 times working load.

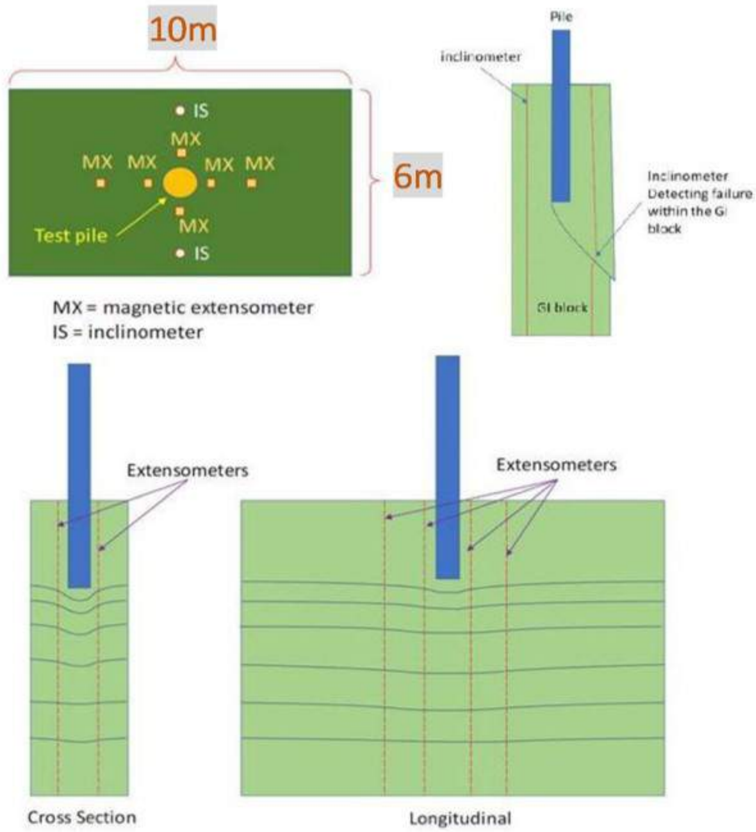


Fig. 29. Innovative design – DSM trial and pile load test.

The Specification for the ground improvement was to achieve an unconfined compressive strength (UCS) of 1.6 MN/m2 and a Young’s modulus of 280 MN/m2. Figure 30 shows the load-displacement of the test pile up to the maximum test load of 42MN. At a test load of about 25MN the mobilized shaft friction was of the order of 420 kPa, which increased to about 730 kPa at the maximum test load. Load transfer curves derived from the pile instrumentation, Figure 31, indicated a mobilized end-bearing resistance of 6MN at a test load of 25MN, increasing to about 9 MN at the maximum test load. The pile performance was better than originally expected primarily because the actual strength and stiffness of the DSM soilcrete was significantly higher than the specified target values. Figure 30 also shows the calculated pile load-settlement behavior based on the specified strength and stiffness parameters for the deep ground improvement (using a PLAXIS Mohr-Coulomb model to simulate the DSM behavior). Predicted settlement is about 3 times larger than observed settlement at 1.5 times working load. Testing of core samples from the DSM gave a median soilcrete UCS of about 4.2 MN/m2 and a coefficient of variation (COV) of 0.35, together with a mean Young's modulus of about 1.0 GN/m2. Although the DSM soilcrete strength and stiffness is high, the variability is also high and this variability is a common feature of DSM. Soilcrete also tends to be a brittle material (Kitazume and Terashi 2017) and because of the variability and brittleness it is necessary to ensure working loads are well below the ultimate capacity of the soilcrete. Updating the PLAXIS analysis with the measured strength and stiffness of the DSM soilcrete gave a good match with observed pile settlement up to a load of about 25MN (ie 1.5 times working load), which was considered adequate for preliminary design purposes. For loads in excess of about 34MN (ie 2 times working load) non-linear behavior becomes significant and a more sophisticated model would be needed to simulate the load-displacement behavior.

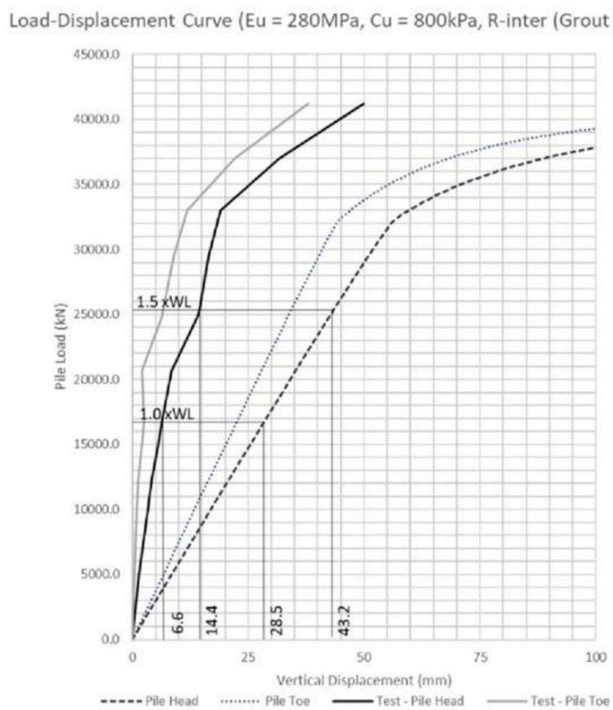


Fig. 30. Pile load test data versus calculated (using specification parameters).

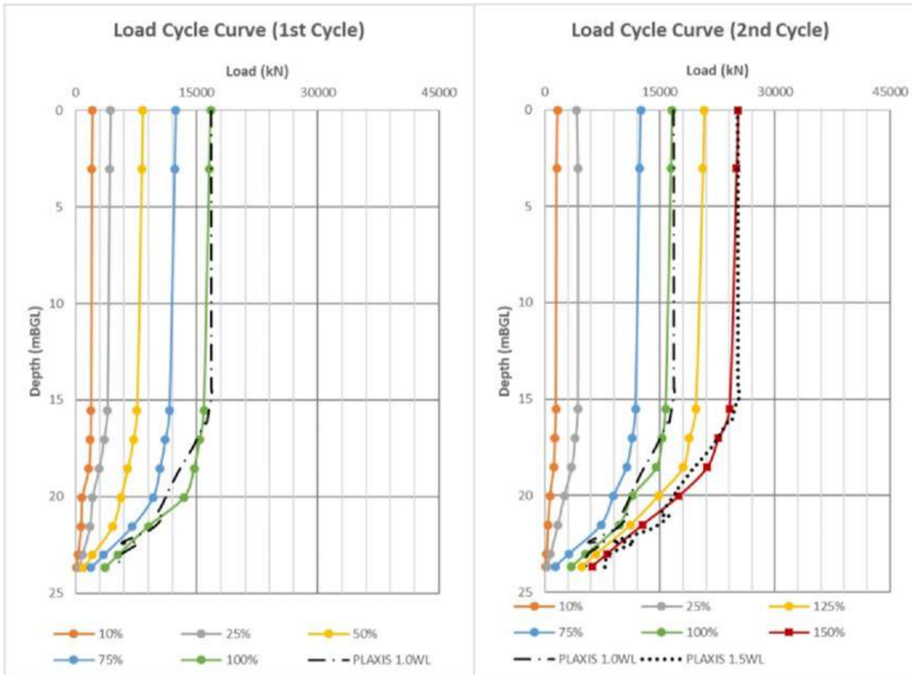


Fig. 31(a) load transfer, 16.8MN. Fig 31(b) load transfer 25MN.

Fig 31 Load transfer curves, test pile versus Plaxis 3D (using measured DSM strength/stiffness)

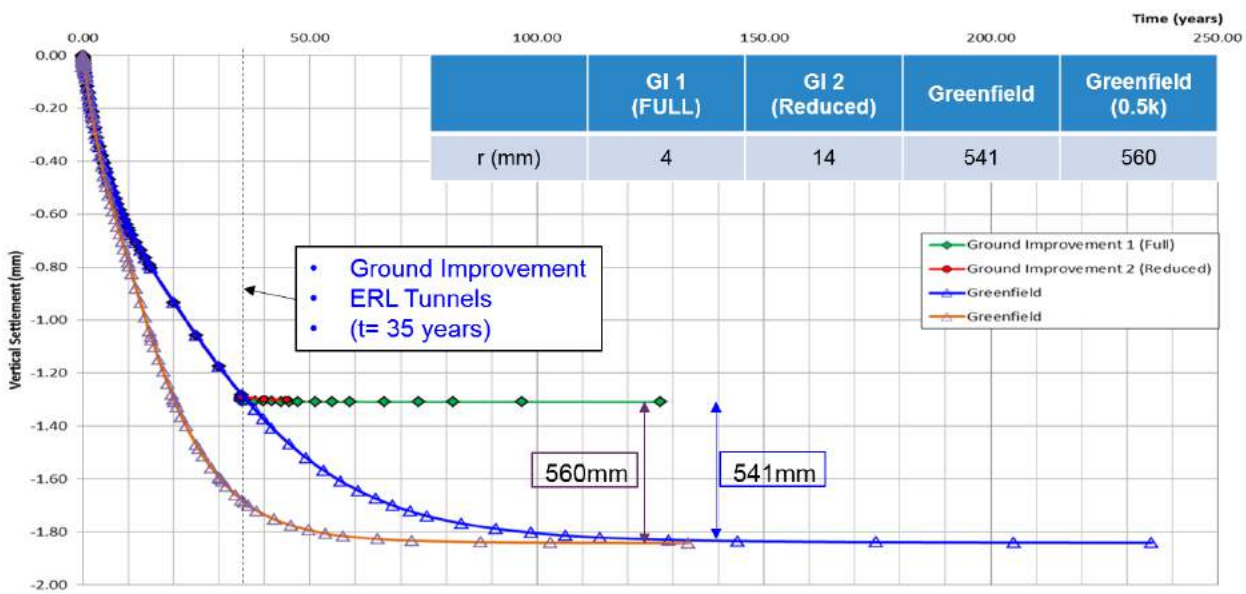


Fig 32. Settlement versus time, with and without ground improvement.

Figure 32 illustrates the settlement-time curve for one of the reclamation areas still undergoing consolidation, based on a suite of PLAXIS analyses (calibrated against historic field observations of reclamation settlement). The “greenfield” analyses assume no ground improvement and allow for the anticipated variation of in-situ permeability. Also shown are the two ground improvement options (the original “full” rectangular block, versus the “reduced” trouser-leg geometry). Both the ground improvement options practically eliminate ongoing settlement and there is little significant difference in settlement performance between the "full" and "reduced" schemes. The value engineered ground improvement scheme provided considerable cost, carbon and programme savings, with a 35% reduction in the volume of ground improvement (using the trouser-leg configuration) and a 65% reduction in piling (with the piles founded in the DSM).

8. CONCLUSIONS

Future urban developments will place increasing demands on the foundation engineering industry (both contractors and designers). In the congested underground urban space, larger and deeper foundations will need to be constructed close to existing, potentially fragile, existing structures. Society will expect the industry to deliver these projects on time and budget with minimal impact on existing infrastructure and the environment. There are considerable opportunities for innovation, with potential cost, carbon and construction schedule savings. However, delivering innovation needs: a supportive contractual and commercial environment; a coherent design approach with a single organization taking responsibility for foundation design (and potential design silos between design disciplines, eg structural and geotechnical engineers also need to be eliminated).

Three examples of foundation engineering have been described, key issues include:

(i) **Pile Groups** – if the pile cap/sub-structure has adequate strength and stiffness to re-distribute loads across a pile group, then the local geotechnical factor of safety of individual piles within a group is irrelevant. Pile group deformation at working load and under code factored conditions (serviceability and ultimate limit states for limit state codes) will often be the key concern. Deformation at working loads is controlled be the ground stiffness at very small strain, Go. Hence, designers should specify field geophysics testing to enable Go to be measured directly. For pile groups subject to large moment/lateral loads the use of full non-linear stress-strain models is appropriate. Hyperbolic stress-strain models are generally adequate and can be readily implemented for practical design purposes;

(ii) **Piled-rafts** – some simple definitions are helpful when considering the use of piled-rafts. A "raft-enhanced pile group" is a pile group with enhanced capacity from a raft, which allows the pile group to operate more efficiently (piles at spacing > 4d). A "pile-enhanced raft" uses a small number of piles (mobilizing their full geotechnical capacity) at very wide spacing (>8d) to reduce peaks of stress below a large raft and to reduce differential settlement. A "compensated piled-raft" is a piled-raft within a deep excavation below the water table. As outlined in this paper, many factors must be considered in the design of a compensated piled-raft. However, this type of piled-raft is becoming important for the design of large Metro schemes, since it allows "Over-Site Developments" (OSDs) to be built above the underground Metro box. Non-linear ground stiffness and consideration of the large changes in effective stress are important aspects of design. Control of differential settlement across the piled-raft and with connecting Metro tunnels is usually the critical design consideration. Hence, effective communication between structural and geotechnical engineers is important together with pragmatic choices about limits for differential settlement. Proper assessment usually needs well calibrated 3D non-linear numerical modelling, under the direction of experienced senior specialists;

(iii) **Combining ground improvement with piling** – for soft ground sites there are opportunities to combine specialist ground improvement (such as deep soil mixing or jet grouting) with piling to deal with complex geotechnical challenges. These can create very large capacity deep foundations in a cost-effective way, whilst minimizing environmental and other adverse impacts on adjacent areas. An example is given in this paper for a site in Singapore. The combined system was highly effective both in controlling ground movements and producing high capacity foundations for an OSD at a fraction of the cost of a conventional solution. Mobilized friction and end-bearing was higher than originally anticipated, since the deep soil mixing (at the project site) was highly effective in creating a high strength soilcrete. However, the high variability of soilcrete and its potentially brittle failure characteristics mean that working loads must be kept well below ultimate capacity.

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