Investigation into the effect of uncertainty of CPT-based soil type estimation on the accuracy of CPT-based pile bearing capacity analysis

O. Hamza
University of Derby, UK, o.hamza@derby.ac.uk

A. Bellis
Atkins Ltd, UK

ABSTRACT
Cone Penetration Test (CPT or CPTu) is commonly used for estimating soil types and also for the geotechnical design of pile foundation. However, the level of agreement between the CPT-based soil types and the traditional identification of soil types based on samples may vary significantly; and it is not clearly understood if this variation has any sort of relationship with the CPT-based pile design. To investigate into this area, a ground investigation trial was carried out at six different locations as part of a highway scheme in East of England. At each location the trial comprised one CPTu adjacent to one borehole (BH) with conventional sampling and laboratory testing. The soil types were estimated from the CPTs and compared with the boreholes findings, and the levels of correlation between them were established. Similarly, the ultimate bearing capacity of a typical bored pile based on the CPTs and on the BHs were calculated and compared. Despite the variable level of disagreement of the CPT-based soil type estimation with the BHs findings, the pile capacity based on CPT data was found to be generally consistent with the values obtained from the traditional BHs-based pile design.

Keywords: In situ testing, CPT, Piles, Design

1 INTRODUCTION

The cone penetration test (CPT or CPTu) has been extensively used for characterization of soils due to its specific advantages such as fast operation, relatively low cost, near-continuous profile, and stratigraphic detailing. In addition to the determination of soil stratigraphy and the identification of soil type, CPT-data can be used directly in the design of pile foundation with high reliability (Lunne et al. 1997, Robertson, P.K. & Cabal, 2012).

However, the currently available semi-empirical methods may present a significant variability in the estimation of soil type (Robertson, 2010, Robertson, P.K. & Cabal, 2012). In particular, for some mixed soils (i.e. sand-mixtures & silt-mixtures) where the CPT-based SBT (Soil Behaviour Type) may not always agree with traditional soil classification system (such as USCS and BS) which are based on samples and laboratory testing. Furthermore, it is not clearly known if the uncertainty in soil type estimation has any relationship with pile capacity based on direct application of CPT.

This paper investigates if there is any correlation between the uncertainty of CPT-based interpretation of soil type and the CPT-based pile design. This was conducted by accessing data from a CPT/Boreholes trial carried out at 6 different locations as part of a recent site investigation for a highway scheme in East of England (Atkins, 2009). The CPT-based pile design (the calculation of the ultimate vertical bearing capacity of a typical bored pile) was carried out and compared with the results of the design based on standard boreholes, where the soil parameters were conventionally obtained from Standard Penetration Test (SPT) results and laboratory testing. Figure 1 shows the conceptual model of the aim of the study presented in this paper.

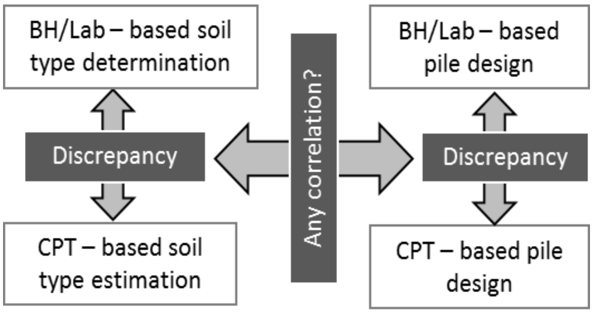

Figure 1 Conceptual model showing the aim of the study.

2 BACKGROUND ABOUT THE SITE INVESTIGATION

The geotechnical data used for this study was retrieved from a recent ground investigation (GI) which was conducted for a highway project "A14 Ellington to Fen Ditton Improvement Scheme - Section 1" (Atkins, 2009). This section bypasses the developed areas around Huntingdon (see Figure 2); it begins 1.0km west of the A1/A14 Brampton Hut interchange and runs approximately south alongside the A1.
The main GI included a total of 80 borings and 70 CPT soundings performed along the project route. In situ testing included the Cone Penetration Test (CPT) and the Standard Penetration Test (SPT). As part of the boring program, the retrieved samples were subjected to a full suite of laboratory index tests to determine water content, unit weight, specific gravity of solids, liquid limit, plastic limit, and particle size distribution by means of sieve and hydrometer.
During this ground investigation, a CPT trial was undertaken adjacent to the locations of 6 no. cable-percussion boreholes. This particular data was used within the scope of the paper.

3 BOREHOLES (BHS) AND GEOLOGY OF THE SITE

At the locations of the CPT trial, 6 no. cable-percussion boreholes have been excavated (BH2003, BH2005, BH 2015, BH 2017, BH2024 and BH2031). From the borehole findings the geology of the site comprises Glacial Till and Oxford Clay, overlain by Head deposits and River Terrace Gravels in the western quarter of site. The borehole logs of the exploratory holes are shown in Figure 3. Both the Glacial Till and Oxford Clay are heavily overconsolidated deposits. The extension of Glacial deposits revealed at BH 2005/2015 a buried channel. The depth of groundwater tends to lie at depths of between 1m and 3m bgl. More details can be found in the Ground Investigation Report (Atkins, 2009) and Factual Report (Lankelma, 2008).

Figure 2 Map showing approximate location of the site investigation

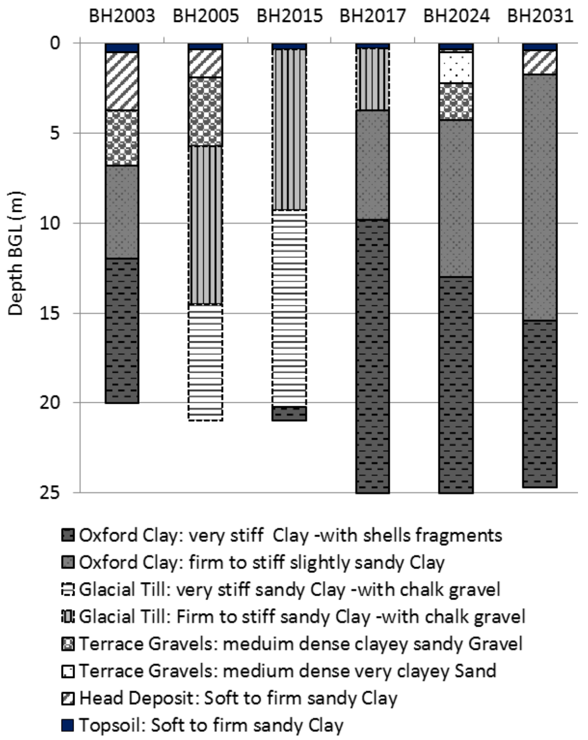

Figure 3 Borehole logs showing the geology of the site where the CPT trial has taken place.

4 SOIL TESTING AND PARAMETERS

In addition to the boreholes, in-situ tests including Standard Penetration Tests (SPT), and a selection of laboratory tests were carried out to confirm the soil type and provide data which would enable the geotechnical design. The laboratory testing included: MC, PSD, Atterberg Limits (PL, LL), Bulk density, Quick Undrained Triaxial (UU), Consolidated Undrained Triaxial (CU), and Hand Shear Vane (HSV). Table 1 summaries the soil properties and their typical values obtained from some of the tests, and considered in the study.

5 CPT TESTING AND INTERPRETATION

The investigation consisted of performing 6 electric Piezocone Penetration Tests (CPTU's) to a maximum depth of 23m or refusal. The Cone Penetration Tests were performed with a track mounted CPT unit equipped with a 20 Tonne Capacity Hydraulic ram set. A single electric piezocone conforming to the requirements of clause 3.1 of BS1377: 1990: Part 9 was used on this investigation. All tests measured the cone end resistance (qc), the local side friction (fs) and porewater pressure (u2). The test results are presented in Figure 4.
Many studies have been performed on the interpretation of the estimated soil type from the CPT test data (Robertson et. al., 1986; Meigh, 1987; Zhang & Tumay, 1999).
One of the more common CPT-based methods to estimate soil type is the chart suggested by Robertson et al (1986) based on cone resistance, qc and friction ratio, Rf. Although newer charts have been developed based on normalized parameters, the simple chart based on qc and Rf is still popular because of its simplicity (Robertson, 2010; Long, 2008). Therefore in this study the original Robertson et al (1986) chart was used for the interpretation of the CPT data (i.e. to evaluate soil type) as presented in the next section.

6 COMPARISON BETWEEN THE FINDINGS OF THE BOREHOLES (BHS) AND CPTS

The soil type information obtained from the CPT tests (based on the method proposed by Robertson et al 1986) was compared with the soils encountered within the adjacent exploratory holes, which were described according to British Classification System (BS5930).
In order to determine how far the information obtained from the exploratory holes (BHs) and the CPT reveals similar ground conditions, a system of three different categories (i.e. Good, Acceptable and Poor)

Table 1 Summary of engineering properties of soil layers found in the site investigation trial.

Parameter (Unit)		**Range (Typical value)**			
		HD	**RT**	**GT**	**OC**
Natural Moisture Content (%)		10-46 (23)	5-25	15-25 (17)	**15 – 45(24)**
Bulk Density (Mg/m^3)		1.7 - 2.1	2.0 ‡	2.1-2.2	**1.9 -2.2**
Liquid Limit (%)		23-87 (45)	20-70 Ґ	40-55 (45)	**40 – 75(57)**
Plastic Limit (%)		12-35 (18)	12-23 Ґ	12-22 (18)	**16 – 30(22)**
Plasticity index (%)		10–41(25)	5-46 Ґ	4-42 (30)	**25 – 55(35)**
Particle Size Distribution (PSD)	Clay (%)	5-56 (27)	(16)	37	**51**
	Silt (%)	10-30 (25)	(8)	38	**43**
	Sand (%)	5-40 (28)	(37)	14	**3**
	Gravel (%)	0-50 (10)	(35)	9	**3**
	Cobble (%)	0-1 (1)	(4)	1	**1**
SPT 'N'		2 - 30	10 - 35	(13+2.0Z)	**(5+1.9Z)**
Shear Strength	c' (kN/m^2)	0-5 ₸	-	0-5 ₸	**0-5 ₸**
	ϕ' (degree)	26-33 ₸	30–37 ¥	25-26.5 ₸	**23.6-26 ₸**
	C_u (kN/m^2)	5-129	-	(60+12Z) ¥₸≠	**45+10Z ¥₸≠**
Notation: ¥ : based on empirical correlation with SPT (Peck, 1974; Tomlinson, 2001); ‡ : Typical value found in literature; ₸ : based on Traixial Test; Z: depth from top of layer; ≠ : based on HSV test; Ґ : carried out on the cohesive content of the materials.					

was proposed by the Authors. The three levels of agreement between CPTs and BHs are explained as follow:

- Good: when the description of the soil generally agree with each other
- Acceptable: when the major class of soil

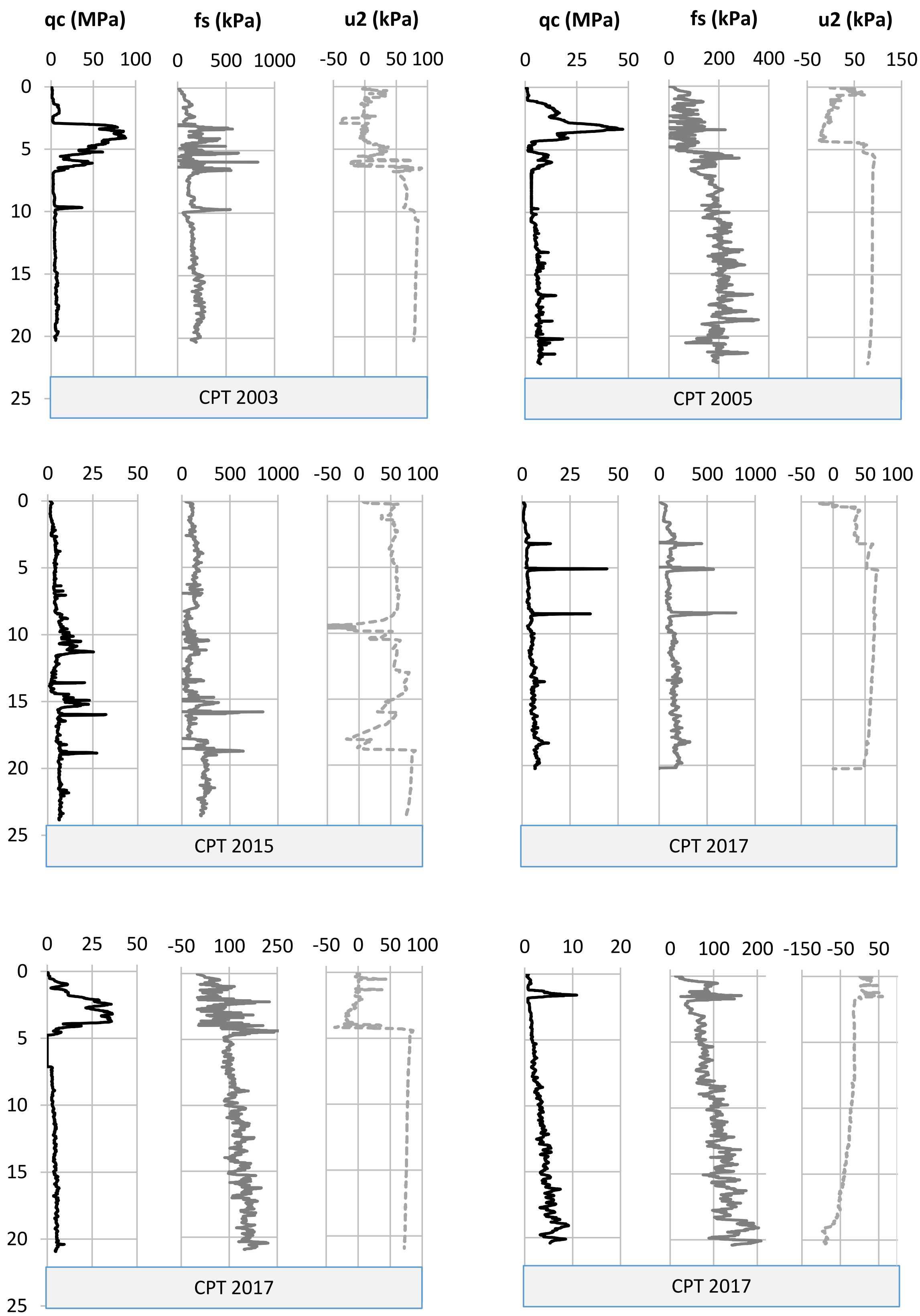

Figure 4 The measurements of CPTUs: Tip Resistance qc, Sleeve Resistance fs, and Pore Pressure u2.

agree (Coarse-grained and Fine-grained soil), but the sub-class does not agree (Clay against Silt, and Sand against Gravel)

- Poor: when the major class of soil totally disagree

The tables from 2 to 5 represents the findings from both Boreholes and CPT for each type of soil found in the site investigation trial (excluding the top soil); the correlation level was also added to the tables.

7 FINDINGS OBTIANED FROM THE COMPARISION OF BHS AND CPTS

7.1 Head Deposits

This soil type was encountered in four boreholes at shallow depth of about 0.3-0.5mbgl (see Table 2). From the four cases encountered, only 52% of the total length (see Figure 5) coincided (i.e. Good correlation level) with the description of the exploratory holes but with a slight difference in soil conditions and strength. The rest (i.e. 48%) described totally different major type of soils that the conditions described on the CPT.

7.2 Terrace Gravel (TG)

The Terrace Gravel was encountered below the Head Deposits between a minimum depth of 0.5mbgl in BH2024 to a maximum of depth of 6.80m bgl in BH2003.
The CPT-based soil descriptions appear to be similar to the soils encountered into the boreholes with slight differences in density; however two of the descriptions (Table 3) differ completely in soil class and strength. The general correlation levels for this soil were: 16% Good, 77% Acceptable, and 7% Poor as shown in Figure 5.

7.3 TG Glacial Till (GT)

Glacial Till was encountered in three boreholes between a minimum depth of 0.25mbgl in BH2017 to a maximum of depth of 20.25m bgl in BH2015 (see Table 4).
A total of 59% of the descriptions from CPT test appears to be correct, 16% was acceptable, and 26% was poor.

7.4 Oxford Clay

The Oxford Clay is encountered between a minimum depth of 2.0mbgl in BH2031 to a maximum of depth of 25.0m bgl in BH2003. There was a total run of 82m of this type soil in all boreholes; only 33% of the description was correct, 55% was acceptable but with some differences in soil material and strength, and 46% of the description was totally different (i.e. Poor).

7.5 General comment

As shown in Figure 5, the overall level of correlation for the four types of soil discussed above scored 42% as “Good”, 40% “Acceptable”, and 18% “Poor”. The lowest score (i.e. Poor correlation) was occurred in Head Deposit followed next by the Oxford Clay.
Head Deposit was described as “firm sandy clay” which falls within mixed soils region (i.e. sand mixtures) where CPT has been reported to have some difficulty predicting the soil type (Roberson, 2010).
For the case of Oxford Clay, the majority of the description of this soil was given as stiff to very stiff (slightly) sandy clay - with some gravel. The difficulty of predicting soil type by CPT may be explained in this particular case on the basis that very stiff, heavily overconsolidated fine-grained soils are more inclined to behave like a coarse-grained soil in that they tend to dilate under shear and can have high undrained shear strength compared to their drained strength and can have a CPT-based soil behavior type in a coarse grained zones (Roberson, 2010).
However, it is commonly accepted (Roberson, 2010) that the geotechnical design is more connected with in-situ soil behaviour than a classification based on grain-size distribution and plasticity carried out on disturbed samples, although knowledge of both is helpful. Therefore, the second part of this study tried to find out if the uncertainty encountered in CPT-based soil type estimation has any relationship with the CPT-based geotechnical design, taking pile foundation as an example.

Table 2 Head Deposits (HD)

CPT / BH no.	BH log		CPT log		Correlation level
	Depth bgl(m)	Soil description	Depth bgl(m)	Soil description	
2003	0.5 – 1.8	Firm slightly sandy clay (HD)	0.5 -0.9	Firm clay	**Good**
			0.9-1.2	Stiff clay	**Good**
			1.2- 1.5	Very stiff clay	**Good**
			1.5-1.8	Medium dense sand	**Poor**
2003	1.8 – 3.70	Firm sandy clay (HD)	1.8 – 2.5	Medium dense sand	**Poor**
			2.5 – 3.0	Stiff clayey silt to silty clay	**Good**
			3.0 – 3.7	Very dense gravelly sand	**Poor**
2005	0.35 – 1.90	Firm sandy clay (HD)	0.35 – 1.1	Firm silty clay	**Good**
			1.1 – 1.6	Medium dense silty sand	**Poor**
			1.6 – 1.9	Dense sand	**Poor**
2024	0.3 – 0.50	Firm sandy clay (HD)	0.3 – 0.5	Firm clay	**Good**
2031	0.40 – 1.20	Soft sandy clay (HD)	0.40 – 1.20	Soft to firm clay	**Good**
2031	**1.20 – 1.70**	**Firm slightly sandy clay (HD)**	**1.20 – 1.70**	**Medium dense sand**	**Poor**

Table 3 Terrace Gravel (TG)

CPT / BH no.	BH log		CPT log		Correlation level
	Depth bgl(m)	Soil description	Depth bgl(m)	Soil description	
2003	3.70 – 6.80	Medium dense clayey very sandy gravel (TG)	3.7 – 6.4	Very dense gravelly sand	**Acceptable**
			6.4 – 6.80	Dense silty sand	**Acceptable**
2005	1.9 – 5.7	Medium dense clayey sandy gravel (TG)	1.9 – 4.6	Dense to very dense sand	**Acceptable**
			4.6 – 5.1	Stiff clayey silt to silty clay	**Poor**
			5.1 – 5.7	Medium dense silty sand	**Acceptable**
2024	0.50 – 2.20	Medium very clayey sand (TG)	0.5 – 2.1	Medium dense sand	**Good**
			2.1 – 2.2	Very dense sand	**Good**
2024	2.20 - 4.25	Medium very sandy gravel	2.20 – 4.25	Very dense sand	**Acceptable**
2031	**1.70 – 2.00**	**Loose slightly sandy gravel**	**1.70 – 2.00**	**Soft clay**	**Poor**

Table 4 Glacial Till (GT)

CPT / BH no.	BH log		CPT log		Correlation level
	Depth bgl(m)	Soil description	Depth bgl(m)	Soil description	
2005	5.7 – 13.30	Stiff sandy clay (GT)	5.7 – 6.5	dense to medium dense sand	**Poor**
			6.5 – 7.6	Stiff clay	**Good**
			7.6 – 13.30	Very stiff clay	**Good**
2005	13.30 – 22.0	Stiff clay - becoming very stiff	13.30 – 13.8	Medium dense sandy silt	**Acceptable**

		from 14.50m depth. (GT)	13.8- 16.4	Very stiff clayey silt to silty clay	**Good**
			16.4 – 22.18	Medium dense sandy silt to clayey silt	**Acceptable**
2015	0.3 – 1.1	Firm slightly sandy clay with chalk gravel (GT)	0.3 – 1.1	Firm clay	**Good**
2015	1.1 – 9.25	Stiff slightly sandy clay -with chalk gravel (GT)	1.1 – 2.6	Stiff clay	**Good**
			2.6 – 6.9	Very stiff clayey silt to silty clay	**Acceptable**
			6.9 – 9.25	Medium dense sand to silty sand	**Poor**
2015	9.25 – 20.25	Very stiff slightly sandy clay/silt - with chalk gravel (GT)	9.25 – 12.5	Medium dense sand to silty sand	**Poor**
			12.5-15.0	Loose silty sand to sandy silt	**Good**
			15.0 – 18.8	Medium dense silty sand	**Poor**
			18.8 – 20.25	Very stiff clayey silt to silty clay	**Good**
2017	0.25 – 1.65	Firm slightly sandy clay -with chalk gravel (GT)	0.25 – 0.9	Firm clay	**Good**
			0.9 – 1.65	Soft clay	**Good**
2017	1.65 – 3.70	Stiff slightly sandy clay -with chalk gravel (GT)	1.65 – 3.1	Stiff clay	**Good**
			3.1 – 3.70	**Very stiff clay**	**Good**

Table 5 Oxford Clay

CPT / BH no.	**BH log**		**CPT log**		**Correlation level**
	Depth bgl(m)	**Soil description**	**Depth bgl(m)**	**Soil description**	
2003	6.80 – 25.0	Stiff slightly sandy clay - with little gravel become very stiff from 12m depth. (Oxford Clay)	6.80 – 7.2	Stiff clay	**Good**
			7.2 – 8.6	Stiff silty clay	**Good**
			8.6 – 9.6	Very stiff clayey silt	**Acceptable**
			9.6 – 10.3	Medium dense to dense silty sand	**Poor**
			10.3 – 11.1	Medium dense sandy silt	**Acceptable**
			11.1 - 15	Very stiff clayey silt to silty clay	**Good**
			15 – 16.5	Medium dense sandy clay	**Good**
			16.5 – 20.39	Alternate of very stiff clayey silt and medium dense sandy silt	**Acceptable**
2015	20.25 – 21.0	Very stiff clay (Oxford Clay)	20.25 -21.6	Very stiff clayey silt to silty clay	**Good**
			21.6 – 23.93	Medium dense sandy silt	**Acceptable**
2017	3.70 – 9.80	Stiff Slightly sandy clay (Oxford Clay)	3.70 – 6.1	Stiff clay to silty clay	**Good**
			6.1 – 7.6	Very stiff clayey silt	**Acceptable**
			7.6 – 9.80	Loose to medium dense silt	**Acceptable**

2017	9.80 - 25	Very stiff slightly sandy clay	9.8 – 20.26	Medium dense sandy silt to silty sand	**Poor**
2024	4.25 – 13.0	Stiff clay -with shells fragments (Oxford Clay)	4.25 -8.6	Stiff clay	**Good**
			8.6 – 10.5	Very stiff silt	**Acceptable**
			10.5 – 12.0	Loose to medium dense sandy silt	**Acceptable**
			12.0 – 13.0	Very stiff clayey silt to silty clay	**Good**
2024	13.0 – 25.0	Very stiff clay -with shells fragments	13.0 – 21.0	Medium dense sandy silt to clayey silt	**Acceptable**
2031	2.0 – 7.70	Firm clay -with shells and gypsum.	2.0- 7.70	Firm clay becoming stiff at 4.1	**Good**
2031	7.70 – 15.70	Stiff clay -with shells and gypsum. (Oxford Clay)	7.70 – 12.5	Stiff to Very stiff clayey silt to silty clay	**Good**
			12.50 – 15.70	Loose to medium dense sandy silt	**Acceptable**
2031	**15.70 - 25**	**Very stiff clay -with shells and gypsum**	**15.70 – 20.49**	**Medium dense silty sand.**	**Poor**

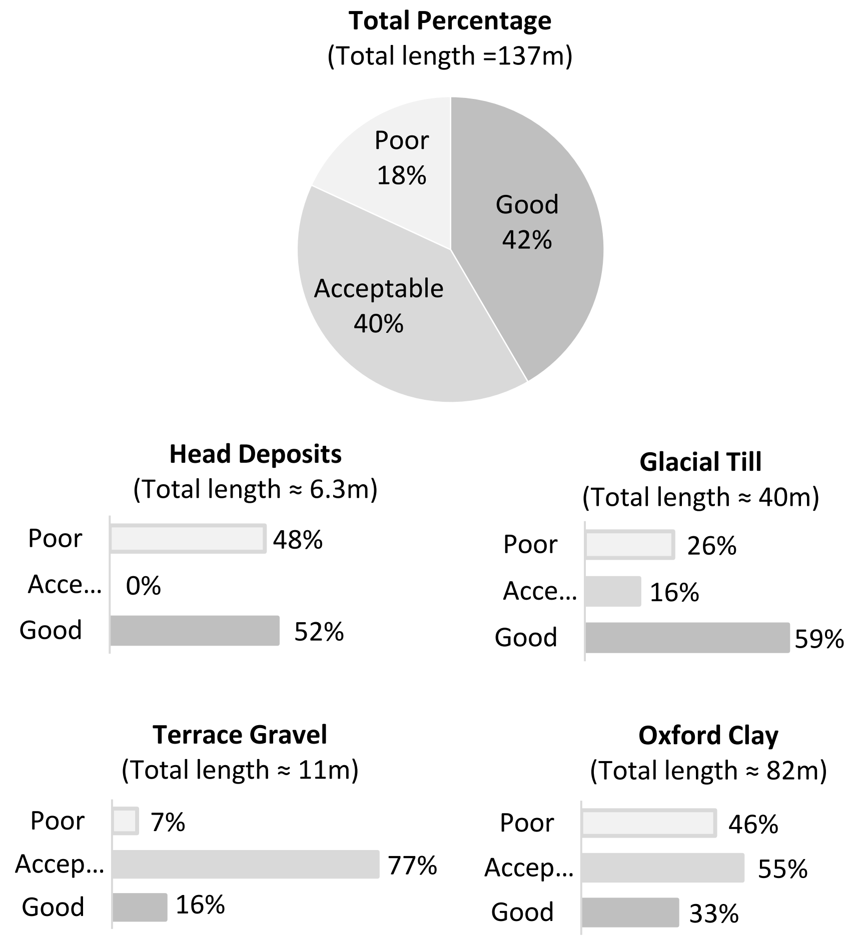

Figure 5 Percentage of the level of agreement between BH-based and CPT-based soil types.

8 PILE DESIGN ANALYSIS –METHOD

To investigate the effect of uncertainty of CPT-based soil type estimation on the geotechnical design of pile foundation (particularly the ultimate bearing capacity of a single axially loaded pile), a typical bored pile is used to evaluate this effect.
CPT results are widely used by geotechnical engineers to predict the ultimate load carrying capacity (Briaud 1988; Eslami & Fellenius, 1997; Price & Wardle, 1982; Titi & Abu-Farsakh, 1999, Tumay & Fakhroo, 1982.). Bored piles are the most common type of non-displacement piles and many design methods have been well established in literature (Poulos & Davis, 1980). The ultimate pile load carrying capacity was calculated in this study applying two analytical methods:

- The first design method was Meyerhof's method (1976) which is based on soil properties conventionally acquired from the BHs and its associated laboratory and in-situ tests.
- The second method of estimating the ultimate pile carrying capacity was based on Bustamante method (LCPC) (Bustamante & Gianeselli, 1982) which is a direct approach that utilizes data on the cone penetration test (CPT).

Both methods represent static analytical approach where the load carrying capacity of the pile consists of two components: shaft tangential resistance Q_s and base compressive resistance Q_b. Thus, the ultimate load-carrying capacity of a pile is given by equation 1.

$$Q_u = Q_b + Q_s \tag{1}$$

The ultimate pile load-carrying capacity obtained by both methods will allow the comparison between BH- based design and CPT-based design, and ultimately any discrepancy between them will be compared with the discrepancy of soil type estimation obtained from CPT and BH (see Figure 1). The assumption used in both pile design methods are summarised in Section 8.1 and 8.2. In both methods the pile was assumed of a plain bored type with circular cross section of a diameter of 0.5m (uniform along the pile length) and a total length of approximately 23m i.e. corresponding to CPTs and BHs depths.

8.1 BHs-based pile design

Using the shear strength parameters obtained from the testing associated with the boreholes (BHs) it is possible to determine the ultimate bearing capacity of a pile using Meyerhof's method (1976) (explained in Braja, 2010):
For cohesive soil:

$$Q_b = 9\, C_u\, A \tag{2}$$

$$Q_s = p\, \Sigma\Delta L\;\, \alpha\, C_u \tag{3}$$

For granular soil:

$$Q_b = \min\{A\, q'\, Nq;\, 0.5P_a\, N_q \tan \phi'\} \tag{4}$$

$$Q_s = p\, \Sigma\Delta L\, K\, \sigma'_0 \tan \delta' \tag{5}$$

where C_u: undrained cohesion; ϕ': effective soil friction angle of the bearing stratum; p: perimeter of the pile; L: pile length; A: cross section area of the pile; δ': soil-pile friction angle=2/3ϕ'; K: effective earth pressure coefficient =1- sin ϕ'; σ'_0 : effective vertical stress at the depth under consideration; Nq: bearing capacity factor; its variation with soil friction angle is estimated according to Meyerhof (1976); q': effective vertical stress at the level of the pile tip; P_a: atmospheric pressure =100kN/m2; α: empirical adhesion factor estimated according to Terzaghi, Peck and Mesri (1996).
The multilayer effect on end bearing was ignored.

8.2 CPT-based pile design

According on LCPC method (Bustamante & Gianeselli, 1982), pile ultimate bearing capacity is estimated as:

$$Q_b = A\, k_c\, q_{eq} \tag{6}$$

where k_c= is an empirical end bearing factor that varies from 0.15– 0.60 depending on soil type and installation procedure.

q_{eq} = equivalent average of qc values of zone ranging from 1.5D below pile tip to 1.5D above pile tip, where D is pile diameter.

$$Q_s = p\ \Sigma \Delta L\ f_p \qquad (7)$$

where f_p : unit side friction = $q_c / \alpha_S \leq max$
α_S= friction coefficient =30– 200 depending on soil type, pile type and installation procedures.

9 PILE DESIGN ANALYSIS – RESULTS & DISCUSSION

The pile design analysis was carried out using the soil profiles of the 6 boreholes and the adjacent 6 CPTs. To allow comparison between the BH-based & CPT-based design, the results are arranged in couples as presented in Figure 6. Each couple represents the variation of the ultimate bearing capacity Q_u with depth at a single location.
As shown in Figure 6, the Q_u curves exhibited similar trends and general increases with depth. However there were three exceptions (in BH/CPT2003, 2005, and 2024) of sharp increases over relatively short runs, which were found to be coincided with the existence of Terrace Gravel (TG). Granular materials (such as Terrace Gravel) tend to have a base resistance Q_b several times larger than the shaft frictional resistance Q_s.
The maximum agreement between the two sets of results (i.e. BH & CPT based analyses) were found at three locations (BH/CPT 2015, 2017, and 2031), which are referred as “Group A” in Figure 6. However, at the other three locations (BH/CPT2003, 2005, and 2024) the CPT-based pile capacity is remarkably larger than the values obtained from the BH-based analysis. These three locations are referred as “Group B” in Figure 6. It is apparent from this figure the higher divergence in Group B is more pronounced around the location of Terrace Gravel (TG) as the divergence became narrower with depth.
Figure 6 shows also that for Group A, the poor correlation between BHs-based and CPTs-based soil types varied from 0 to 45%. Despite this variation in predicting soil type, the bearing capacity results was consistent. On the contrary in Group B, where the “poor” prediction of soil type varied from 0 to 12% only, the bearing capacity results showed more discrepancy between BH-based analysis and CPT-based analysis. However the discrepancy has a consistent trend and therefore it is very likely to be caused by the different assumptions used in each method

10 CONCLUSIONS

A site investigation trial consisting of 6 boreholes (BHs) and 6 Cone Penetration tests (CPTUs) was conducted in this study to investigate if there is any relationship between the uncertainty of CPT-based soil type estimation and CPT-based pile ultimate bearing capacity estimation.
This study has shown that the soil types established from the boreholes (based on soil samples and classification tests) and from the CPT (based on the original Robertson-1986 chart) appeared to be more similar for the shallow granular geology strata: Terrace Gravel. On the contrary, the CPT-based soil descriptions along the cohesive soils (Head Deposit, Glacial Till and Oxford Clay) divided the geological units in small layers that included great variations in composition (cohesive and granular) and strength/density, this may be due to the nature of soil mixture and the present of some cobbles or shells into the ground. However, the uncertainty in soil type estimation using CPT may be reduced using other chart/ method, which will be part of future work.
This study has also found that the variation in soil type estimation did not show any particular relationship with the pile geotechnical design (ultimate bearing capacity). This finding conforms to previous findings and contributes additional evidence suggesting that the cone responds to the in-situ mechanical behavior of the soil and not directly to soil classification criteria based on grain-size distribution and soil plasticity (Roberson et al, 2012).
The findings of the paper also conform to the perceived applicability of the CPTu for pile design and bearing capacity as highly reliable

(Bond A. and Harris, 2008). However, the reliability rating decreases for settlement and other geotechnical design. Therefore, investigating the relationship between the uncertainty of CPT-based soil type determination and CPT-based pile settlement will be part of future work.

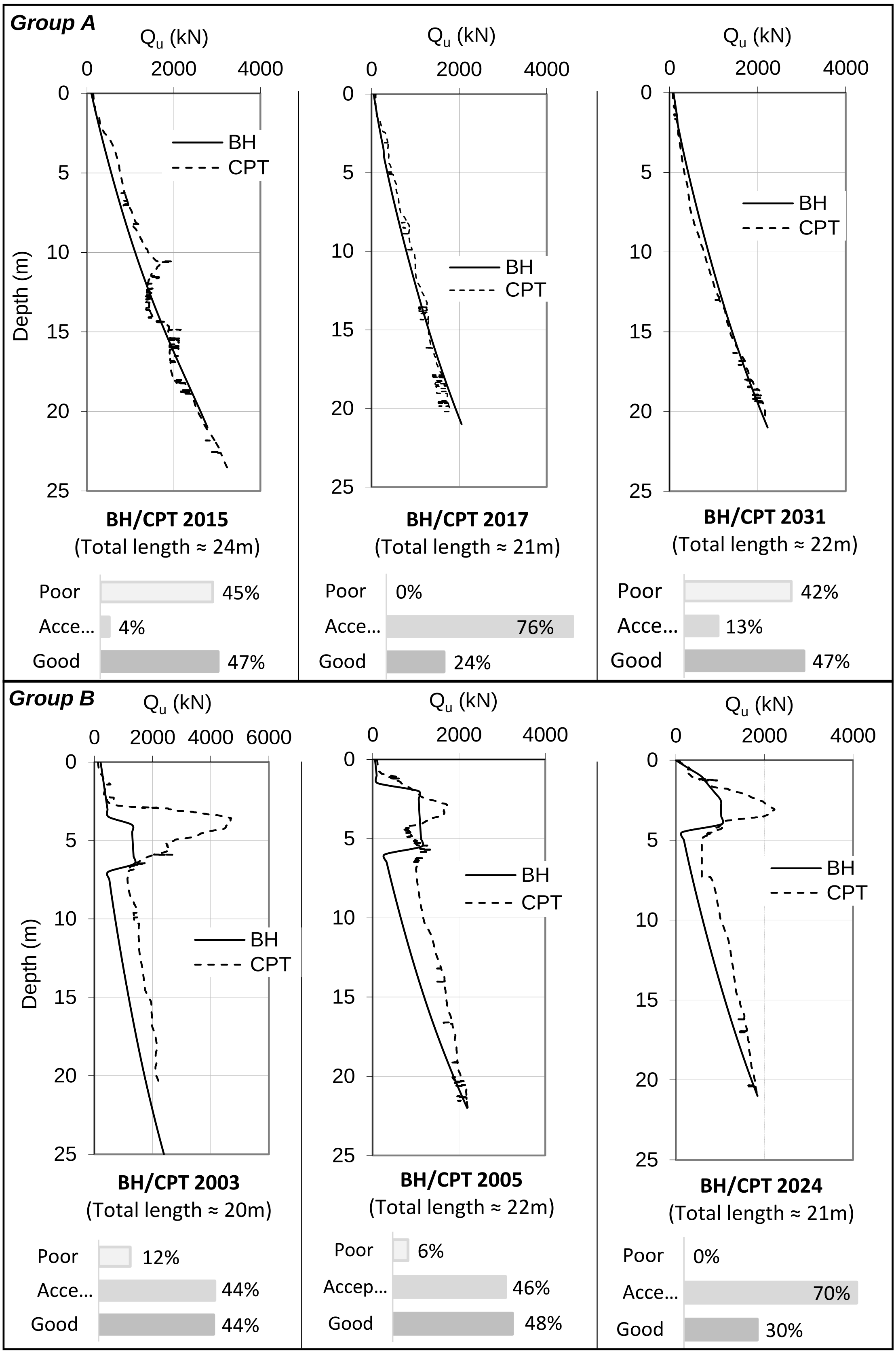

Figure 6 Variation of the predicted ultimate pile capacity (Q_u) with depth at the six locations. The level of agreement of soil type estimation is also added for comparison purposes.

11 ACKNOWLEDGEMENT

The authors gratefully acknowledge the support of the Highways England.

12 REFERENCES

Atkins Ltd, (2009). A14 Ellington to Fen Ditton-Phase 1a: Ground Investigation Report: Section 1. Document reference: 5062774-TE-DO-GEO-606.

Bond, A. & Harris, A. (2008). Decoding Eurocode 7, London: Taylor & Francis, 616pp.

Braja, M.D., (2010). Principle of Foundation Engineering, 7th edn. USA: Cengage Learning.

Briaud, J.L., & Tucker, L. M. (1988). Measured and predicted axial response of 98 piles. J. Geotech. Eng., 114(9), 984–1001.

BS 1377, (1990). Methods of Test for Soils for Civil Engineering Purposes. Part 9: In-situ tests. BSI, London, UK

BS 5930, (1999). Code of practice for site investigations. BSI, London, UK.

Bustamante, M. & Gianeeselli, L. (1982). Pile bearing capacity predictions by means of static penetrometer CPT. Proceedings of the Second European Symposium on Penetration Testing ESOPT-2, Amsterdam, May 24fi27, AA. Balkema, Vol. l.

Eslami, A., & Fellenius, B. H. (1997). Pile capacity by direct CPT and CPTU methods applied to 102 case histories. Can. Geotech. J., 34, 886–904.

Lankelma Ltd (2008). Static Cone Penetration Test. Factual Report. Report Ref Report No: 85071R001EL.

Long, M. (2008). Design parameters from in situ tests in soft ground – recent developments.

Lunne, T., Robertson, P.K. & Powell, J.J.M. (1997). Cone penetration testing in geotechnical practice. UK: Blackie Academic & Professional 1997: 16424. Mayne, PW 2001.

Meigh, A.C., (1987). Cone Penetration Testing - Methods and Interpretation. CIRIA. Butterworths.

Meyerhof, G.G. (1976). Bearing capacity and settlement of pile foundations. ASCE Journal of Geotechnical Engineering. 102(GT3)197-228.

Peck, R. B., Hanson, W. E. & Thornburn, T. H. (1974). Foundations Engineering, 2nd edn. New York, NY: John Wiley and Sons.

Poulos, H. G. & Davis, E. H. (1980). Pile Foundation Analysis and Design. New York: John Wiley.

Price, G., & Wardle, I. F. (1982). A comparison between cone penetration test results and the performance of small diameter instrumented piles in stiff clay. Proceedings of the 2nd European Symposium on Penetration Testing, Amsterdam, The Netherlands, Vol. 2, 775–780.

Proceedings of Geotechnical and Geophysical Site Characterization. Taylor & Francis Group, 89-116.

Robertson, P.K. & Cabal, K. L. (2012). Guide to Cone Penetration Testing for Geotechnical Engineering

Robertson, P.K. (2010). Soil behaviour type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT'10, Huntington Beach, CA, USA.

Terzaghi, K., Peck, R. B., & Mesri, G. (1996). Soil Mechanics in Engineering Practice, 3rd Edition, Wiley.

Titi, H. H., & Abu-Farsakh, M. Y. (1999). Evaluation of bearing capacity of piles from cone penetration test data. Rep. No. FHWA/ LA.99/334, Louisiana Transportation Research Center, Baton Rouge, La.

Tomlinson, M. J. (2001). Foundation Design and Construction, 7th edn, 569 pages, Upper Saddle River, NJ : Prentice Hall.

Tumay, M. T., & Fakhroo, M. (1982). Friction pile capacity prediction in cohesive soils using electric quasi-static penetration tests. Interim Research Rep. No. 1, Louisiana Department of Transportation and Development, Research and Development Section, Baton Rouge, La.

Zhang, Z., & Tumay, M. T. (1999). Statistical to fuzzy approach toward CPT soil classification. J. Geotech. Geoenviron. Eng., 125(3), 179–186.