



IMPERIAL COLLEGE LONDON

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**Assessing the applicability of CPT and
Piezocone tests for identifying the presence of
fault damage zones on geotechnical
engineering projects**

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2017-2018

DECLARATION OF OWN WORK

Declaration:

This submission is my own work. Any quotation from, or description of, the work of others is acknowledged herein by reference to the sources, whether published or unpublished.

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ABSTRACT

Several researchers have pointed out how tectonism reduces ground strength, and it has been recognised that faulting induces mechanical weakening of the rock mass locally. However, the extent and exact location of the damage zone around the shear plane remains uncertain. It is evident that there is a need for cost-effective tools to identify the presence and extent of damage zones on sites, minimizing the impact of unfavorable ground conditions faced.

In light of the current uncertainties, this research analyzed in detail the use of Cone Penetration Test (CPT) and piezocones as tools to determine the presence and impact of faulting, unveiling these hazardous ground conditions. A workflow on how to determine shear zones was developed. Two materials were investigated at locations known to have faulting either on site, or nearby: London Clay at St. James' Square, and the White Chalk in the Heart of Essex project. Through detailed analysis the main findings indicate that the impact of faulting on strength is traceable and identifiable from CPT and piezocone tests.

The most remarkable finding is the repeatability of cone tip resistance profiles with distance showed by CPT and piezocone tests. The latter provides an insight on how strength varies spatially, and even if no faults are found on site, a potential direction of weakening towards existing, nearby damaged zones could be identified.

ACKNOWLEDGMENT

The completion of this research is a huge milestone for me, and it does not belong to me, but to my parents, Grimaldo and Sofia, to whom I will be always in a great debt, as their effort and support guided me through life.

I would like to express my sincere gratitude to my advisor Dr Richard Ghail for the continuous support to my research, for his patience, motivation and knowledge. His guidance encouraged me to overcome every difficulty.

Last but not the least, I thank my fellow PhD, friend and advisor, Tom Morgan, for his precise interventions, which helped me to improve and direct better some key aspects of this research.

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1 INTRODUCTION

It is known that faulting generates brittle damage zones around its shear plane, weakening the rock mass locally. It is therefore detrimental to geotechnical engineering. For instance, faulting has been recognized in London. However, its mechanical impact and exact location are unknown.

The need of tools to assess the extent or presence of fault damage zones in geotechnical materials is evident. Some cost-effective tools are CPT and piezocone tests. However, the usefulness of these in situ tests to assess damaged zones remains unclear. In light of the current uncertainties, the present research aims to determine whether or not the CPT or Piezocone tests are able to identify the influence of faulting in ground conditions.

1.1 Introduction

The main aim of this thesis is to investigate whether the mechanical impact of nearby faulting can be determined from CPT and piezocone data. The ground conditions were assessed at two localities: (1) the London Clay formation at St. James' Square, and (2) Chalk formation at the Heart of East Greenwich (former Greenwich District Hospital).

Regarding specific objectives, this thesis has 3 objectives, and all of them are based on existing information corresponding to CPT and piezocone data, and geology.

The objectives are described as follows:

Objective 1. Carry out a detailed analysis of CPT and piezocone data, and determine if these tests can identify faulting.

Objective 2. Assess how different formations are affected by faulting, and if it is possible to observe their behavioral response in CPT or piezocone profiles.

Objective 3. Evaluate what other processes affect CPT and piezocone values, such as: geological processes, natural spatial variability, ground water conditions, fissures, fines content, and so on. This will prevent a false interpretation of faulting.

1.2 Thesis outline

Chapter 1 Introduction: Brief description of the thesis chapters, and a list of the main aim and objectives.

Chapter 2 Literature Review: Thorough review of seminal papers regarding London Clay and Chalk formation, and highlight the current uncertainties and geological anomalies within them, as well as previously identified faults or periglacial features.

Chapter 3 Cone Penetration Test (CPT) and Piezocone (CPTu): Description of relevant features, such as: Soil behavioral classification, Soil behavior type (SBT) charts, and limitations. Also some typical CPT and piezocone profiles, performed in Clay and Chalk, to aid in the interpretation were included.

Chapter 4 Methodology: Description of the proposed work flow to identify weakened zones using CPT and piezocone data.

Chapter 5 Results and interpretation: Analysis of the available CPT and piezocone data at St. James' Square and the Heart of East Greenwich projects following this proposed work flow. Summary of the procedure followed in each software package, mainly about processing CPT and piezocone data with CPeT-IT®, and how to input CPT and piezocone data into Move® for further interpretation. Interpretation of mechanical impact of faulting on London Clay formation, identified at St. James' Square, as well as, potential weakening in Chalk formation induced by a near off-site fault, at the Heart of East Greenwich.

Chapter 6 Discussion: Main findings related to the identification of faulting and its impact on strength, identified at St. James' Square and the Heart of East Greenwich, employing CPT and piezocone data.

Chapter 7 Conclusions and recommendations: Conclusions embodying main findings and limitations, as well as recommendations for further research.

2 LITERATURE REVIEW

The present chapter provides an insight of the London Clay and Chalk formation, their complexity and how it may affect ground engineering.

This research project was focus on two case studies: (1) St. James' Square at which London Clay formation was found, and (2) the Heart of East Greenwich at which Chalk formation was found.

At the end of this chapter, a summary of the existing information of the two study areas is presented, the information involves: Identified faults, geomorphological and structural features, and site investigation campaigns.

2.1 Regional Geology

2.1.1 London Basin

According to de Freitas (2009), the dip of the Chilterns (North of London) and the North Downs (South of London) is a clear evidence of a basin structure, which was later filled by sediments.

The London Basin has been compartmentalized into blocks by faults, which coincide with a system of river and tributary channels which run through the London Basin which is retrained by discontinuities (present in the bedrock and basement) running NE to SW and SW to NE (de Freitas, 2009), also it is expected that part of the system of discontinuities may undergo shear and hence faults. As it was pointed out earlier by Cunningham & Mann (2007), an arrangement of dip-slip and strike-slip faults may create a series of transpressional and transtensional structures, which in turn leads to the weakening of some areas and also topographic features such as high elevation areas or lowland due to transpressional and transtensional structures, respectively. The transtensional structures may explain the presence of drift filled hollows.

A practical evidence of the above mentioned has been outlined by Ghail et al. (2015). Ghail et al. stated that existing site investigation campaigns carried out all over London, along with river deposits of the Thames and its tributaries unveiled complex ground conditions. The latter has caused many delays to engineering works. Moreover, Ghail et al. put forward the idea that the present anomalies are a result of a continuous inversion of the London Basin by these major faults.

It is clear that the London Basin should be investigated in detail, and because this research was focus on specific sites, the local scale is more relevant. In this regard

periglacial features; existing faults and river deposits were the features to look for during the strength variability analysis.

2.1.2 London Clay

It is of paramount importance to highlight the relationship between geology and engineering behavior, to avoid failures as the Heathrow tunnel collapse in 1994.

The number of complex projects is increasing, hence a deep understanding of the London Clay and how its geological features influence engineering is a key aspect.

The London Clay has been assumed to be relatively uniform, but as it was pointed out by Ghail et al. (2015), it is not and has many subsurface features that influence engineering. In this regard, in the following paragraphs some of the main geological features of London Clay were summarized. This information helped to establish a base line for comparison, as the main objective of this research was to look for reworked material or weakened zones by relating in situ testing (CPT or Piezocone tests), and the geology of the site.

In the late 90s, King (1981, 1991) proposed a biostratigraphic division of London Clay based on depositional characteristics. The biostratigraphic division could be used to age and categorize the sediments. So, if a borehole is available, it is possible to relate geology to engineering properties.

As part of the post-depositional processes, there are 3 major factors:

- The influence of the Alpine orogeny.
- Erosion.
- Weathering.

The network of faults present in London Clay, were possibly generated as a result of the Alpine orogeny, these network have divided London clay into blocks. Also there is evidence of relative movements between each other by the order of meters, breaking and generating discontinuities within London Clay (Gasparre, 2005).

Evidence of hundreds of meters of erosion that has happened in London Clay during the Tertiary and Pleistocene epochs, explain the over-consolidated state of London Clay. This erosive process affected also the materials deposited on top, with a greater influence along Thames valley, later on some gravel layers were accumulated on top of London Clay (King, 1981; Gasparre, 2005).

Some features of London Clay, like brownish color, sub-vertical discontinuities and periglacial structures, are all a result of weathering. According to Hight et al. (2003), the weathered zone varies between 3,0 and 6,0 m thick, which is related to the lithology. In some areas where London Clay outcrops, the weathered zone is thicker (i.e. 9,0 m). In general the clay is highly weathered in the first 1,5 m, but as we go deeper, the influence of weathering decreases, except for some changes in color related to oxidation. In areas where terrace gravels underlie the clay, the weathering process is limited to a narrow range just after the gravel layer. The weathered clay layer is mainly 1,0 m thick, as well as soft and fissured. Below the weathered clay, the weathering process is insignificant and the transition between weathered and unweathered is very clear.

2.2 Strength of London Clay

2.2.1 Discontinuities

Since mid-50s, the relevance of natural discontinuities in the London Clay was highlighted by several researchers i.e. Skempton et al. (1969), consequently its common types of discontinuities were classified in 5 categories, these are:

- Bedding.
- Joints.
- Fissures.
- Faults.
- Sheeting.

The beddings typically contains some silty soils, combined with pieces of wood or shells, they may be horizons between different lithological sequences. Joints are described as extensive fractures, intersecting in a curvilinear pattern, with a vertical to subvertical orientation. Fissures are more localized and they are constrained to beds or layers, these can be defined as narrow fractures present in Clay and siltstone layers. Faults are buried structures, closely related to strength variability at larger scale. Skempton et al. (1969) defined them as low-angle joints.

The bedding structure and erosion processes which underwent in London Clay, can be explained by the arrangement and direction of the discontinuities, this is worth noting because engineers may be able to identify the primary cause of their formation. Fookes & Parrish (1969) (as cited by Gasparre, 2005) pointed out that even if fissures are chaotically spread and have a non-uniform area, their shape and topography made possible to assess the causes of their origin.

For instance, joints which underwent shear have polished surfaces and curved shape, compare to those related to tension which are planar and rough. In general, the orientation of both kinds of failure is related to tectonics and folding. In London Clay, they may have been generated chronologically at the same epoch as the syncline, especially during the Alpine orogeny.

The effect of discontinuities on the strength of London Clay was studied in great detail by Skempton et al. (1969). They drew attention to the fact that minor fissure, joints and faults may decrease the strength to “as low as one fifth to one tenth” compare to intact samples. This was illustrated by case studies where the strength of the brown London Clay decreased after several years due to a strength softening localized in fissures, leading to a progressive failure. They restated their opinion by highlighting the usefulness of structural geology, as a way to unveil the origin and occurrence of discontinuities.

2.2.2 Filled Hollows across London

At one of the study areas, specifically at St James’s Square, Drift filled hollow was identified by Dixon (2015). For this reason, in this section a brief summary of the characteristics and typical dimensions of this type of anomaly was prepared.

With a widths ranging between 50 and 500 m, and depths of the order of 25 and 60 m, drift filled hollows (DFHs), are one of the most hazardous unexpected ground conditions for engineering in the London Clay basin. The uncertainties are mainly related to its extent, location, infilling material, presence of perched water, altered ground conditions along its sides, weakened areas. Even though their origin is unclear, several hypotheses has been made about it, as Toms et al. (2016) points out that there are two types of DFHs, a shallow one related only to scouring and the other related to a combine effect of a network of faults and scouring.

Many of these structures where identified in the South Lambeth-Battersea-Westminster-Southwark zone, with a varying shape. One example of the influence of DFHs on engineering is clearly represent by the Northern Line Extension (NLE), which pass through many Thames tributaries, and as it has been discussed earlier, these areas are likely to present some of these DFHs, also characterized by a high groundwater flux, perched water, and active faults with some differential displacements (Toms et al., 2016).

2.3 Mechanical impact of brittle tectonics on London Clay

Several researchers, has pointed out how tectonism has reduced London Clay strength, some of them are:

Skempton et al. (1969): Several discontinuities, in London Clay, some related to tectonism has reduced intact London Clay strength to 1/5th and 1/10th its value.

Chandler et al. (1998): Highlighted the fact that unsheared clay has relatively higher shear strength compare to clay samples retrieved from a shear zone (See Figure 2.1).

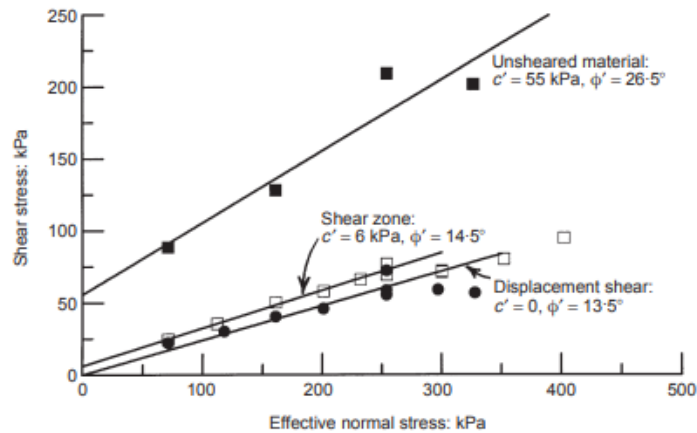


Figure 2.1: Shear strength intact and reworked London Clay by Chandler et al. (1998)

Linde-Arias et al. (2017): Reported that the values of undrained shear strength for London Clay at the Limmo site (where faults were identified) were lower compare to other sites where faults were absent (See Figure 2.2).

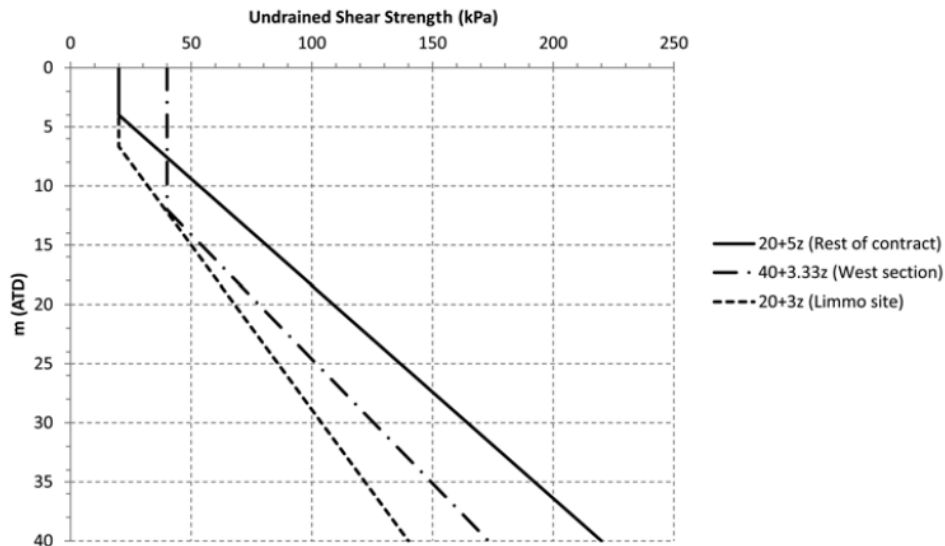


Figure 2.2: Undrained shear strength obtained for Limmo site, and for all other Crossrail sites by Linde-Arias et al. (2017)

2.4 Chalk

Chalk has been used as an engineering material and its geology and strength properties, helps to determine the most suitable construction techniques.

2.4.1 Chalk discontinuities and weathering

According to Lord et al. (2002), style of fracturing, aperture and weathering are the most relevant factors influencing engineering in Chalk. For instance style of fracture may determine the design of a road cutting, whereas weathering and aperture may induce instability in tunnels or controls mass permeability, respectively.

2.4.2 Classification of Chalk by grades

As reported by Lord et al. (2002), initially Chalk was classified using Mundford grades with the objective to relate the structure of Chalk to mass strength properties. Unfortunately, these grades were for a specific site, and obtained under certain conditions, but practitioners generalized its use (See Table 2.1).

To overcome the inaccuracies related to Mundford grades, a new engineering classification system was proposed by Lord et al. (2002), which has a more general approach, based on 3 rock mass characteristics: (1) Hardness of the intact chalk, (2) spacing and pattern of discontinuities or bedding, and (3) aperture (See Table 2.2).

Table 2.1
Mundford grades (after Burland and Lord, 1970; and Wakeling, 1970; based on Ward et al., 1968)

| Grade | Brief description |
|-------|---|
| VI* | Extremely soft, structureless chalk containing small lumps of intact chalk |
| V | Structureless remoulded chalk containing small lumps of intact chalk |
| IV | Rubbly, partly weathered chalk with bedding and jointing. Joints 10–60 mm apart, open to 20 mm, and often infilled with soft, remoulded chalk and fragments |
| III | Rubbly to blocky unweathered chalk. Joints 60–200 mm apart, open to 3 mm, and sometimes infilled with fragments |
| II | Blocky medium-hard chalk. Joints more than 200 mm apart and closed |
| I | As for Grade II, but hard and brittle |

* Added by Wakeling (1970)

Table 2.2
CIRIA grades by Lord et al. (2002)

| CIRIA grade | Chalk type |
|--------------------|---|
| A | Structured with bedding and/or jointing |
| B and C | Structured with bedding and/or jointing |
| Dc | Structureless, clast dominated |
| Dm | Structureless, matrix dominated |

2.5 Case studies

The main objective of this research project was to show the benefits of using CPT and Piezocone data to identify the mechanical impact of faulting on London Clay and Chalk formations. Two sites were chosen as they had CPT and Piezocone data, these sites were:

- St. James' Square.
- The Heart of East Greenwich.

In the following paragraphs a summary of research projects, site investigation campaigns, or geological descriptions that were carried out in these sites is presented. The information was used to support the main findings and to raise awareness about the fact that available CPT and Piezocone data around London, may be useful to mitigate the hazards related to complex and risky ground conditions.

2.5.1 Case study St. James' Square

This research was carried out by Dixon (2015). The project was located 200 m NW of St. James' Park, almost at the middle in between Green Park, Charing Cross and Picadilly Circus London Underground Stations.

This research was focused on a qualitative assessment of the impact of the local geology and geomorphology on the shape of the top surface of London Clay. To build the 3D surface of the upper limit of London Clay, Dixon (2015), gathered information about site investigation campaigns, 3D images, site walk over, and photographs taken during the excavation of the site.

Based on the 3D model of London Clay, Dixon (2015), managed to identify some faults, which then were subjected to a detailed analysis to verify its applicability according to

existing research, geological maps, and formational processes of the London Basin. To confirm if the potential faulted zones underwent shearing, Dixon used laboratory results, water content profiles and Standard Penetration Tests (SPT) and Piezocone (CPTu) tests.

Dixon's main findings were summarized in the following paragraph, taking in to account the main limitations of his conceptual 3D model.

Geomorphological structures

In terms of geomorphological structures, Dixon (2015), interpreted the following structures on site:

- Flow channels, representing parallel and conjugate discontinuities.
- Ice wedge casts.
- An elevated plateau.
- A possible closed pingo structure.

Geological structures

Based on existing project reports, site investigation campaigns, laboratory results and geological maps, Dixon (2015), come up with the following faults:

- 2 sinistral strike-slip faults running NW to SE.
- 2 Dip-slip faults oriented W-SW to E-NE.

Discussion of main findings

According to Dixon (2015), the degree of weathering is related to the topography, where highlands and less weathered compared to lowlands, in the latter the weathering ranges from 1,0 m to 3,0 m thick (brown London Clay). Dixon used ground water levels to relate them to topography changes. However, all boreholes were dry, which forced him to assume a groundwater level taking as a reference the bottom of each borehole. His study suggests that by knowing the topography and groundwater conditions, we can get a useful insight of the predicted properties and weathering influence.

He analyzed SPT and CPT profiles separately. From the SPT "N" value no clear evidence was found, in relation to changes in properties or lithology. From the CPT values, along with water content profiles, a clear subdivision of lithology were found, in addition to a vertical offset of 5 m identified in the unit B of London Clay, compare to the ones at St. James' Park, suggesting a zone of faulting.

Finally, based on his 3D model, he interpreted the topographic features, and how these features reflected the geological control exerted over London Clay surface, such as:

- Parallel discontinuities represented by existing channels.
- Distribution of weathering over the whole surface, where highland zones were less weathered compare to lowland areas.
- Vertical boundaries aligned with the existing channels, which may represent a strike-slip fault, which a repeated pattern parallel to it, implying another discontinuity, most likely a another strike-slip fault (See Figure 2.3).
- Two normal faults, which ended up moving as dip-slip faults, related to the deposition of the river terrace deposits and to the elevated stratigraphy, and orientation of channels indicating movement of these faulted zones (See Figure 2.4).

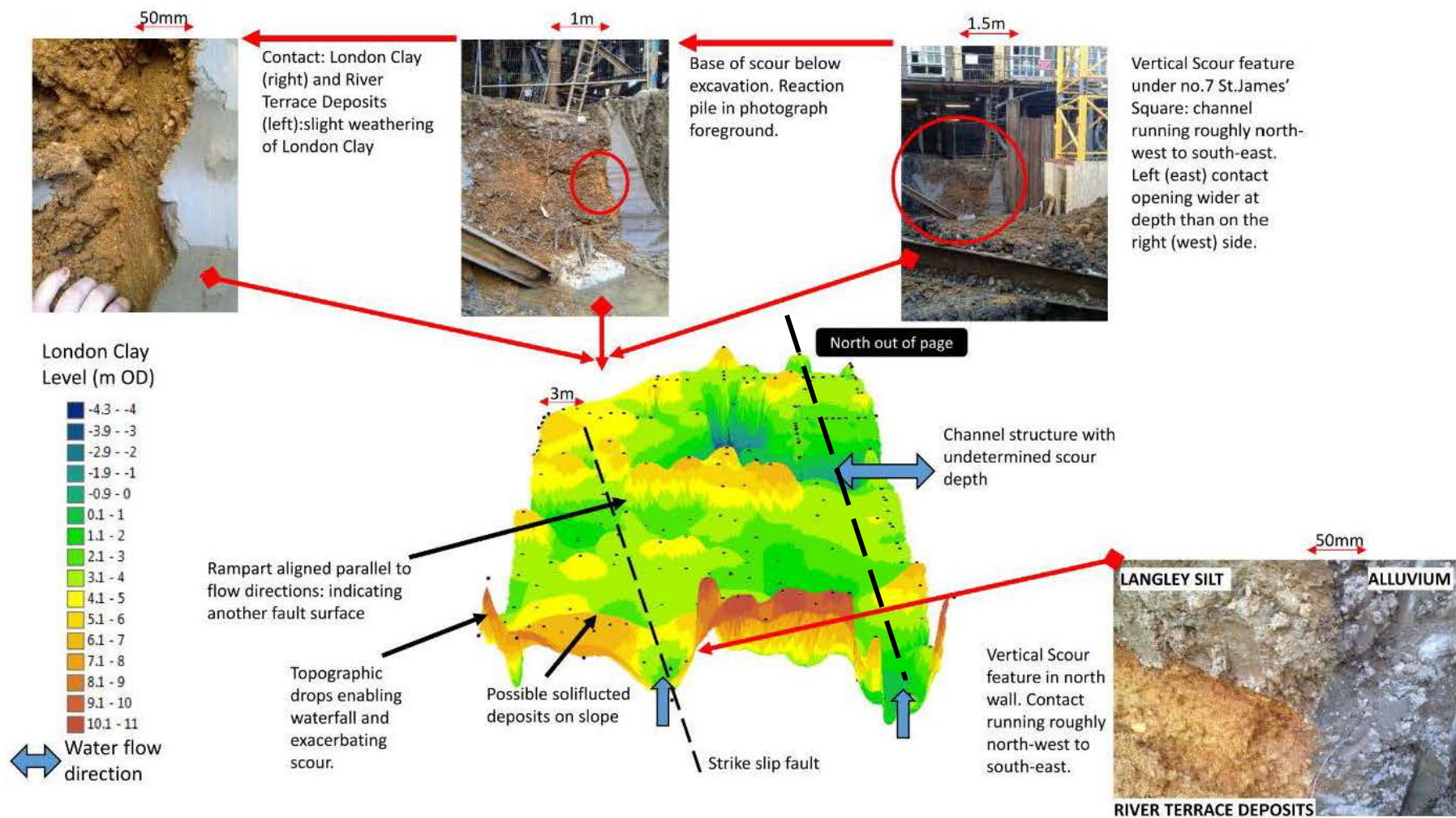


Figure 2.3: Strike-slip faults at St. James's Square by Dixon (2015)

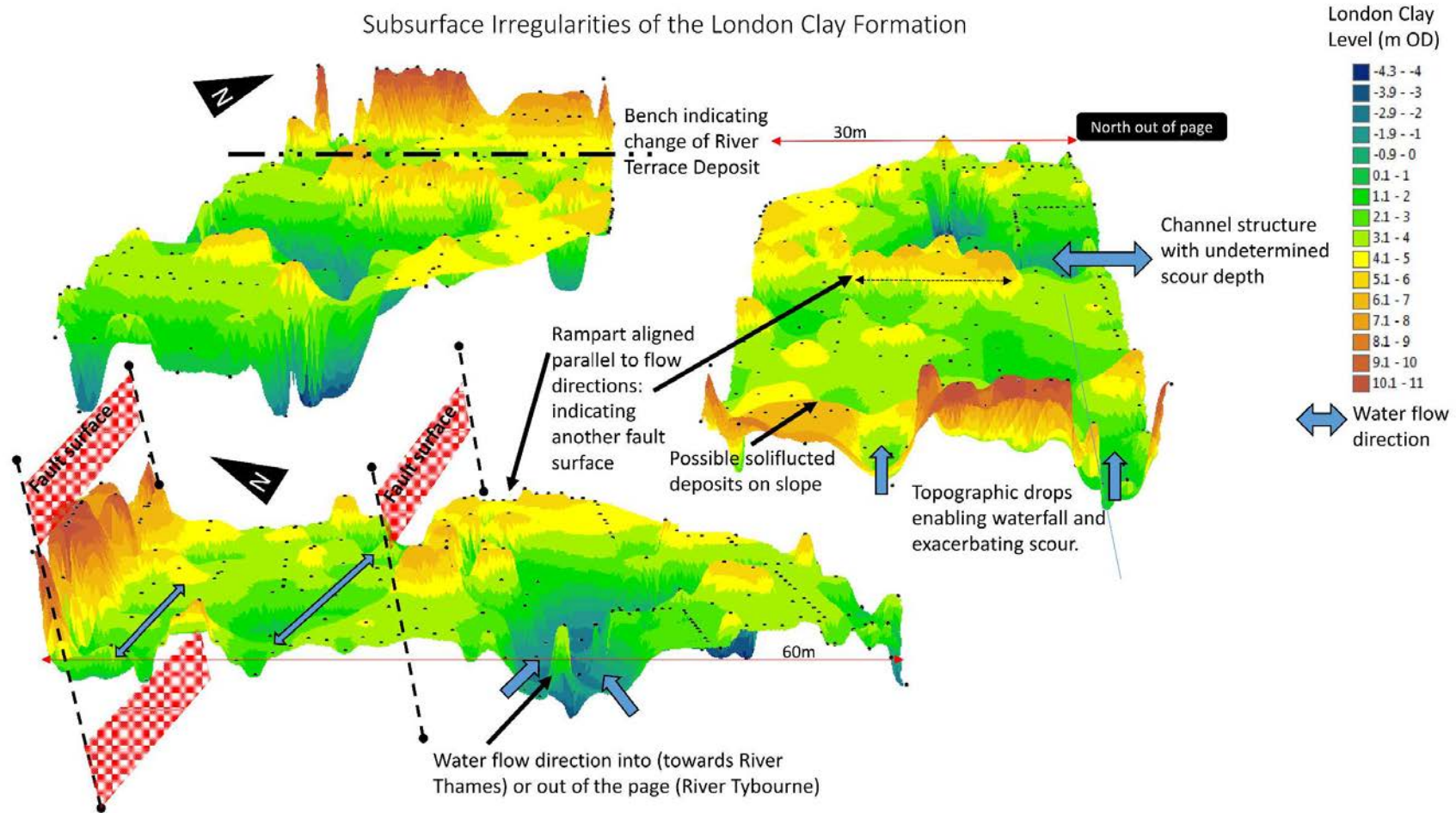


Figure 2.4: Dip-slip faults at St. James's Square by Dixon (2015)

Limitations and further research

The effect of the ground water level variation in London Clay was not considered, as the main factor that shaped the current subsurface was the geological processes. Other features as slopes and evidence of scouring were disregarded.

The identified strike-slip faults are in agreement with similar faults present on a regional scale, mainly about its shape and orientation. Dixon (2015), points out that there is a need of a improved ground model to better define the extent of the buried flow channels, in order to establish the location of the faults accurately.

Dixon recommended carrying out cross-hole seismic tests along with electrical resistivity tests, and based on these 2D profiles a 3D ground model can be generated aiding the interpretation of the buried geological structures. Dixon's recommendations are vital, as the uncertainty about the locations of faults cannot be addressed with conventional methods, in this regard, the use of CPT or piezocone tests (CPTu) may be crucial as they provides a very sensitive sounding of the ground conditions, in the order of centimeters, a further discussion was made in this thesis in Chapter 3 regarding the applicability of cone and piezocone tests in the identification of anomalies within an specific study area.

2.5.2 Case study the Heart of East Greenwich

The study area is located at the Heart of East Greenwich, Woolwich Road, SE10 0DG in London. This area is located at southeast of the Limmo Peninsula, which was one of the most complicated geology of London's Crossrail Project, and adjacent to the River Lea (See Figure 2.5, location map).

Local geology

Based on the available information of the site, published by Digimap, the superficial deposits belong to the Kempton Park Gravel Formation of the Devensian Age, with no artificial ground. The bedrock belongs to the Seaford Chalk Formation of the Coniacian age and Santonia Age. (See Figure 2.6, geological map of the study area).

Existing faults and drift filled hollows in the area

According to the geological map obtained from Digimap, there are two normal inferred faults close to the area of study, one at 300 m to the north and the second one 170 m to the south west. In addition to these faults, Ghail et al. (2015), pointed out that is likely to

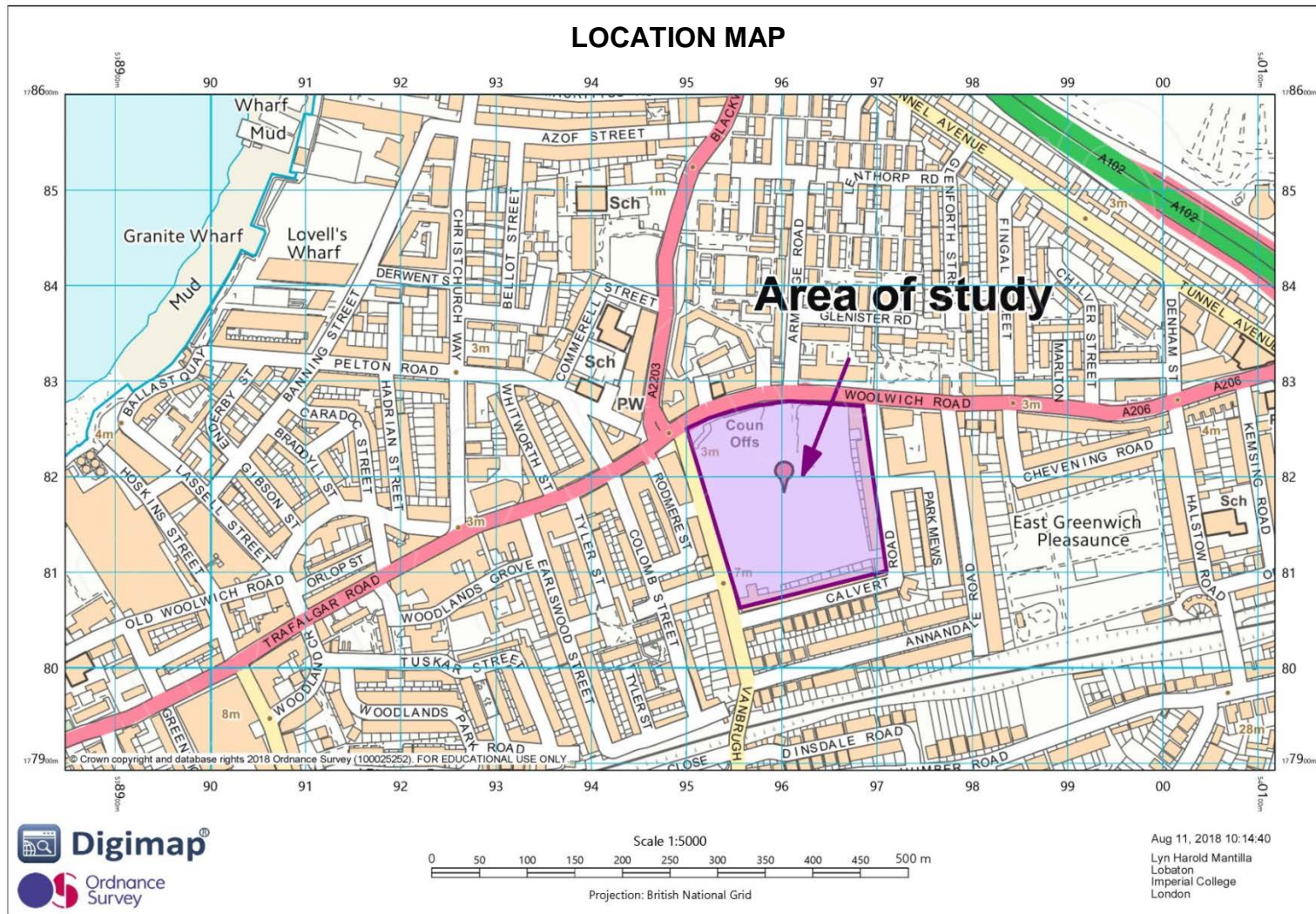


Figure 2.5: Location map

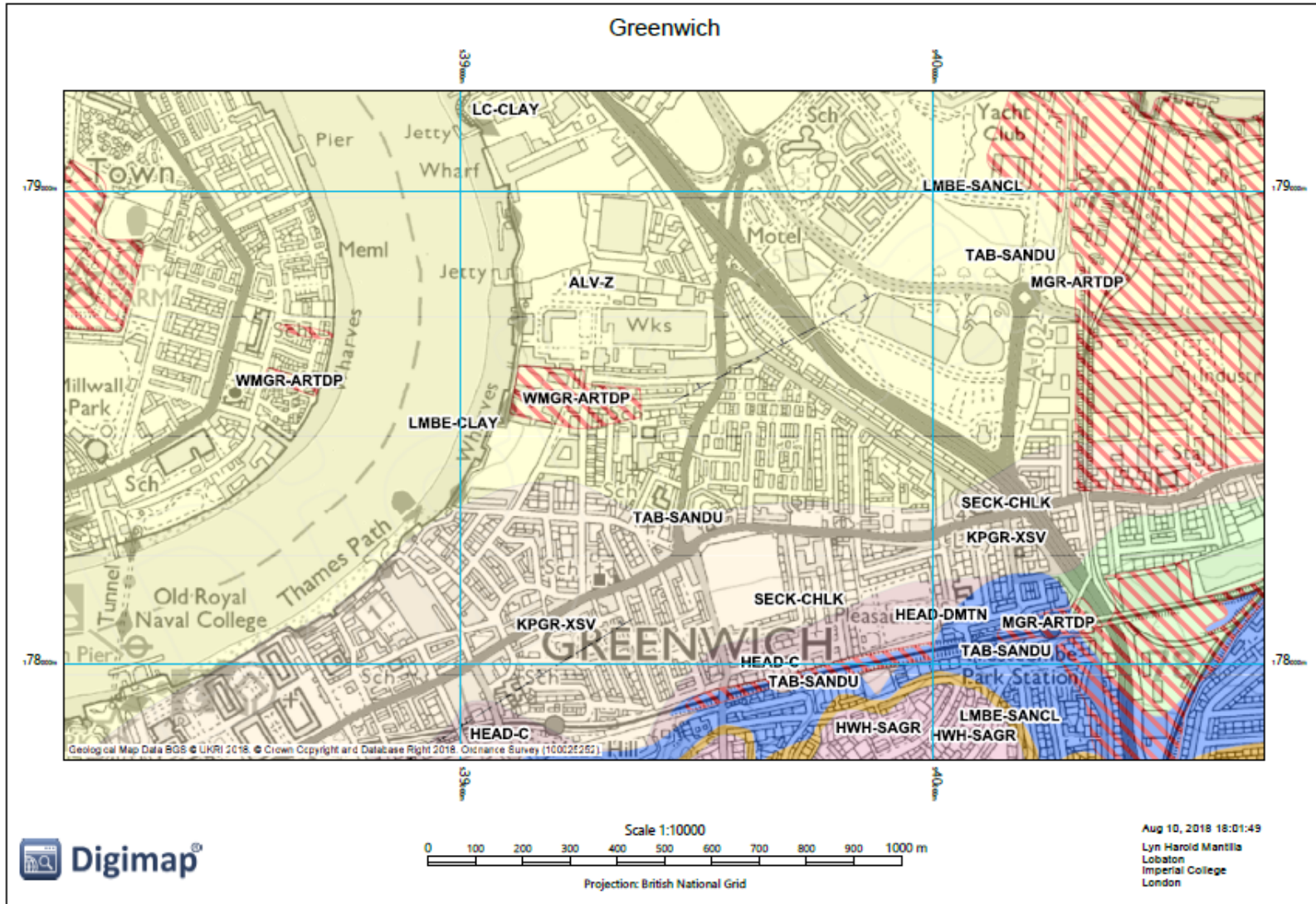


Figure 2.6: Local geology

find some problematic ground areas, close to sharp and straight bends along rivers in London, as these features limit the extent of active transcurrent faults and confining bends. It is worth noting that Mason et al. (2015) (as cited by Linde-Arias et al., 2017), identified an active transcurrent fault, near the study area, running SE-NW. (Refer to Figure 2.7, location of existing fault).

As it was pointed out by Linde-Arias et al. (2017), exceptionally low values of undrained shear strength were found in the London Clay of the Limmo Peninsula, which Linde-Arias et al. (2017) interpreted as a result of tectonics. This peninsula encompasses several drift filled hollows, aligned with rock outcrop on the upper Chalk surface as shown in Figure 2.7, these depressions can reach up to 70 m deep filled with transported material. Their origin may be associated to fluvial scour and pingo formations. Two major drift filled hollows are presented in Figure 2.7, that Linde-Arias et al. (2017) described as “releasing-bend flower structures” given rise to “transtensional basins”.

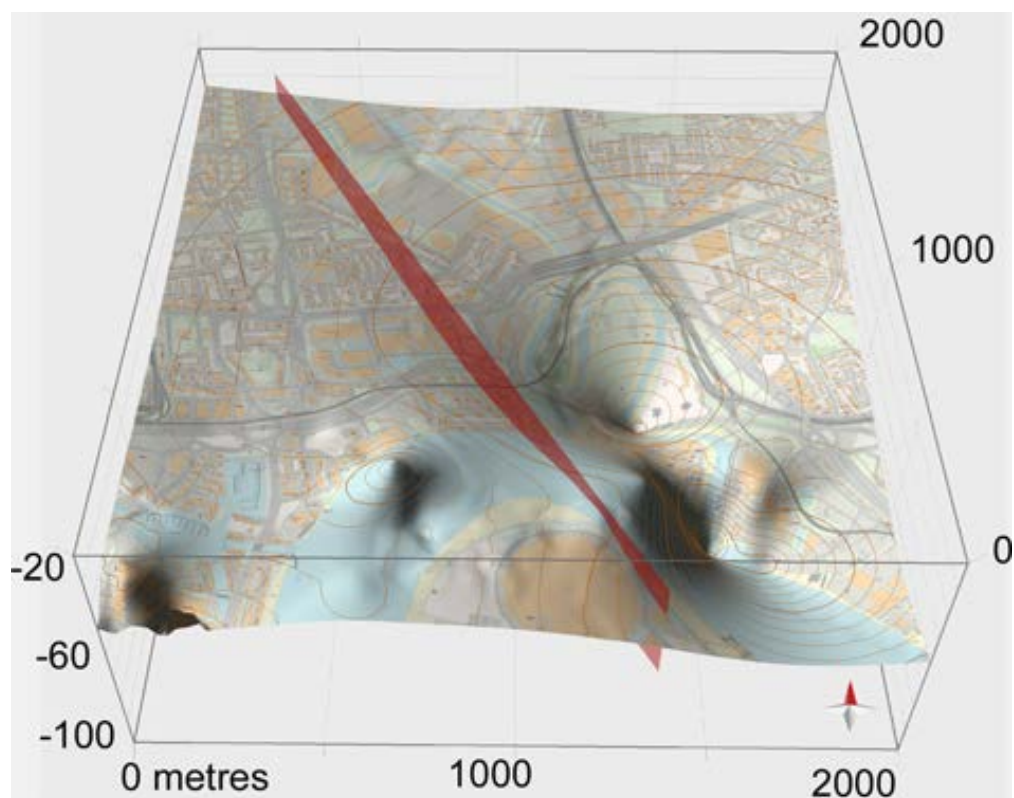


Figure 2.7: Sinistral wrench fault, Upper Chalk surface and Drift filled hollows by Linde-Arias et al. (2017).

Hydrogeology

Two principal aquifers represent the hydrogeological conditions of the London Basin, these are the upper aquifer made up of River Terrace and alluvial deposits, and its water is maintained through a surface groundwater recharge. The second aquifer, is the lower aquifer, and is made up of Chalk and Thanet sand, along with granular soils of the Lambeth Group.

Within the London Basin, the gap between the two aquifers is either London Clay or fine-grained soils of the Lambeth Group (known as aquitards). The lower aquifer's recharge system encompasses water that flows into the Chalk outcropping (north and south of London).

The River Terrace deposits and the Thanet Sand are in contact near the study area, which speeds up the aquifer recharge. Roberts et al. (2015) (as cited in Linde-Arias et al., 2017) stated that there is a granular bed, considered as an area of regional erosional boundary, in the Lambeth Group and Harwich Formation, which is connected just in some areas to the lower aquifer. Linde-Arias et al. (2017) argued that these findings were crucial as the possible connections may be through the existing drift-filled hollows and faults.

Site investigation overview

In this site 01 Borehole was first executed up to 10,0 m depth, using standard cable percussion boring methods with an equipment of 200,0 mm in diameter. Soil samples were retrieved each 1,0 m. The standard penetration test (SPT) was performed at equal intervals of 1,0 m the first 5,0 m, and at 1,5 m the remaining. The registered blow counts "N" were written next to the correspondent soil layer.

Once the river terrace deposits were surpassed, a Geo-bore "S" wireline system was applied, drilling up to 30,0 m depth. Samples were 1,5 m long, each of them were divided and described in situ as the work was progressing. A summary of the borehole log highlighting soil layering, rock basement and blow counts, is presented in Table 2.3:

Table 2.3
Summary borehole log

| Strata | Description | Depth (m) |
|-------------|--|-------------|
| Top soil | <ul style="list-style-type: none">Turf on silty gravelly sand. | 0,0 - 0,05 |
| Made ground | <ul style="list-style-type: none">Brown to dark grey, sandy gravel to coarse sand, with clinker fragments, and subangular to well-rounded flint. | 0,05 - 0,40 |

| Strata | Description | Depth (m) |
|------------------------|--|---------------|
| River terrace deposits | <ul style="list-style-type: none"> • Medium to very dense, brown to yellow, gravelly sand, with clay pockets and subrounded to well-rounded fine to coarse flint. • Blow counts "N" of 13, 18, 21, 81, 38, and 45. | 0,4 - 8,1 |
| Thanet sand formation | <ul style="list-style-type: none"> • Grey to dark grey, clayey silty fine sand to subangular to rounded coarse flint gravel in a sandy clay matrix. • Blow counts "N" of 22, and 29. | 8,1 - 10,5 |
| Chalk | <ul style="list-style-type: none"> • Highly weathered Chalk, with some angular gravel, weak. Upper Chalk Grade Dm. | 10,5 - 10,78 |
| | <ul style="list-style-type: none"> • Cream Chalk, with sub angular gravel, weak to very weak, loose to medium dense. Upper Chalk Grade Dc. | 10,78 - 11,25 |
| | <ul style="list-style-type: none"> • Yellow to orange Chalk, fractures closely spaced, open clean or with loose cream, with brown staining. Upper Chalk Grade C4. | 11,25 - 12,8 |
| | <ul style="list-style-type: none"> • Weak, medium dense, white Chalk, brown staining along fractures, fractures are closely spaced, infilled with washed Chalk. Upper Chalk Grade B4. Presents gravel size Chalk fragments. | 12,8 - 19,5 |
| | <ul style="list-style-type: none"> • Moderately weak, medium to very dense white Chalk, fractures closely spaced. Upper Chalk Grade A3/A4. Presents some coarse gravel fragments, flint and cobble fragments. | 19,5 - 30,0 |

After drilling works, a stand pipe for pore water pressure measurements was installed, the installation was then covered with a padlock and a concrete block 0,30 m height.

56 CPT, and 9 Piezocone tests were carried out by a subcontractor employing a UK17, a truck mounted CPT unit with an ultimate reaction frame of 33,0 ton capacity. The tests were distributed in an square pattern of 10,0 m x 10,0 m over the study area. The plan view of the tests is shown in Figure 2.8. The maximum depth of exploration was 20.5 m, at CPT17. These tests were reported by Savvidou, O. (2012).



Figure 2.8: Plan view of CPT and Piezocone tests at the Heart of East Greenwich (Savvidou, 2012)

3 CONE PENETRATION TEST (CPT) AND PIEZOCONE (CPTU)

3.1 Introduction

The cone penetration testing is a modern sounding technique, which usually involves pushing an electrical penetrometer, testing and recording continuously soil, as it goes down, usually at a rate of 2,0 mm/s, see Figure 3.1. According to international standards (i.e. ASTM D5778-12) three different parameters can be registered, these are: cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u). Under some specific conditions sands or sandy fills can be traced just by using q_c and f_s , whereas for fine-grained soils pore pressures are required to better define soil stratigraphy. When pore pressures are registered, they must be used to correct the cone tip resistance and sleeve friction, by doing this soil properties can be derived, such as: undrained shear strength (S_u), lateral stress ratio (K_0), etc. (Mayne, 2007).

Regarding processing time, it takes around 20 min to get 10 m ground profile from CPT or Piezocone (CPTu) tests, compared to traditional methods. It is environmentally friendly, because no secondary waste is generated, so is much less disturbing than rotary drilling operations. The latter is useful when dealing with contaminated ground, as workers are not exposed to risky conditions (Mayne, 2007).

CPT can be used in a range of different soils, from very soft clays, silts and sands, to very stiff clays or even soft rock, but it may be troublesome in gravelly soils, or hard rock. Usually soil samples are not retrieved, but it is possible to do this, though the samples will be enough for some basic laboratory testing (Mayne, 2007).

If mapping the stratigraphy is of paramount importance, this can be done by using the cone tip resistance. By superimposing CPTs, it is possible to map out weak layers, even their thicknesses over a specific region (Mayne, 2007).

Finally yet importantly, CPT provides a quick and direct assessment of soils in their in situ state, which allow engineers to determine soil behaviour under certain loading conditions. It can be used as a preliminary assessment of geological anomalies, such as: reworked zones, weak layers, etc. It can be further improved with drilling operations, soil sampling and laboratory tests. In particular this research was focus on soil in situ behaviour, and no correlations were used, as the main aim was to provide an insight on how raw and corrected data from CPT and CPTu tests, leads to the determination of subsurface conditions.

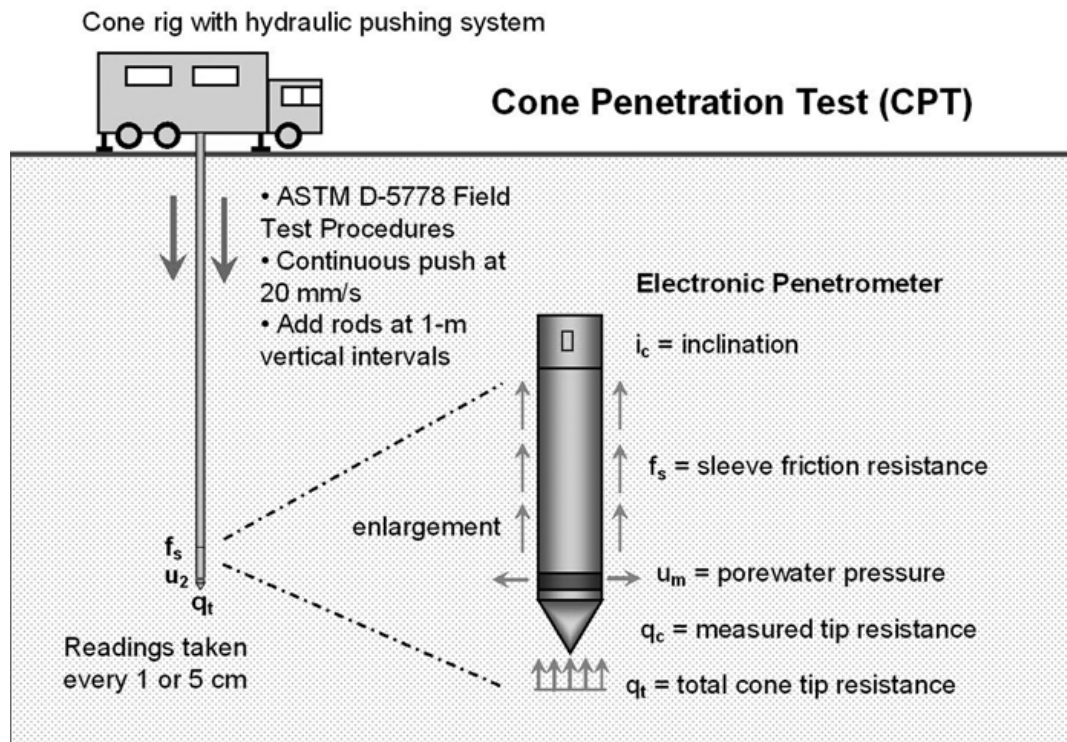


Figure 3.1: Cone penetration test per ASTM D5778 procedures by Mayne (2007).

3.2 Interpretation of Cone Penetration Testing data

The common way of presenting CPT data to aid in the identification of soil stratigraphy, soil behavioral classification, as well as groundwater conditions were summarized (Mayne, 2007).

3.2.1 Stratigraphy

A ground profile is generated from the CPT data by using three different measurements plotted against depth: cone tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u). These are raw uninterpreted results. For the piezocone, instead of the raw cone tip resistance, the corrected tip resistance is used. To properly calculate the excess pore water pressure, it is recommendable to plot the hydrostatic groundwater regime (u_0) (Mayne, 2007). An example of common CPT profiles can be shown in Figure 3.2.

3.2.2 Soil classification by visual interpretation

Soil samples are not usually retrieved during CPT tests, for this reason soil type is inferred from the raw data. Boreholes should be used to complement the raw data, in order to confirm or verify the inferred soil classification (Mayne, 2007).

In order of magnitude, raw values can be ordered as follows: $q_t > f_s$, $q_t > u_1 > u_2 > u_3$. Where the q_t in sands is usually higher than 5,0 MPa (drained conditions), while the q_t for clays is lower ($q_t < 5,0$ MPa) (undrained conditions or low permeability). Similarly, the registered pore water pressure depends on the location of the sensing element and the ground water regime. Pore pressures (PP) for saturated sands are close to hydrostatic ($u_2 \approx u_0$), for clays the values of PP are significantly higher than hydrostatic ($u_2 > u_0$). Instead, if a ratio u_2/u_0 is used, soft clays may have a ratio close to 3, which is even higher for stiff clays (around 10), and much more higher for very hard clays (around 30 or more). The exceptions would be when, clay is fissured and zero to negative PP are likely to be registered (Mayne, 2007).

Another parameter for soil classification is the friction ratio (FR). It is calculated as the ratio between the sleeve friction and cone tip resistance, represented as $FR = R_f = f_s/q_t$, and usually reported in percentage. Some clean quartz sands and siliceous sands showed FR values lower than 1%, on the other hand, low sensitivity clays and clayey silts showed FR values higher than 4%. In high sensitive to quick clays, the FR is close to zero most of the times (Mayne, 2007).

A CPT or Piezocone test could be interpreted using the above mentioned parameters and relating them to soil type. In Figure 3.2 the results of a Piezocone test carried out in Steele, Missouri, displays a ground profile of about 5 layers:

- 0,5 m of sand overlying desiccated fissured clay to 4,5 m, the previous layers overlying a clean sand to 14,0 m, following a soft clay to 24,5 m, and a sandy layer at the end.

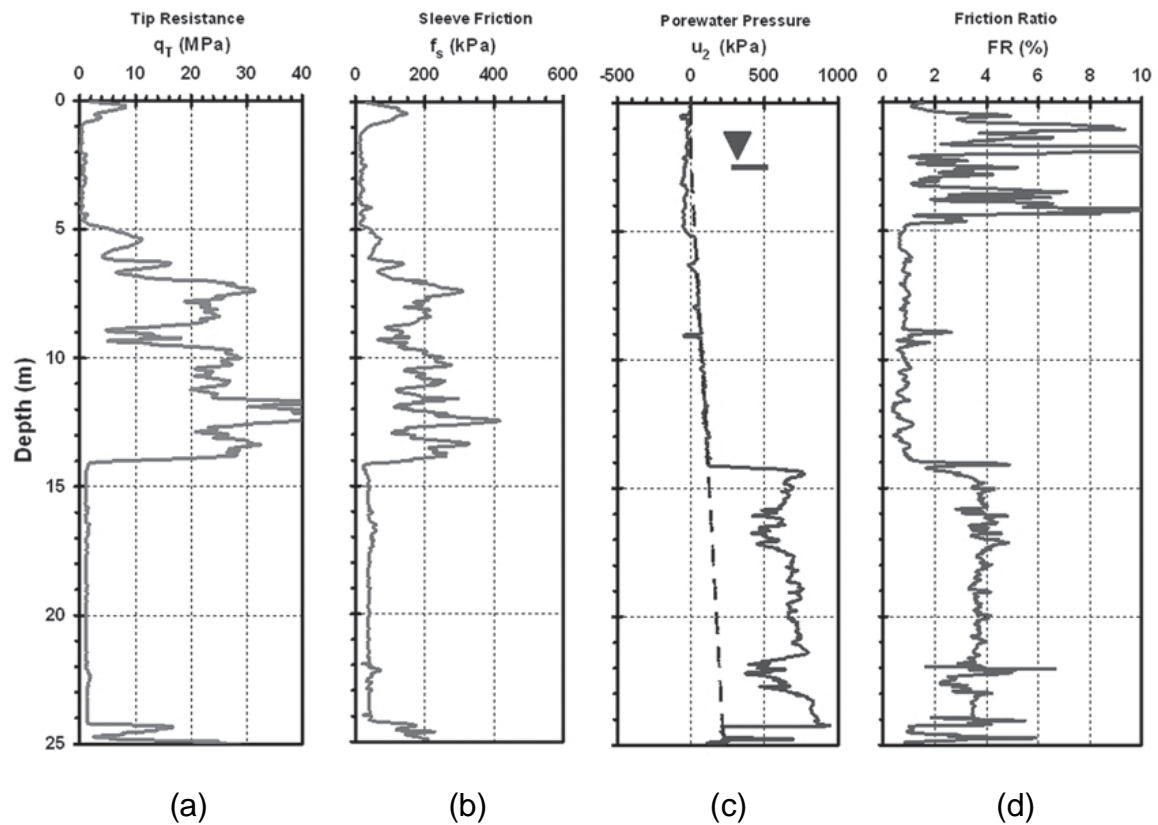


Figure 3.2: CPTu results performed in Steele, Missouri, displaying (a) total cone resistance, (b) sleeve friction, (c) pore water pressures, and (d) friction ratio ($FR=R_f=f_s/q_t$) by Mayne (2007).

3.2.3 Soil behavioural classification - Soil Behaviour Type (SBT) charts

There are more than 25 soil classification methods available some of the authors are Begemann (1965), Schmertmann (1978a), and Robertson (1990) (as cited by Mayne, 2007). Among those, the following charts are preferred:

- Robertson et al. (1986) and Robertson (1990), for CPT and piezocone, with a common SBT description proposed by Robertson (2010).
- Robertson (2016), has updated his charts for soil behavioral classification, to account for microstructure, and to classify soils based on their behavior, for instance, Clay-like - Contractive (CC) or Sand-like - Dilative (SD).
- Schneider et al. (2008), found that the method proposed by Robertson (1990), to account for pore water pressures based on Q_t and B_q is not very accurate, and a better relationship could be obtained using U_2 instead of B_q , where $U_2=\Delta u_2/\sigma'_{v0}$. The resultant chart applies in general to clay like soils, as it is necessary to measure the pore water pressures. However, one challenge for charts built upon

pore water pressures, is that these measurements may not be reliable, especially when the cone has to pass through unsaturated layers.

Based on the above methods, the following table presents the required parameters and their correspondent Charts.

Table 3.1
CPT soil behavior type charts

| Method | Parameters | Associated Charts |
|--|---|--|
| (A) Robertson (1990) | <p>(1) Normalized cone resistance, Q_t</p> $Q_t = \frac{(q_t - \sigma_{vo})}{\sigma'_{vo}}$ <p>(2) Normalized friction ratio, F_r</p> $F_r = \left[\frac{f_s}{(q_t - \sigma_{vo})} \right] 100\%$ <p>(3) Normalized pore pressure parameter, B_q</p> $B_q = \left[\frac{u_2 - u_0}{(q_t - \sigma_{vo})} \right] 100\%$ | <p>$Q_t - F_r$ and $Q_t - B_q$ (See Figure 3.3)</p> <p>Both are normalized SBTn charts, showing contours of I_c, 9 zones to classify SBT, and a zone of normally consolidated soils.</p> |
| (B) Robertson et al. (1986), updated by Robertson (2010) | <p>(1) Dimensionless CPT cone resistance (q_c/p_a) or corrected cone resistance (q_t/p_a)</p> <p>(2) Friction ratio, R_f</p> $R_f = \left(\frac{f_s}{q_c} \right) 100\%$ | <p>q_c/p_a or $q_t/p_a - R_f$ (See Figure 3.4)</p> <p>This is a non-normalized SBT chart, showing contours of 9 zones for each SBT.</p> |
| (C) Robertson (2016) | <p>(1) Normalized cone resistance (Q_{tn}) using a normalization with a variable stress component, n</p> $Q_{tn} = \left[\left(\frac{q_t - \sigma_v}{p_a} \right) \right] \left(\frac{p_a}{\sigma'_{vo}} \right)^n$ <p>(2) Small-strain rigidity index (I_G)</p> $I_G = \left(\frac{G_0}{q_n} \right)$ <p>Where $q_n = (q_t - \sigma_v)$</p> <p>(3) Normalized friction ratio, F_r, it is calculated as in method A</p> <p>(4) Contractive-Dilative boundary (CD)</p> $CD = 70 = (Q_{tn} - 11)(1 + 0.06F_r)^{17}$ <p>(5) Modified soil behavior type index, I_B</p> $I_B = \frac{100(Q_{tn} + 10)}{(Q_{tn}F_r + 70)}$ <p>(6) Normalized small-strain rigidity index, K^*G</p> $K^*G = \left(\frac{G_0}{q_n} \right) (Q_{tn})^{0.75}$ | <p>$Q_{tn} - I_G$ (See Figure 3.5)</p> <p>This is a normalized chart. >Helps to assess soil microstructure (i.e. bonding, aging, cementation), it is necessary to calculate K^*G.</p> <p>$Q_{tn} - F_r$ (%) (See Figure 3.6)</p> <p>This is a normalized chart. >Helps to determine soil behavior type based on a pair of Q_{tn} and F_r (%) values. >Soil type is divided in terms of plasticity by using I_B. Fine-grained soils for I_B values lower than 22, intermediate soils (i.e. silts) if I_B falls between 22 and 32, and coarse-grained soils for I_B values higher than 32.</p> |

| Method | Parameters | Associated Charts |
|--|---|---|
| (D) Schneider et al. (2008) modified by Robertson (2016) | <p>(1) Normalized cone resistance, Q_{tn}, it is calculated as in method C</p> <p>(2) Normalized pore pressure, U_2</p> $U_2 = \left(\frac{u_2 - u_0}{\sigma'_{vo}} \right)$ | <p>$Q_{tn} - U_2$ (See Figure 3.7)</p> <p>Normalized chart with same behavior type terms as in Chart $Q_{tn}-F_r(\%)$ of Method (C).</p> <p>>Is used for pore pressures measured just behind of the tip, u_2 location.</p> <p>>Reflects the behavior of soils in shear at large strain (destructured soil).</p> <p>>Used in conjunction with chart $Q_{tn}-F_r(\%)$ of method (C), may help to identify soil's microstructure.</p> |
| <p>Comments related to method (A)</p> <ul style="list-style-type: none"> I_c, normalized SBT index. It provides more reliable results, but for shallow layers the difference between normalized and non-normalized is negligible, this method is recommended for post-processing. <p>Comments related to method (B)</p> <ul style="list-style-type: none"> Especially useful for real-time monitoring, during CPT tests. P_a=atmospheric pressure = 100,0 kPa. | | |

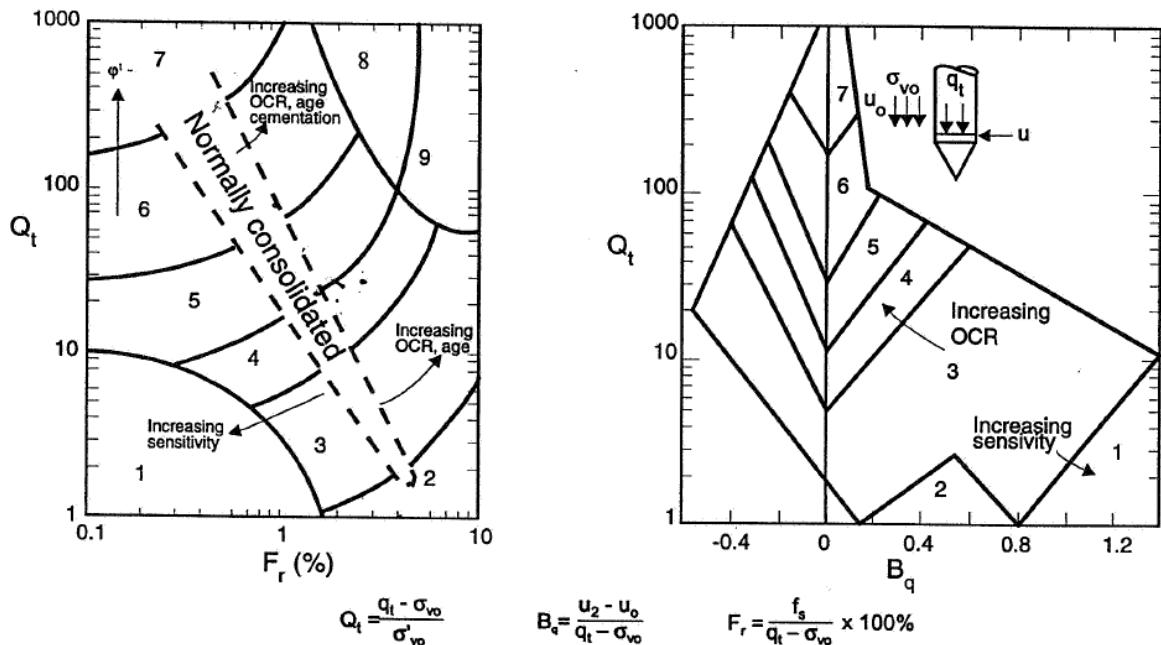
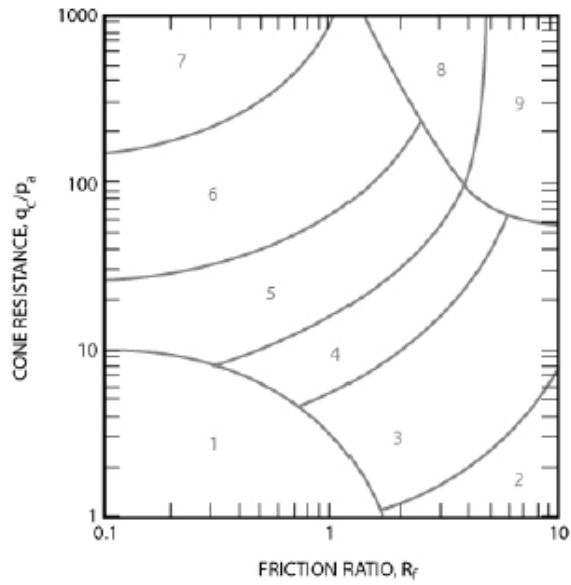


Figure 3.3: Soil behaviour type classification based on normalized CPT/CPTu data (after Robertson, 1990).



| Zone | Soil Behavior Type |
|------|---|
| 1 | Sensitive, fine grained |
| 2 | Organic soils - clay |
| 3 | Clay - silty clay to clay |
| 4 | Silt mixtures - clayey silt to silty clay |
| 5 | Sand mixtures - silty sand to sandy silt |
| 6 | Sands - clean sand to silty sand |
| 7 | Gravelly sand to dense sand |
| 8 | Very stiff sand to clayey sand* |
| 9 | Very stiff fine grained* |

* Heavily overconsolidated or cemented

P_a = atmospheric pressure = 100 kPa = 1 tsf

Figure 3.4: CPT Soil Behaviour Type (SBT) chart (Robertson et al. 1986, updated by Robertson, 2010).

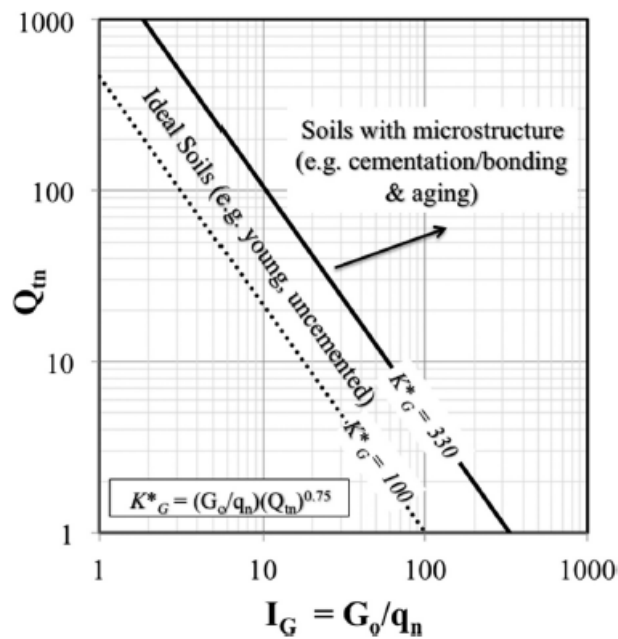


Figure 3.5: Q_{tn} - I_G chart proposed by Robertson (2016).

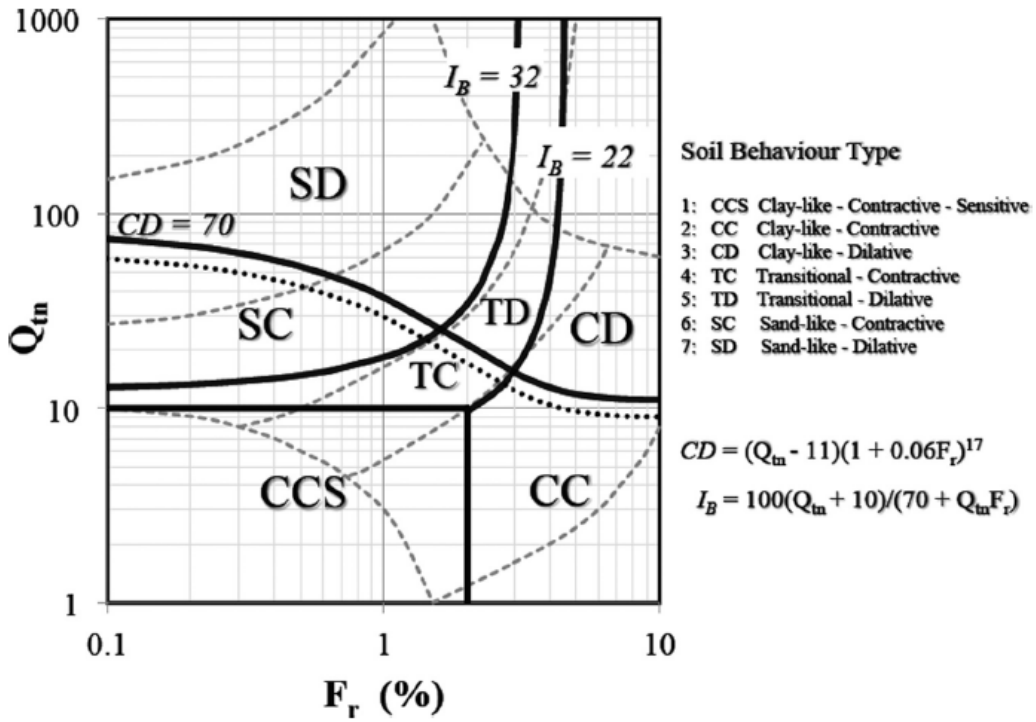


Figure 3.6: Updated SBTn chart based on Q_{tn} - F_r proposed by Robertson (2016).

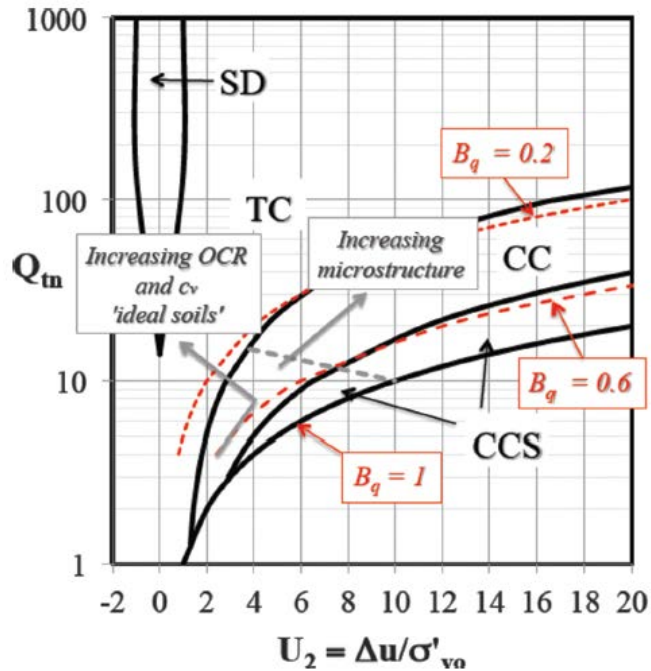


Figure 3.7: Chart based on Q_{tn} - U_2 originally proposed by Schneider et al. (2008) and modified by Robertson (2016).

3.3 Typical CPT and Piezocone results in clay, sand and Chalk

Some typical values of cone tip resistance (q_c), for clay, sand and Chalk, are presented in Table 3.2, these values were proposed by Bustamante and Gianeselli (1982) (as cited by Robertson et al., 2015). The purpose of this table is to aid in the identification of soft or loose layers within the ground profile.

Table 3.2
Typical q_c or q_t values after Bustamante and Gianeselli (1982)

| Nature of soil | q_c or q_t (MPa) |
|---|-------------------------|
| Soft clay and mud | <1 |
| Moderately compact clay | 1 to 5 |
| Silt and loose sand | ≤ 5 |
| Compact to stiff clay and compact silt | > 5 |
| Soft chalk | ≤ 5 |
| Moderately compact sand and gravel | 5 to 12 |
| Weathered to fragmented chalk | > 5 |
| Compact to very compact sand and gravel | > 12 |

Information regarding how CPT and Piezocone parameters (q_c , q_t , f_s , R_f , u_2 , etc.) behave when pushing down the instrumented electrical cone, through different soil layers, was reviewed and summarized based on several research projects, as follows:

3.3.1 Case study in a normally consolidated clay

Lunne et al. (1997), presented a Piezocone profile of a normally consolidated soft alluvial clay. This clay was deposited due to a progressive increase of sea level approximately 8000 years before present. The deposit encompasses clayey silts and silty clays. The stratigraphy is divided into 18,0 m of soft to firm dark grey or black clay, overlying a dense well-graded sand, 3,0 m thick. From a depth of 14,0 m onwards it is more laminated and thinner, this may be related to a change in the deposition environment. The first 1,3 m to 1,6 m of the deposit is a surface crust, with high undrained shear strength.

As explained by Lutenegeger et al. (1989), an overconsolidated surface crust is a result of desiccation and a changing groundwater level, which often overlies a variable thickness of normally consolidated clays. In Piezocone profiles, the normalized tip resistance is usually high and decreases gradually towards a constant value, whereas the normalized pore pressure increases to a peak value across the crust followed by constant values across the normally consolidated strata.

A typical ground profile of the site can be shown in Figure 3.8. It is clear that the water content increases with depth within the clay but decreases when approaching the coarse-grained deposit. The unit weight tends to increase with depth, and the percentage of clay content is characteristic of a clayey silty soil.

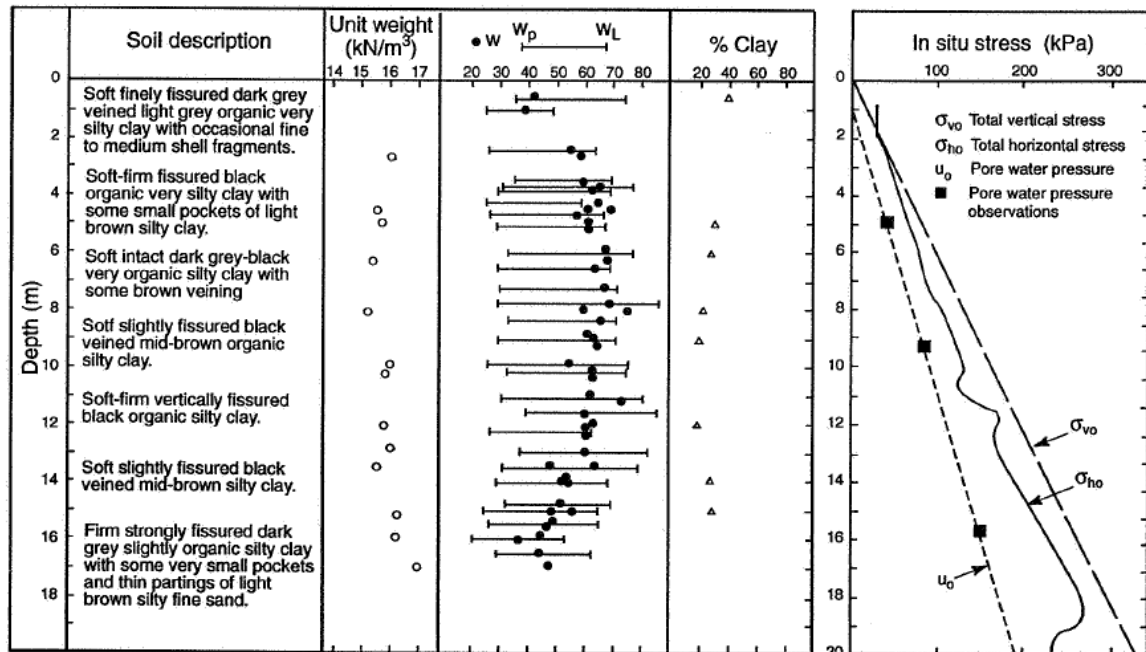


Figure 3.8: Soil profile from Bothkennar (UK) by Lune et al. (1997).

For this site Lune et al. (1997), presented a CPTu profile, with 3 piezometric elements, as shown in Figure 3.9. According to Lune et al. (1997), the pore water pressure profiles are typical of soft clay deposits.

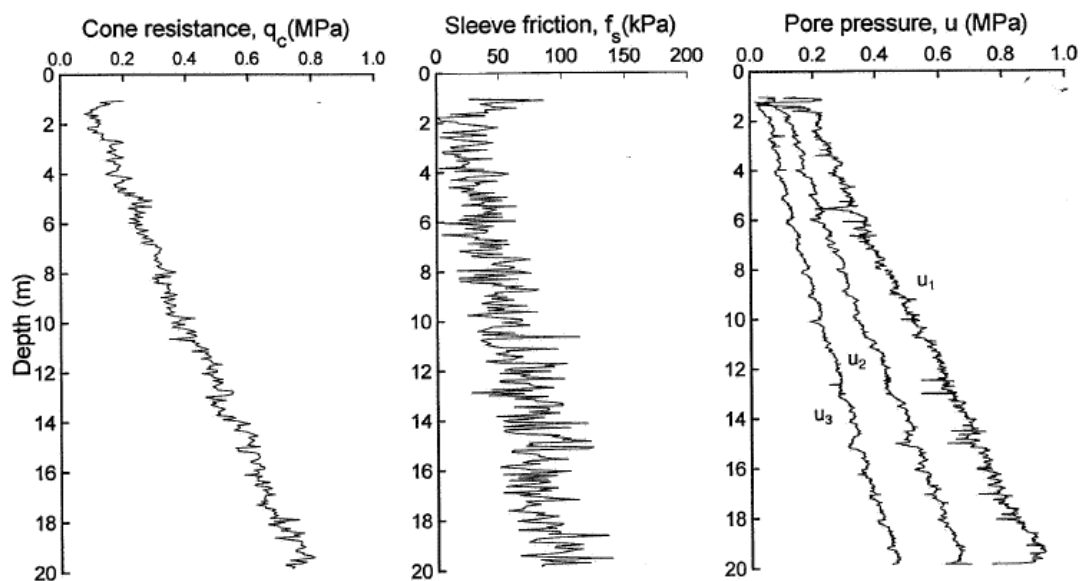


Figure 3.9: Typical CPTu profile, Bothkennar (UK) by Lune et al. (1997).

The calculated CPTu parameters are shown in Figure 3.10, the thinner variations in each profile is related to the different depositional periods. Also the q_t values are lower than 1,0 MPa, which agrees well with Table 3.2. These charts could be expressed in terms of soil parameters, for soft clays typical values of N_{kt} , N_{ke} , and $N_{\Delta u}$ of 10, 5 and 6 were chosen. A good agreement was observed between the estimated undrained shear strength, though is slightly higher than the peak strength from laboratory results (See Figure 3.11).

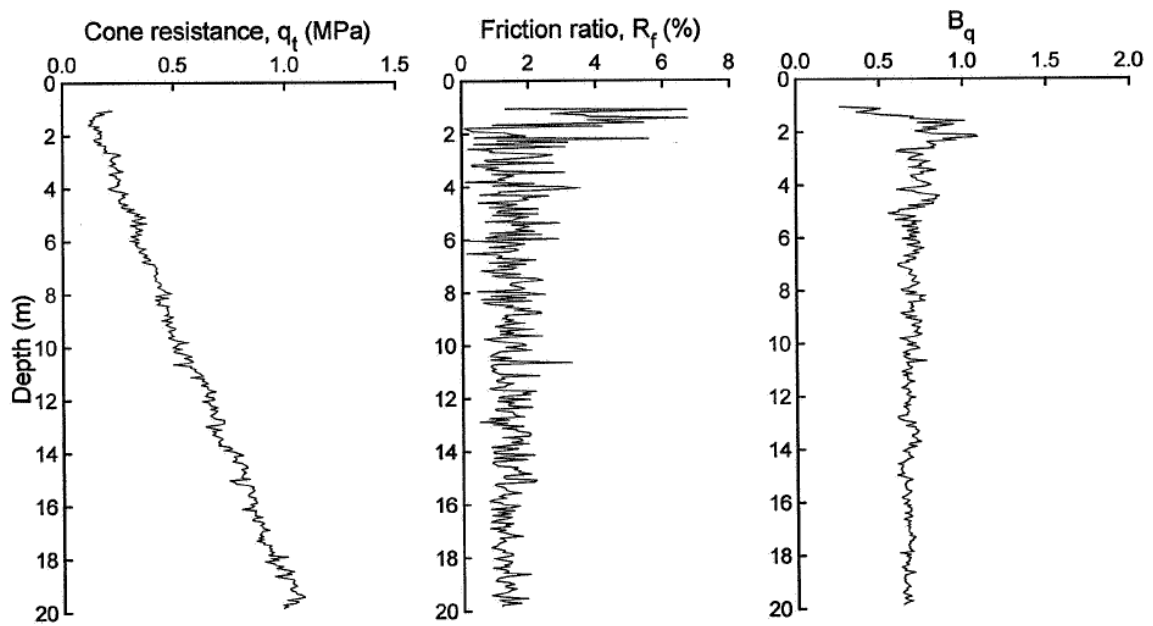


Figure 3.10: Derived CPTu parameters, Bothkennar (UK) by Lune et al. (1997).

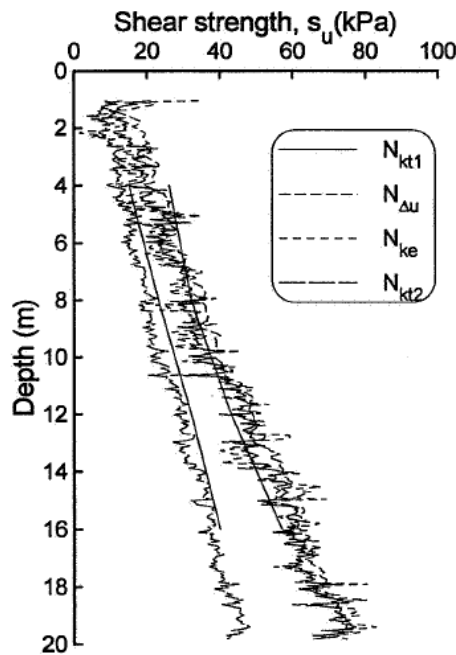


Figure 3.11: CPTu derived shear strengths, Bothkennar (UK) by Lune et al. (1997).

3.3.2 Case study in an overconsolidated clay

Lunne et al. (1997) presented a Piezocone profile of stiff overconsolidated Gault clay at Madingley test site. This clay was deposited more than 110 million years ago during the Cretaceous period, it was deposited in an area which was under constant subsidence, initially loaded with Chalk of about 150,0 m thick, removed later on, which explains the highly overconsolidated state of the deposit.

In this case study, the Gault Clay has a ground profile as shown in Figure 3.12. The first 7,0 m is a weathered, firm to stiff grey/green silty clay of high plasticity (about 50%), deeper (>7,0 m to 13, 0 m) the soil is unweathered and stiffer, densely fissured of similar plasticity. Further down, the plasticity increases, with a slightly decrease in water content, with values close to the plastic limit.

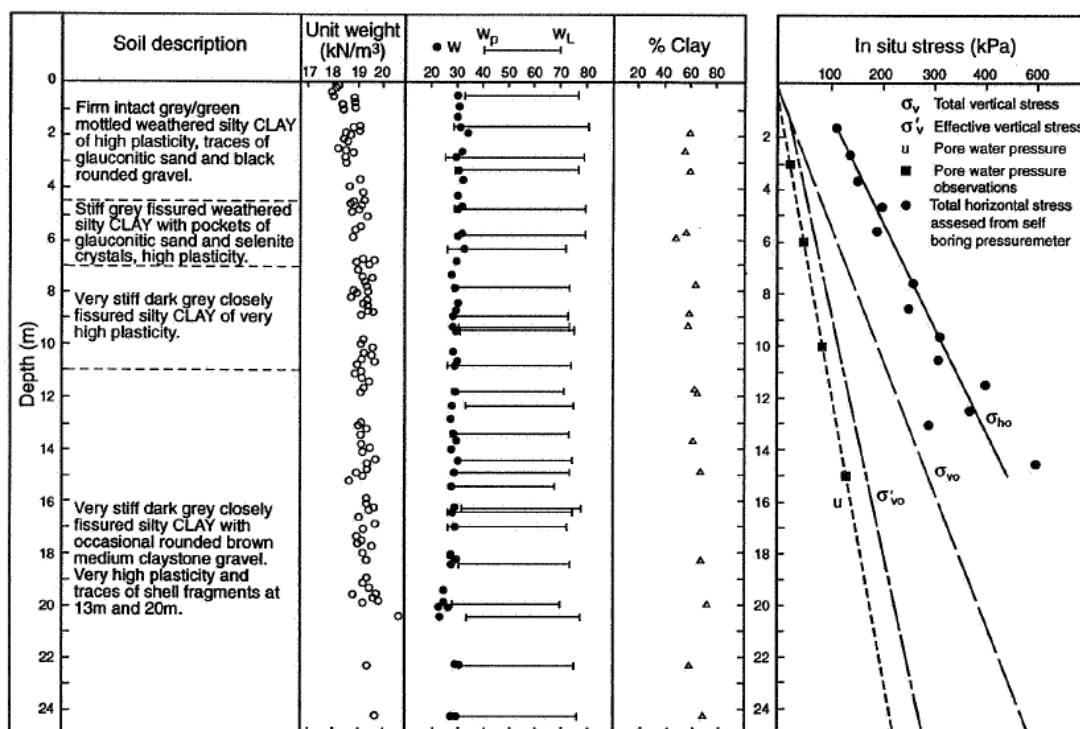


Figure 3.12: Soil profile from Madingley (UK) by Lunne et al. (1997).

Figure 3.13, is a CPTu profile of the site, with 3 piezometric elements in locations u_1 , u_2 and u_3 . Pore water pressures at u_2 and u_3 are nearly zero, whereas u_1 is consistently showing high positive values in agreement to q_c . u_2 and u_3 , were plotted in a different graph, which confirmed that they registered very low pore pressures, representing soil behaviour at these locations.

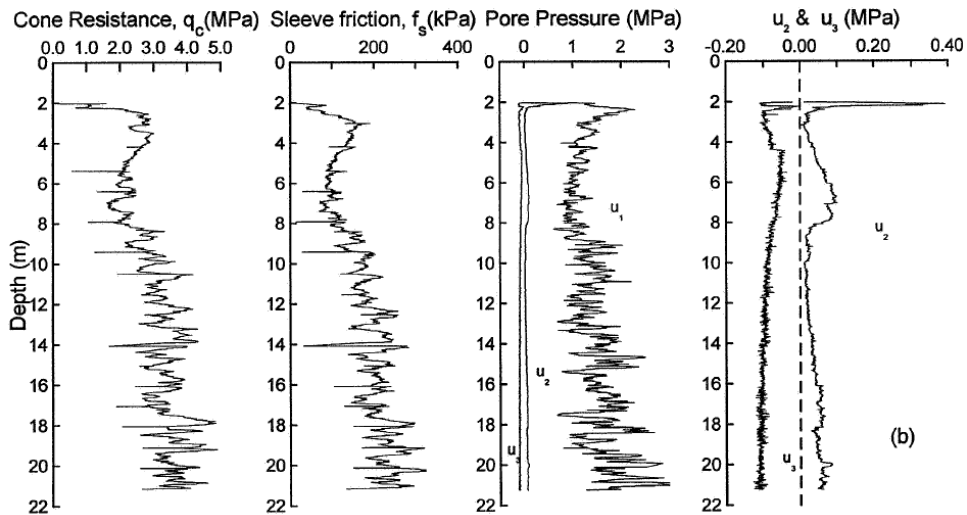


Figure 3.13: Typical CPTu profile, Madingley (UK) by Lunne et al. (1997).

To derive geotechnical parameters was necessary to choose a cone factor (N_{kt}). But because B_q values were too low, Lunne et al. (1997) used the plastic index (I_p) to get the N_{kt} value, which was around 18,0. The problem here was that the N_{kt} - B_q charts require B_q values higher than 0,3, also these charts are for normally to lightly consolidated clays and not for very stiff overconsolidated clay. So, in circumstances when B_q values are negative, it is recommendable to use an N_{kt} - I_p (%) chart.

The resulting shear strength profile is shown in Figure 3.14, and it was compared to triaxial and plate load tests. Overall, the estimated values were higher due to soil fabric and scale effects, to take into account these features; Lunne et al. (1997), used N_{kt} values between 18 and 30 based on different degrees of discontinuities and q_c values. The resultant shear strength profile gave a more accurate prediction.

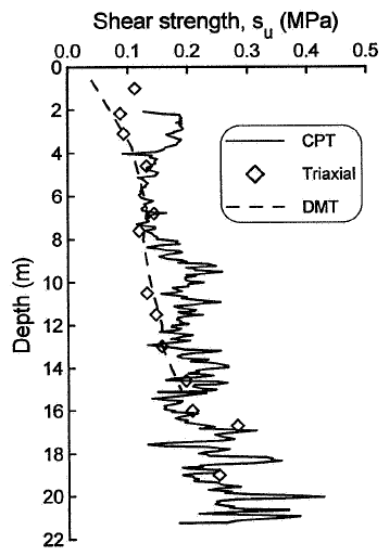


Figure 3.14: CPTu derived shear strength, Madingley (UK) by Lunne et al. (1997).

3.3.3 Chalk

Power (1982) (as cited by Lunne et al., 1997) carried out a detailed study about CPT tests performed in Chalk. Moreover, Power published a table with typical CPT parameters as a function of Mundford grades, q_c (MN/m²) and R_f (%) values (See Figure 3.15).

| Grade | Brief description | q_c (MN/m ²) | R_f (%) |
|-------|--|----------------------------|-----------|
| VI | Extremely soft structureless chalk containing small lumps of intact chalk | < 5 | – |
| V | Structureless remoulded chalk containing lumps of intact chalk | 5–15 | 0.75–1.0 |
| IV | Rubbly partly weathered chalk with bedding and jointing. Joints 10 to 60 mm apart, open to 20 mm, and often infilled with soft remoulded chalk and fragments | 5–15 | 1.0–1.25 |
| III | Rubbly to blocky unweathered chalk. Joints 60 to 200 mm apart, open to 3 mm, and sometimes infilled with fragments | 15–20 | 1.25–1.50 |
| II | Blocky medium-hard chalk. Joints more than 200 mm apart and closed | > 20 | 1.5–20.0 |
| I | As for grade II, but hard and brittle | No penetration | |

Figure 3.15: Chalk grades related to CPT values (after Power, 1982), cited by Lunne et al. 1(1997).

A typical CPT profile in Chalk, at Mundford (Norfolk, England), along with Mundford grades is shown in Figure 3.16. The randomly distributed peaks within the profile were explained as: flints appearing often in Chalk, tendency of cone penetration resistance to increase with depth, particle crushing and/or closure of fissures, changes in density, ageing, and style of fracturing (Lunne et al., 1997).

Later, Powell and Quarterman (1994) (as cited by Lunne et al.1997), assessed the application of CPT tests in different Chalk sites in addition to Mundford site. As part of their results, they highlighted that Power's work was conclusive, as increasing values of q_c and f_s were related to better Chalk conditions with depth (in terms of Mundford grades). However, Powell and Quarterman, stated that even though this trend was similar in all sites, the ranges of q_c and R_f (%) were quite different.

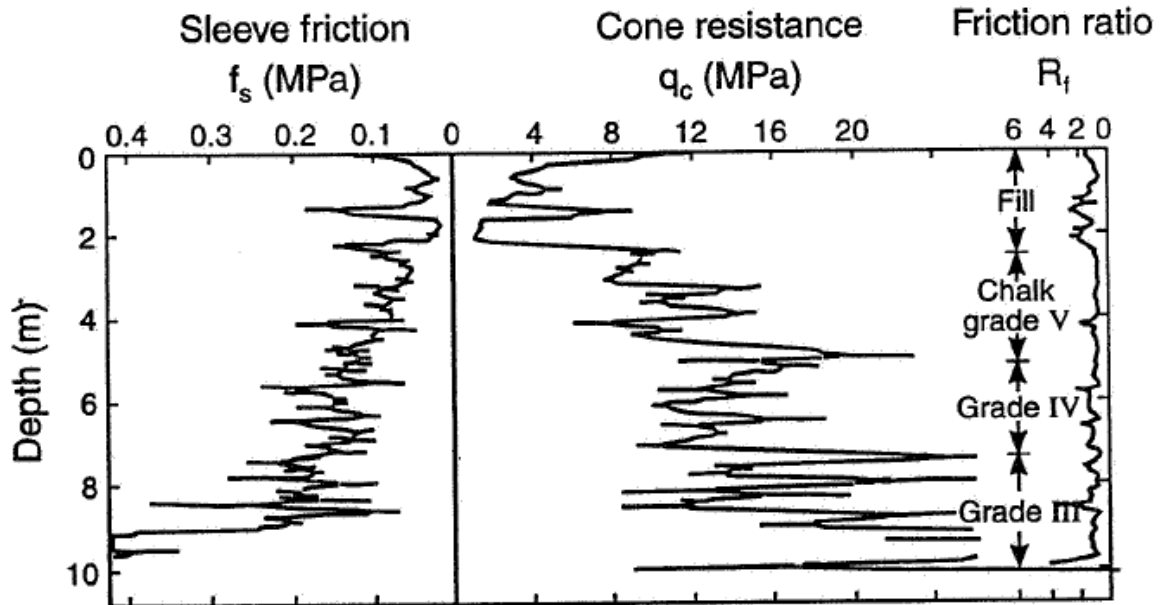


Figure 3.16: Representative CPT profile in Middle Chalk at Mundford, Norfolk, England (after Power, 1982).

Powell and Quarterman, presented their results, including Power's ranges, as shown in Figure 3.17. From this figure it was clear that even for the CPT tests carried out at Mundford site, their data was outside Power's ranges.

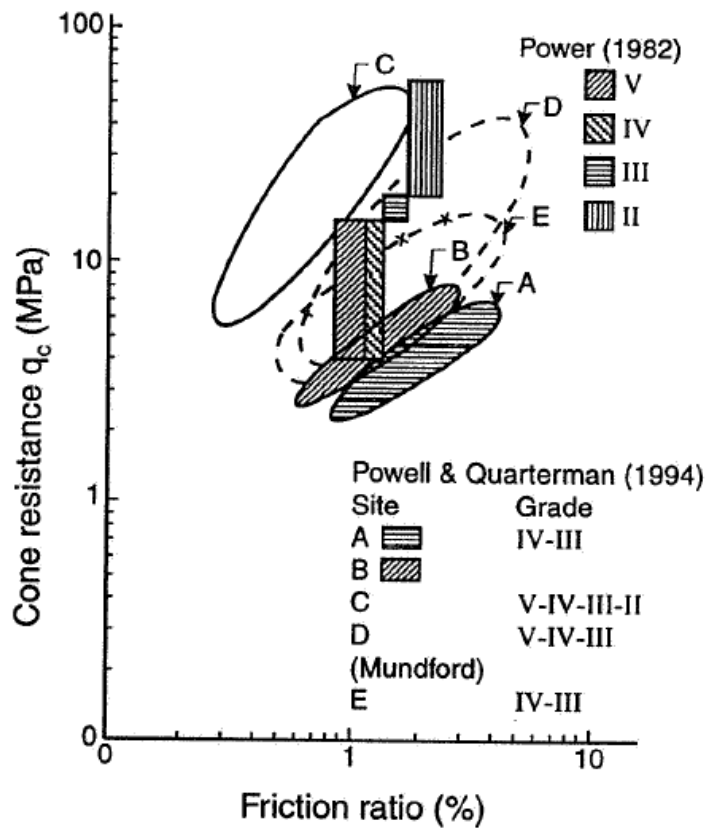


Figure 3.17: Classification of Chalk grade (Powell and Quarterman, 1994).

Chalk density was included as an additional factor to help Powell and Quarterman, explain why the results (q_c and f_s values) were quite different, even for the same classification and site. For instance, they found that site A, with the lowest density, fell in the lower bound of the data.

It is worth noting that, the use of Mundford grades to classify Chalk, were criticized and discussed in detail in the CIRIA report C574 (Engineering in Chalk). In CIRIA C574, it is stated that Mundford grades are applicable only to certain conditions, like medium density chalk, and it does not account for dry density variations of the chalk for different sites. So, Mundford grades cannot be generalized.

More CPT results in Chalk were presented in CIRIA C574 report, such as the work of Smith (2001), who gathered information of 11 different locations. Summarized in Table 3.3. Smith (2001) demonstrated that the Grey Chalk has a higher friction ratio, due to its higher clay content compare to other chalks. Other types of Chalk showed a wider range of friction ratio, that matches Powell and Quarterman's results. The clay content was related to the degree of weathering, because the Northern Province (usually less weathered) showed lower friction ratio values (less clay content) compare to the Southern Province with relative high friction ratio values (3,5 - 5,5 %), and hence higher clay content.

Table 3.3
 q_c and R_f (%) values reported at 11 locations (after Smith 2001)

| | Location | q_c (MN/m ²) | R_f (%) |
|-------------------------------------|---------------------|----------------------------|-----------|
| Grey (formerly Lower) Chalk | Snodland | 4–20 | 3–8 |
| Southern Province Chalk | Dorchester | 4–15 | 0.5–1.5 |
| | Bear Wharf, Reading | 2–20 | 1–1.5 |
| | Oracle, Reading | 1.5–20 | 1–1.5 |
| | Rochester (CTRL) | 10–30 | 1–2.5 |
| | Dartford | 1.5–30 | 1–2.5 |
| | Isle of Grain | 10–30 | 3.5–5.5 |
| | Hemel Hempstead | 1.5–30 | 0.5–3.5 |
| East Anglia (Transitional Province) | Norwich | 2–15 | 1–5 |
| | Saffron Walden | 2.5–20 | 1.5–5 |
| Northern Province Chalk | Immingham | 10–40 | 0.5–1 |

The main drawback of Smith's work is that no direct comparison against borehole data was carried out. However, Smith gave some recommendations if borehole data were available, such as: Identify some weakened zones or infilling features and the possibility to generate some site specific correlations. Always in conjunction with Chalk intact dry density with depth, because as it was shown, the Chalk density influences greatly in the CPT results, as well as stratigraphic interpretation.

Overall, in CIRIA report C574, it is mentioned that due to the broad range of q_c and R_f (%) values, CPT test is unsuitable for interpreting chalk in terms of either density or grade. However, the advantages of CPT tests are highlighted in CIRIA C574. For example: It helps to identify Chalk strata if we assume some typical values of $q_c > 4,0 \text{ MN/m}^2$ and $R_f \approx 1$. Similarly, q_c values higher than 4 MN/m^2 may be associated to compacted chalk fills. In addition, weak layers, manmade or natural cavities, and dissolution features could be mapped.

The above mentioned may serve as reference, for CPTs performed in Chalk. In this regard, Lunne et al. (1997), stated that practitioners should be careful, when extrapolating results from site to site, due to differences in style of fracturing, fissuring, Chalk density, type of infill material, and CPT profiles should be presented along with a detailed engineering description of Chalk from a borehole log, which in turn may aid in the development of site specific correlations.

3.4 Pore pressure distribution

The measurement of instantaneous pore pressure depends highly on the location of the sensing element in relation to the cone tip, which is even more sensitive in overconsolidated clays compare to normally consolidated clays (Lutenegger et al., 1989).

Several researchers (i.e. Lutenegger et al., 1989), have found that the induced pore pressure is not homogeneously distributed either at the cone face or cone shaft. The degree of consolidation affects substantially the non-uniformity. For instance, in overconsolidated clays, the pore pressure is maximum at the cone tip, reducing sharply towards the base of the cone, and nearly constant all the way up along the cone shaft, whereas, in normally consolidated clays, where the pore pressure is much more uniform (Lutenegger et al., 1989).

Usually in Piezocones, the piezometric elements are located at three different positions: on the cone (u_1), behind the cone (u_2) and behind the friction sleeve (u_3) as shown in Figure 3.18. The most common position is behind the cone (u_2). For some research and

complex projects, two or three filter positions are used. Depending on the filter position, the cone tip resistance should be corrected to account for pore water pressure influence (Lunne et al., 1997).

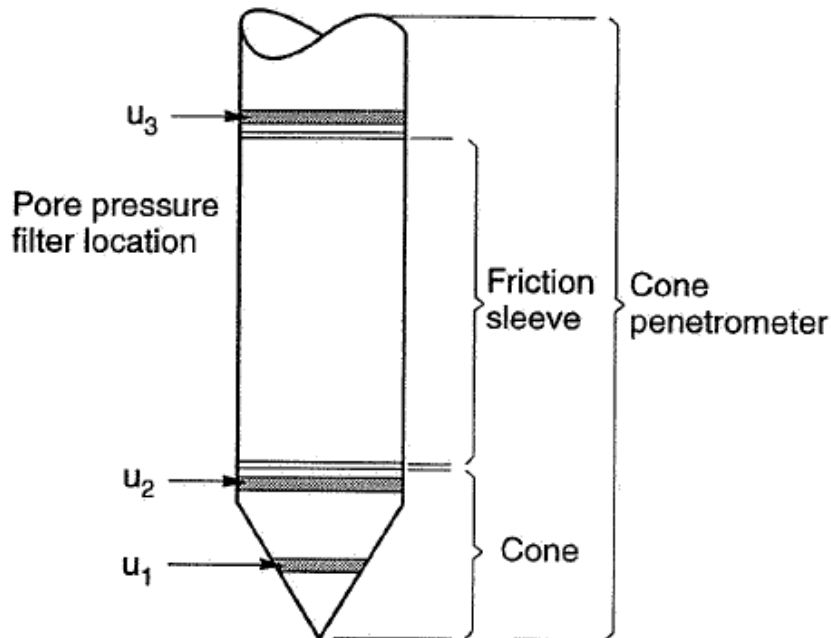


Figure 3.18: Location of Piezometric elements in the Piezocone by Lunne et al. (1997).

3.5 Limitations

3.5.1 Correlations

As point out by Meigh (1989), there is the need for site specific correlations, as many of the available correlations charts are based on specific projects. However, there is an agreement between those, as they show a similar behavioral pattern, which is of great advantage for preliminary designs or when samples cannot be retrieved especially in sands.

3.5.2 Scale effects

This aspect may be of prime importance in clays, in which fissure spacing and fabric may affect: Soil sampling, in situ tests results, etc. One key factor greatly affected by scale effects is the non-dimensional cone factor (N_k), which varies from 10 to 30, assuming the cone diameter is enough to capture the ground conditions (Meigh, 1989).

3.5.3 Resolution and disturbance

As the electrical cone is pushed down, soil is disturbed, and the cone registers the cone tip resistance of the soil which is between 5 and 10 times the cone tip diameter. This

introduces some inaccuracies when trying to identify the limits between different stratus. Because of the above, the resolution of CPT lies within 100 mm thick for a sand layer in a clay deposit, or 150 to 200 mm thick for a clay layer within a sand deposit. The resolution improves if a Piezocone is used (Jardine, 2017).

3.5.4 Correction for pore pressure effects

According to Meigh (1989) the cone resistance (q_c) has to be corrected by the force due to pore pressure developed behind the base of the cone (See Figure 3.19). The resultant value is known as the corrected cone resistance (q_t), and it is calculated as follows:

$$q_t = q_c + u_2(1-a) \quad (1)$$

q_t (MN/m^2): Corrected cone resistance.

q_c (MN/m^2): Cone resistance.

u_2 (MN/m^2): Pore pressure behind the cone.

a : Cone area ratio, ratio of the cross sectional area of the shaft (A_n) to the projected area of the cone (A_c).

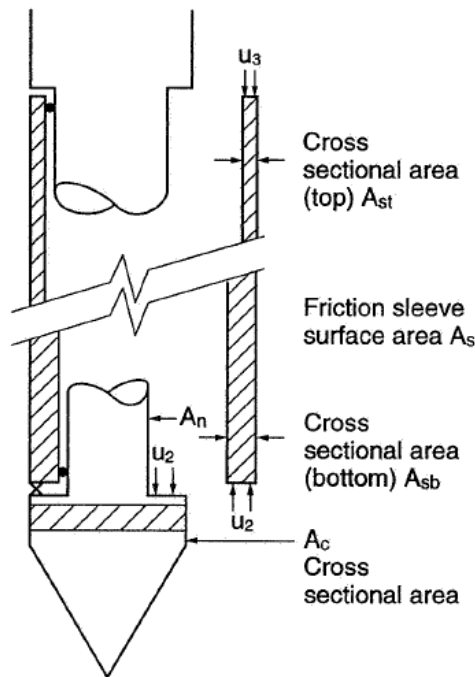


Figure 3.19: Pore water pressure effects by Lunne et al. (1997).

The problem comes when engineers have to decide if the correction will make a difference or will just add more uncertainties. For instance, this correction is relevant in soft saturated soils, as they develop high pore pressures in relation to the cone resistance. Lunne et al. (1997) pointed out that this correction allows to measure the pore water pressure and at the same time to correct the cone resistance. However, to do this the measurements should be accurate and reliable. An example of an uncorrected and

corrected profile is shown in Figure 3.20. These tests were performed with different values cone area ratios, varying between 0,59 and 1,0, at first (right) they were spread over a wide range, once corrected (left), all profiles agreed very well.

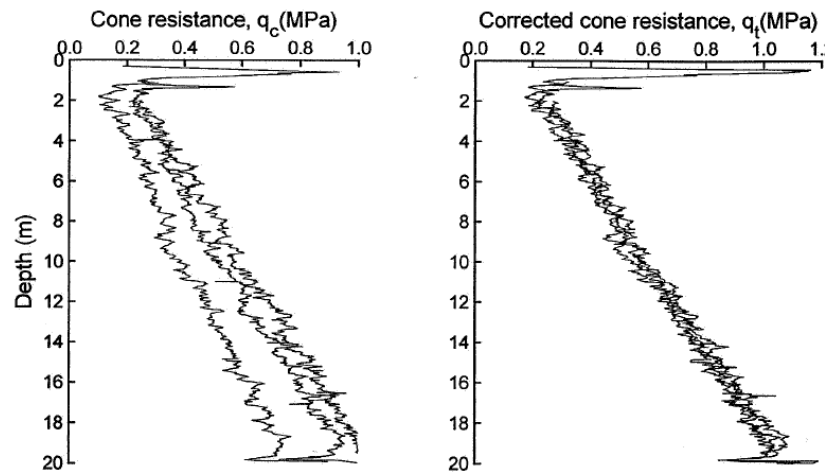


Figure 3.20: Example of the effect of correction q_c in soft soils (Bothkennar) by Lunne et al. (1997).

To mitigate the above, what can be done is to run trial tests, and come up with typical pore pressure profiles. The results will show how influential it is, and if it is worth it to correct the data or not. From the results a site-specific correlation could be obtained.

3.5.5 Discussion

Regarding the outlined limitations, it is clear that the cone penetration test and piezocone, are mainly empirical to some degree, in other words, soil parameters are obtain from tests and inspection, instead of theoretical frameworks.

Wroth (1988) (as cited by Meigh, 1989), stated that the basic requirements of any correlation between in situ tests and soil parameters, has to have the following characteristics:

- They should be stand on physical observation.
- Explainable through a theoretical framework.
- Be dimensionless.

So, what is means is that even though there are out there some correlations, engineers should always look for site-specific correlations. As the results of the penetrating tests have some physical meaning and can be explained under a theoretical framework, it is possible assess soil behaviour. For instance, a specific soil layer could be identified, such as London Clay or Chalk. Consistent results may be compared to some anomalous

results, helping to identify reworked zones or aid in the development of a 3D ground model. Whatever the application, practitioners should be aware of the limitations to avoid biased and ensure a high quality interpretation.

3.6 Main applications

The first application of the CPT/CPTu was as a profiling apparatus, and the addition of a piezometric sensing element allowed for the detection of thinner layers, moreover when combined, pore pressure and cone tip measurements, engineers are able to predict more accurately soil type, and its behaviour. (Houlsby, 1989)

To illustrate this, is common to get a rise in tip resistance and a decrease of pore pressure as we pass through a silty or sandy layer, because of its tendency to dilate (tip resistance goes up and pore pressure goes down). For instance, this finding is of paramount importance to assess the rate of settlement in layered soil. (Houlsby, 1989)

In the following paragraphs a critical review about some case studies is presented. These case studies are mainly related to the detection of weak layers, or shear zones. For instance, after a landslide, the reworked materials have low q_c values, moreover the registered excess pore pressures are very low Meigh (1987).

3.6.1 Case study 1: Landslide characterization, New Zealand

This case study was focus on the assessment of unstable soil layers in an existing landslide, located at Pyes Pa landslide in New Zealand. The geology of the site is made up of Pleistocene volcanic materials. The first 3,0 m consists of ash layers, overlying a paleosol layer from 3,0 m to 6,80 m depth, followed by a clayey soil from 7,0 m to 13,7 m depth, and the last layer from 13,7 m to 16,0 m depth was a coarse-grain ignimbrite. No ground water level was found.

3 different in situ tests were carried out: static, seismic and dissipation tests.

- From the static CPTu or piezocone: Soil sensitivity, SBT classification, and undrained shear strength were estimated.
- From the seismic (SCPTu): Liquefaction potential was assessed, and
- From the dissipation tests: The horizontal permeability was calculated.

The focus of this review was put on the use of static Piezocone for landslide characterization.

The landslide was classified as a shallow landslide, and is shown in Figure 3.21, along with the location of the 3 tests, which were performed 2,0 m away from each other.

The SBT charts from Robertson et al. (1986) were used, and soil sensitivity was estimated using Schmertmann (1978) formulation, as in the following equation:

$$S_t = N_s / R_f$$

Where:

N_s is a constant and R_f is friction ratio.

An N_s value of 7,5 was used to determine the sensitivity (S_t) profile with depth (See Figure 3.22). This sensitivity parameter according to Skempton and Northey (1952) (as cited by Jorat et al., 2014) is the relationship between the maximum undrained strength of intact clay to the maximum undrained strength of remolded clay. Skempton and Northey, recommended a grading system based on sensitivity as in Table 3.4.

Table 3.4
Classification of sensitive clays (Skempton and Northey, 1952)

| Sensitivity (S_t) | Classification |
|-----------------------|------------------|
| ~ 1 | Insensitive |
| 1 - 2 | Low sensitive |
| 2 - 4 | Medium sensitive |
| 4 - 8 | Sensitive |
| 8 - 16 | Extra sensitive |
| > 16 | Quick |

The results of the static Piezocone along with the interpretation made by Jorat et al. (2014) is shown in Figure 3.23.

Jorat et al. (2014) interpreted the changes in q_t values with depth as layers of different degrees of weathering, and the rise in pore water pressure as clay or silt minerals. The drop in pore water pressure at 13,7 m matched very well with a coarse-grained material identified at the same depth.

A potential slip plane was detected at a depth of 13,7 m, because of the high sensitivity values, low cone tip resistance, low sleeve friction and a sudden drop in pore water pressure.

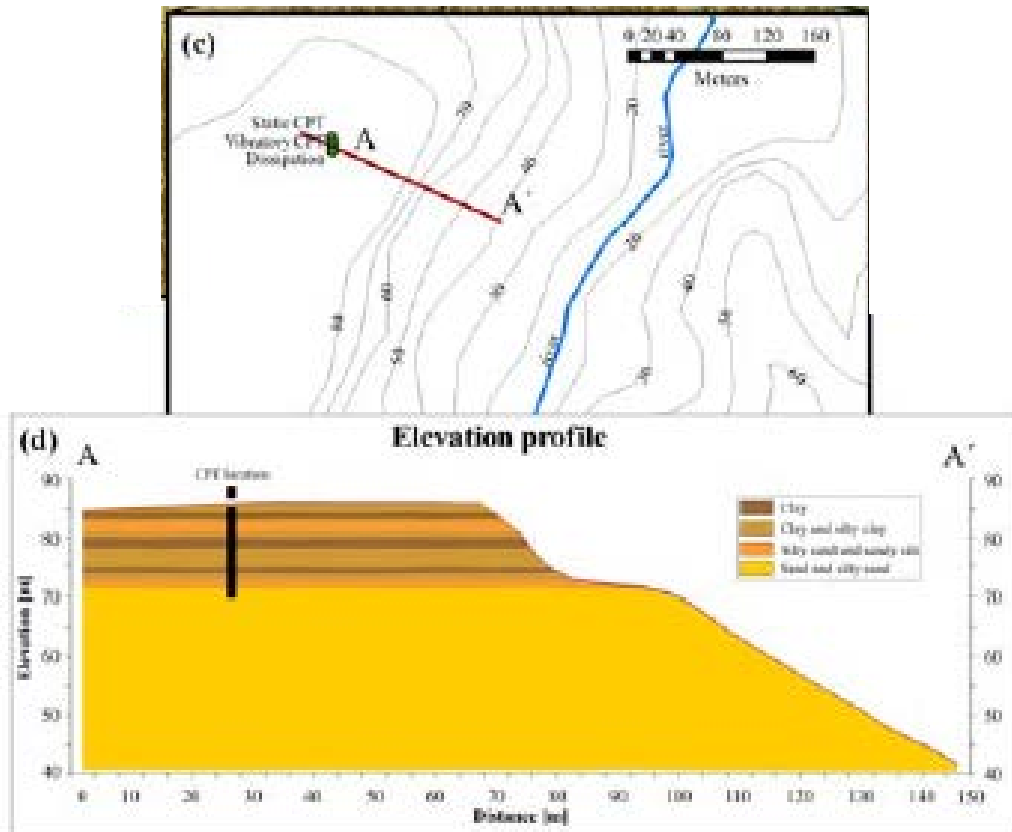


Figure 3.21: Landslide and CPTs location, and landslide cross section by Jorat et al. (2014)

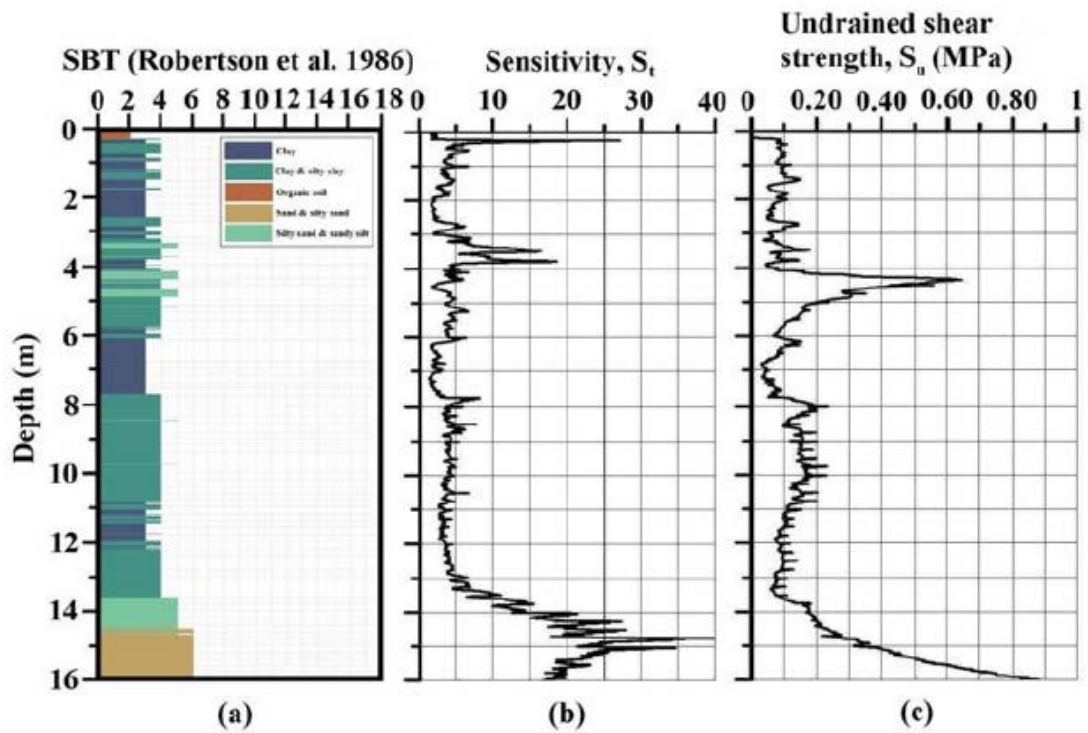


Figure 3.22: (a) Soil Behaviour Type, (b) sensitivity and (c) undrained strength by Jorat et al. (2014)

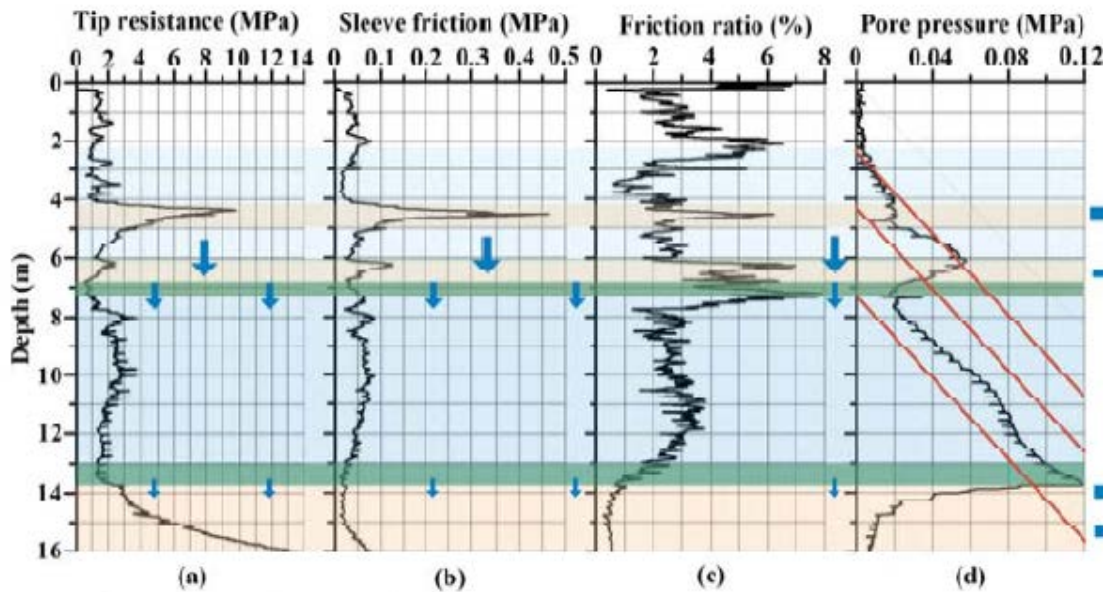


Figure 3.23: Static CPT result (a) tip resistance, (b) sleeve friction, (c) friction ratio and (d) pore water pressure by Jorat et al. (2014)

3.6.2 Case study 2: Detection of shear zones in a natural clay slope

This case study was focused on the assessment of weakened shear zones in clays, using CPT and continuous dynamic sampling (CDS).

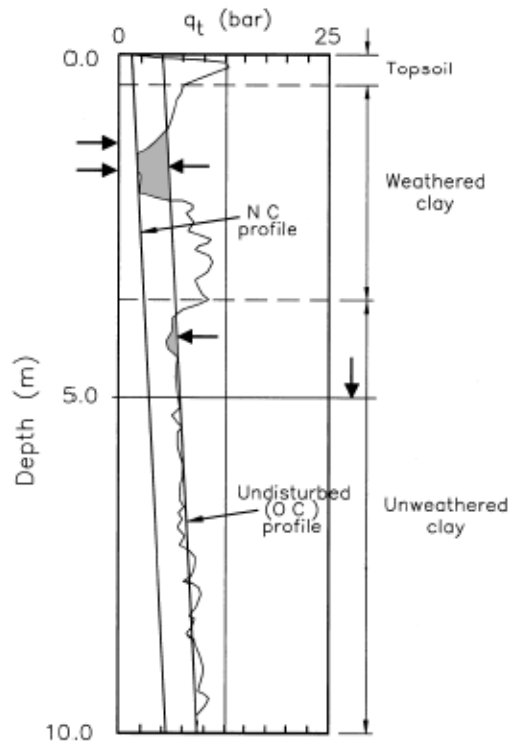
Mahmoud et al. (2000), highlighted the fact that even if common drilling and sampling techniques are used, the shear zones can be missed, and at the same time are expensive, because several samples are required. Instead Mahmoud et al. (2000) pointed out that over the years CPT has proven useful in providing a repeatable ground profile in terms of cone tip resistance q_c .

The geology at this site is made up of interbedded marine and glaciomarine soils, and it has a record of slope failures. Mahmoud et al. (2000) reported an instability study in this area, generated due to a road cutting at the toe of the slope, with movements of around 35 mm/day.

The CDS and CPT were used in conjunction. CDS allows for the direct assessment of retrieved samples (to a maximum depth of 8,5 m), some shear or weakened zones were identified as they were slickened samples (polished surfaces). These zones were located at the top of the weathered crust and unweathered layer, at about 3,0 m below the surface.

A typical CPT profile based on q_t is shown in Figure 3.24. In this profile, an overconsolidated surface crust is followed by a weathered clay layer overlying

unweathered overconsolidated clay. To identify shear zones, two boundaries were plotted, one corresponding to normally consolidated clays (NC) and the second or upper limit corresponding to an undisturbed strata (OC). The values of q_t , lower than the OC line were regarded as potential shear zones.



LEGEND

| | |
|--------------------------|---|
| NC profile | - Estimated q_t for normally consolidated clay |
| Undisturbed (OC) profile | - Estimated q_t for undisturbed clay |
| | - Zone of softened clay |
| | - Softened zone shown in Figure 6 |
| | - Observed shear surfaces at adjacent testhole CDS4 |
| | - Maximum depth of adjacent testhole CDS4 |

Figure 3.24: Identification of softened zones from CPT profiles by Mahmoud et al. (2000)

Mahmoud et al. (2000), concluded that CDS and CPT complement each other in the identification of shear zones, and has proven useful, however, even with those tools, the identification of shear zones is difficult, but they can be used successfully to assess the presence of weakened zones in active landslides of clay deposits.

4 METHODOLOGY

4.1 Proposed work flow to assess weakened zones

By considering the information concerning CPT and Piezocone tests, their limitations and applicability, as well as case studies, were described in detail in chapter 3. In response, a methodology workflow on how to identify weaker, slip layers, or anomalies, was prepared.

The final interpretation could be made base on several sources of information, which may in turn determine how accurate the interpretation could be. As a rule of thumb, the most basic and relevant information from which an interpretation could be made is summarized in Table 4.1.

Also depending on the available information, the final interpretation could be classified as low, medium or high (See Table 4.1).

Table 4.1
Interpretation quality

| Information | Description |
|--|---|
| CPTu raw profiles (q_t , f_s , u_2), boreholes, and geology of the site. | The interpretation could be of high quality and accurate. |
| CPTu raw profiles (q_c , f_s , u_2), and geology of the site. | The lack of boreholes induces additional uncertainties. |
| CPT (q_c and f_s), and geology of the site. | The lack of boreholes and pore pressure measurements, may reduce the accuracy of the CPT tests, resulting in a rough estimation of ground strength, especially in fine-grained soils. |
| <p>Notes:</p> <p>When possible, it is recommended to use piezocone tests, along with boreholes and geological background. The pore pressure measurements increase the resolution for instance:</p> <ul style="list-style-type: none"> • CPT resolves a sand layer or a clay layer, 100 mm and 150 mm thick, respectively. (Jardine, 2017). • Piezocone improves the resolution and the thinnest recognizable layer is 30 to 50 mm thick. (Meigh, 1987). | |

Once we know which data is available, the process to identify reworked materials, weak layers, or potential slip layers could be as follows:

- **Step 1:** Prepare a plan view of the existing boreholes, CPTs, and Piezocone tests. (See Figure 4.1).
- **Step 2:** Draw cross sections, through different CPTs and Piezocone tests within the area of study, in this ideal example, the sections were labelled as section 1 and section 2, as shown in Figure 4.1.

- **Step 3:** Prepare a simplified ground profile from any existing borehole, and next to it, place a graph with the superimpose CPT or piezocone profiles. The graphs could be in terms of q_t/q_c , f_s , R_f (%), and pore pressure (u_1 , u_2 , or u_3). It is recommendable to plot the data corresponding to the CPT or piezocone tests, which lies within the same section. An example of this is shown in Figure 4.2, where: (a) Borehole and soil stratigraphy, (b) q_t profiles, (c) f_s profiles, (d) R_f profiles, (e) pore water pressure profiles, and a dashed line representing a reference limit.
- **Step 4:** If a good agreement between the borehole (geology, stratigraphy, etc.), CPT and Piezocone profiles is found, then a “signature” or common pattern of the site can be defined, so based on this a baseline could be established.
- **Step 5:** Determine the reference limit for each q_t/q_c , f_s , pore pressure profile with depth. The reference limit should be defined, as the lowest q_t/q_c , f_s and pore pressure values within each stratum.
- **Step 6:** Superimpose the reference limit and the CPT or Piezocone profile which would be analyzed. Shear zones will be defined at depths where q_t/q_c , f_s , and pore pressure values are much lower than their corresponding reference limit. (See Figure 4.3), where: (a) Soil stratigraphy and potential shear zones, (b) q_t profiles with identified shear zones, (c) f_s profiles with identified shear zones, (d) very low pore pressures located at potential shear zones.

The procedure described here is a recommended workflow based on the literature review.

It is important to point out that the reference limits are determined considering that the geology, soil stratigraphy or soil properties, cannot change drastically in a small area. If that happens, it should be considered an indication of reworked zones or periglacial features.

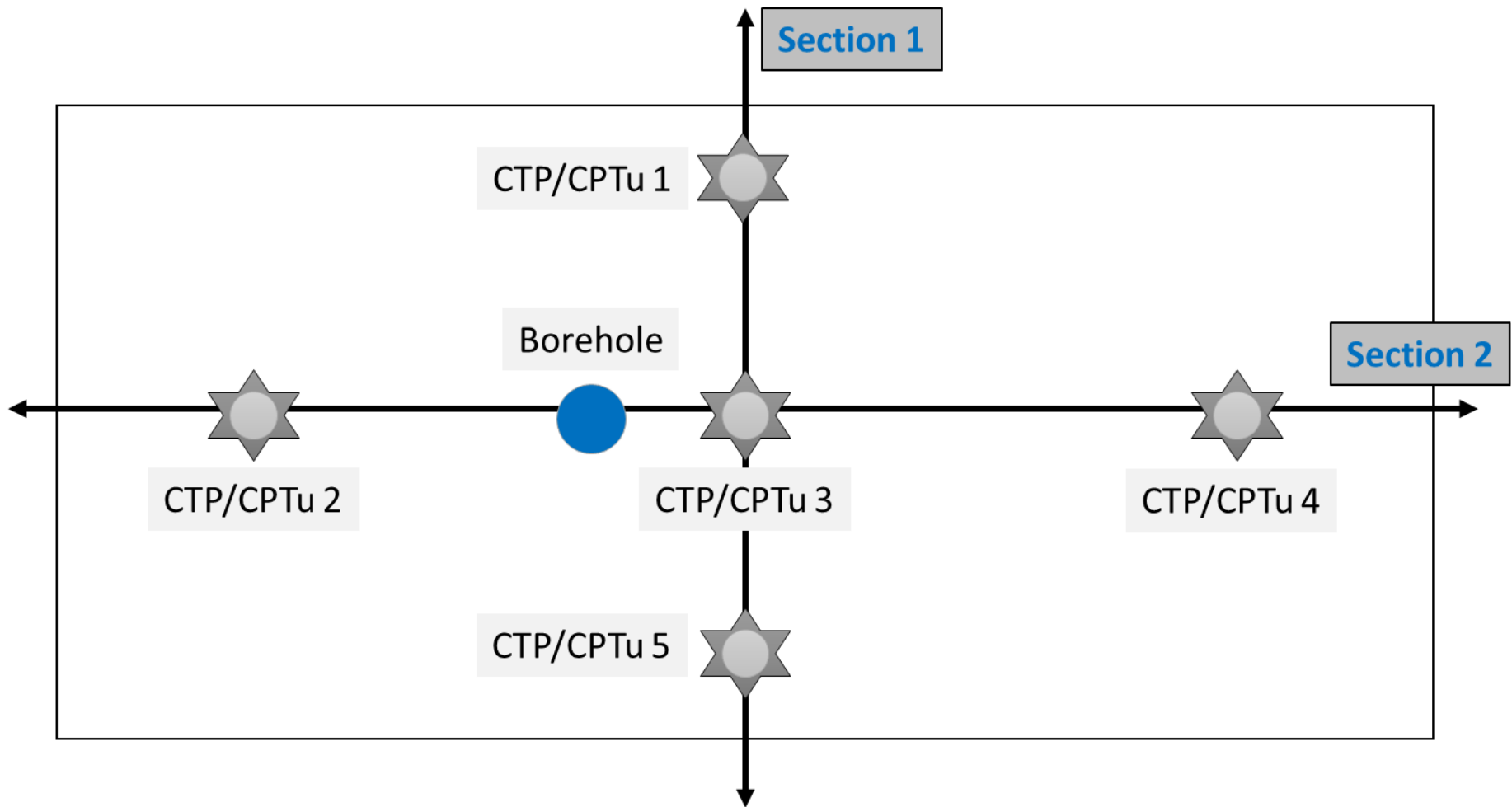


Figure 4.1: Plan view of a site investigation.

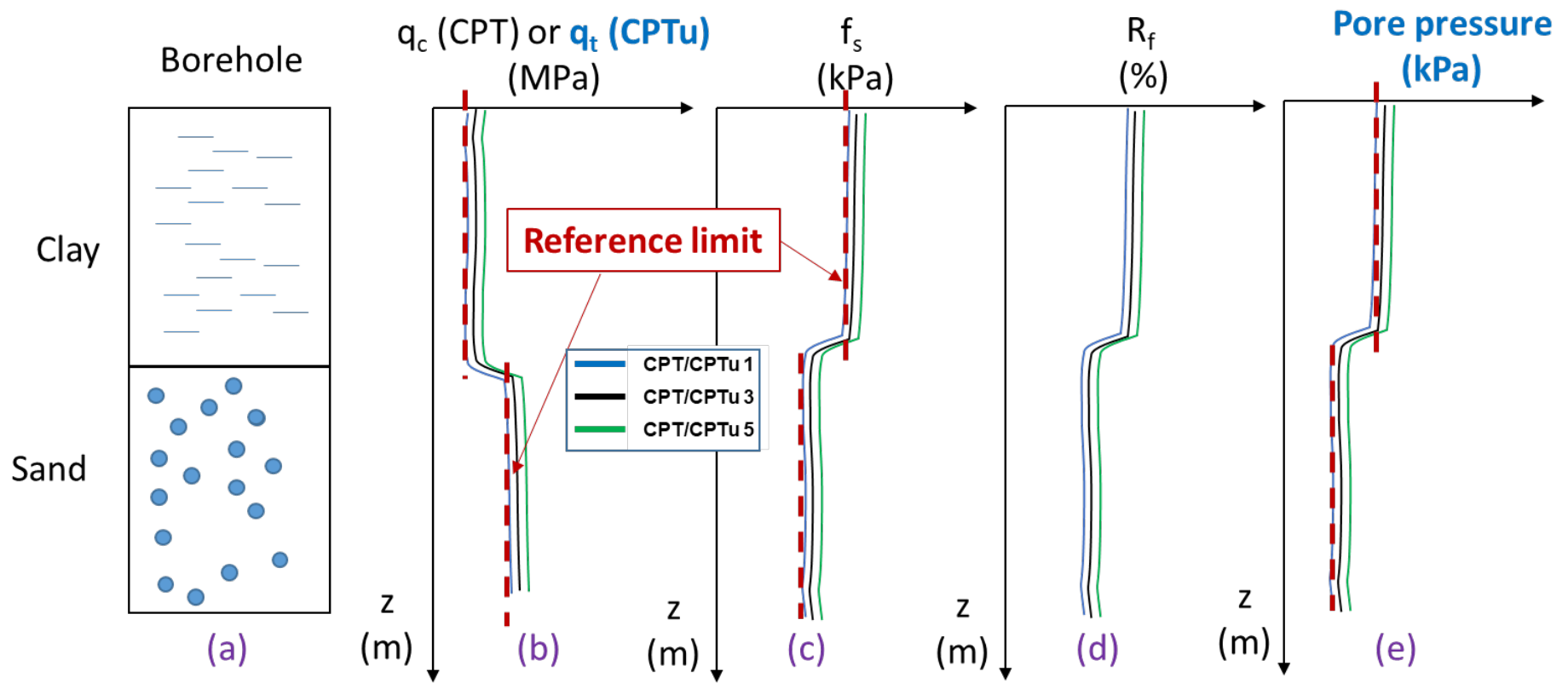


Figure 4.2: Borehole and all superimposed CPT/CPTu profiles from Section 1.

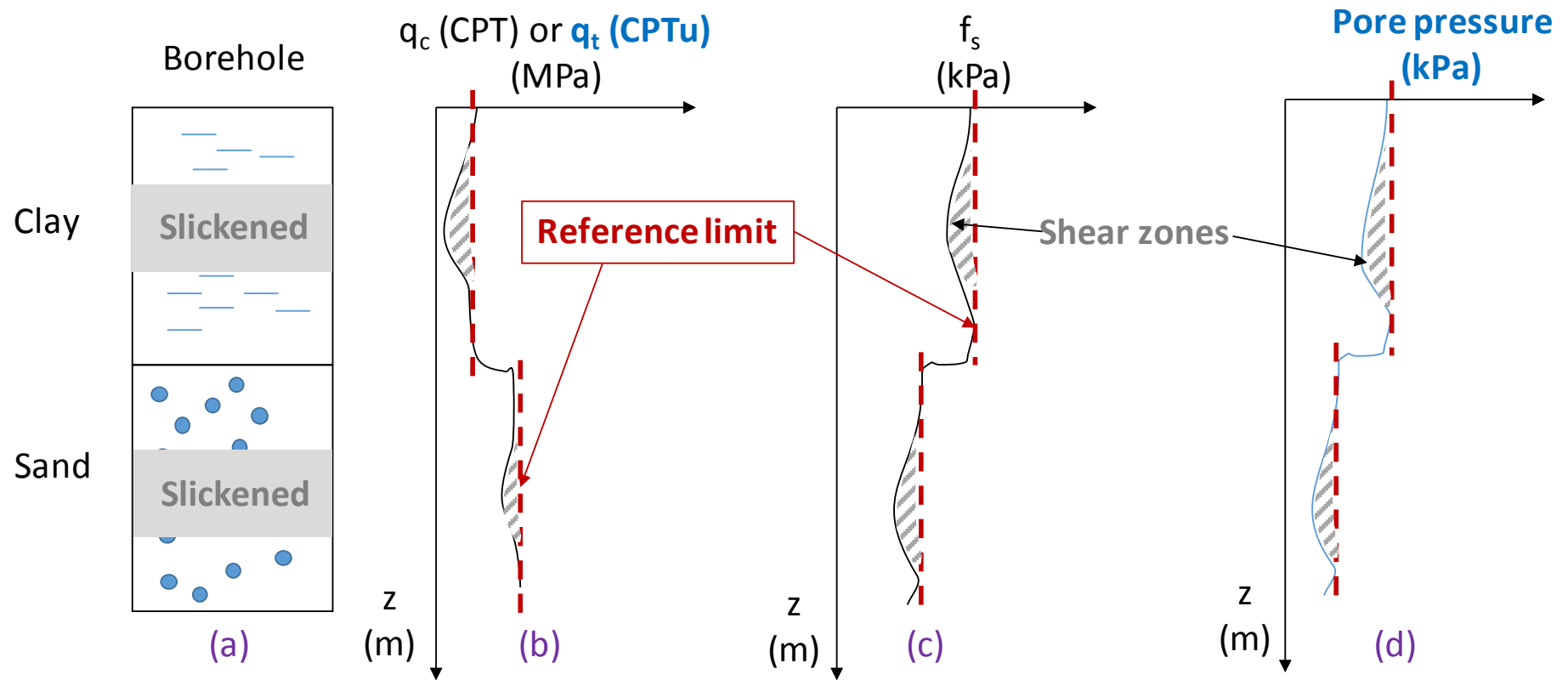


Figure 4.3: Identification of shear zones

5 RESULTS AND INTERPRETATION

5.1 Introduction

In this chapter, two areas were analyzed, St. James' Square and the Heart of East Greenwich (former Greenwich District Hospital).

- At St. James' Square, 06 piezocone tests were performed in London Clay.
- At the former Greenwich District Hospital, 56 CPTs and 9 piezocones were performed in Chalk.

In both sites the data was processed and interpreted in order to identify shear zones, establish reference limits and discuss about the repeatability and accuracy of the executed tests.

5.2 Software packages

At St. James' Square, the raw data from the piezocones was available, and the information was processed using Move and CPeT. At the former Greenwich District Hospital only the soft copy of the report was available, so the data was digitalized, however, this data cannot be used directly in Move or CPeT, as the q_c and f_s , should be at the same interval as the rate of penetration, in order to derived soil parameters, instead spreadsheets to plot the CPT data and cross sections across the site were used.

5.2.1 CPeT-IT®

This software helps to process and interpret CPT data. Is a result of a partnership between leading companies, under the supervision of Professor Peter Robertson. It allows the interpretation of CPT data in terms of Soil Behaviour Type (SBT). Soil parameters are derived from current correlations, recommended extensively by Lunne et al. (1997), in addition to some updates by Professor Robertson.

The procedure on how the CPeT-IT® software was used is detailed as follows:

- The raw CPT data was prepared in a compatible format, and imported using the **"Import CPT File"** bottom in the **"CPT data"** tab.
- Once the raw data is imported, it is necessary to make few changes to the "calculation properties" set by default in the **CPT data tab > CPT properties**.

Some values are ground elevation, probe radius, ground water table (GWT), cone area ratio, etc.

- After changing the calculation properties, the raw plots, basic plots, normalized plots and estimated parameters are generated.
- As an example the processed and interpret CPT data in terms of SBT index and SBT is shown in Figure 5.1.
- The processed data was then used in Move®.

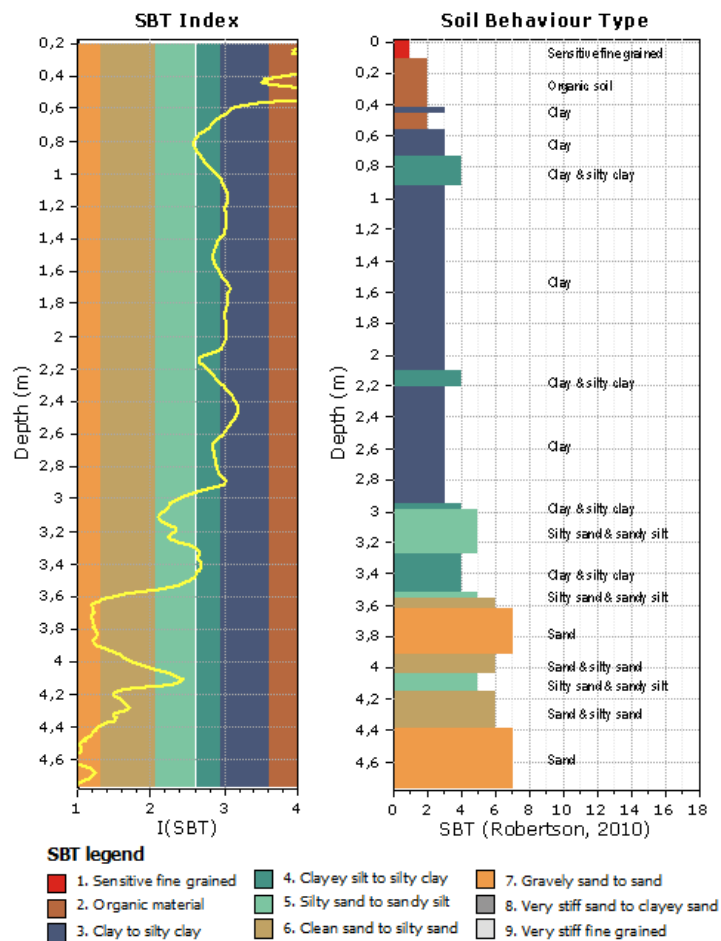


Figure 5.1: SBT index and SBT profile obtained in CPeT-IT® software.

5.2.2 Move ®

Move® is an analysis tool, with aids in the integration of data to build 3D ground models, as well as cross sections and advance analysis related to structural geology such as fracture modelling, fault and stress analysis.

The CPT profiles were added to the program as follows:

- Define coordinates, elevation and processed CPT data (q_t , f_s , u_2) in a compatible format.
- Assign CPT data to each marker top and well track, where each marker defines the geological top of each horizon.
- To display the CPT in terms of a specific parameter (q_t , f_s , u_2), it was necessary to use the “**Colour Map**” tool, located at the “**data analysis panel**”. Once the CPT borehole is selected, go to **Colour Map>Attributes>Vertex** and choose between q_t , f_s , or u_2 .
- To display different colours within **3D View Interpolation Mode>Vertex Colour Interpolation**, activate “**Display Areas With No Attribute Value**”, and choose `red_white_blue`. (See Figure 5.2).

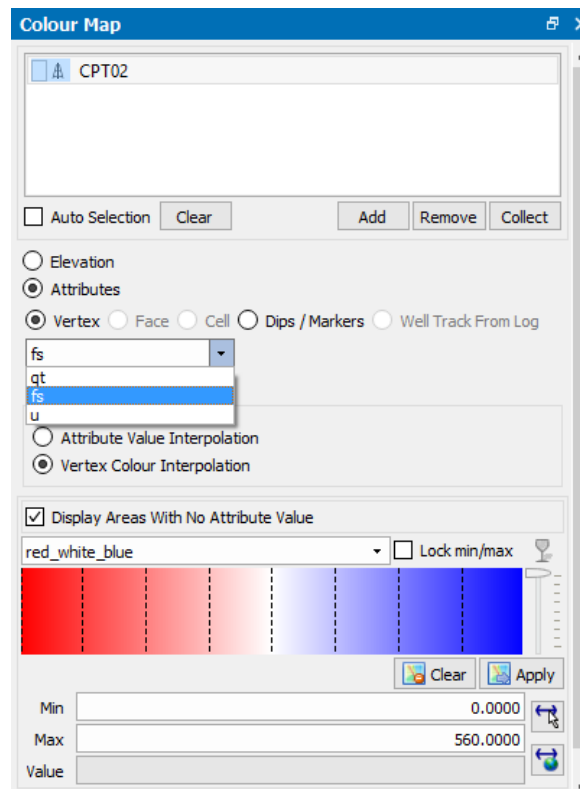


Figure 5.2: Colour Map window.

- To display different colours within **3D View Interpolation Mode>Vertex Colour**.
- With the desired values displayed as a function of different colours, an interpretation of the strength variability across the site was performed.

5.3 St. James' Square

The 6 piezocone tests performed at this site were analyzed according to the recommended procedure in chapter 4. To aid in the interpretation, the piezocone tests were included in a 3D ground model of the upper layer of London Clay prepared by Dixon (2015), as shown in Figure 5.3. The key findings are described in the following paragraphs.

5.3.1 Interpretation of Piezocone data

For reference purposes, the top of London Clay identified in each piezocone by Dixon (2015), was summarized in the following table:

Table 5.1
Depth of London Clay

| Piezocone | Depth of London Clay (m) | (*)SBT classification by Robertson et al. (1986) |
|-----------|--------------------------|--|
| CPT01 | 2,78 | Stiff CLAY (3) |
| CPT02 | 2,73 | Loose sandy SILT to clayey SILT (6) |
| CPT03 | 3,33 | Stiff CLAY (3) |
| CPT04A | 3,24 | Stiff silty CLAY to CLAY (4) |
| CPT05 | 2,05 | Medium dense sandy SILT to clayey SILT (6) |
| CPTS6 | 0,5 | Stiff CLAY (3) |

Notes:

(*)SBT: Soil Behaviour Type.

Also, 2 sections were drawn across the site. The piezocone tests belonging to each section were listed in Table 5.2.

Table 5.2
Sections and their corresponding piezocone tests - St. James' Square

| Section | piezocone tests |
|---------|---|
| 1 | <ul style="list-style-type: none"> CPT01, CPT02, CPT03, and CPT04. |
| 2 | <ul style="list-style-type: none"> CPT04, CPT05, and CPT30. |

The soil layering was obtained from an existing borehole, and further interpretation was carried out based on a detailed inspection of q_t , f_s , R_f (%), and u_2 (pore pressure) profiles.

In addition to section 1 and section 2, a 3D interpretation of the data, in conjunction with the available 3D ground model, identified faults and periglacial features, was carried out.

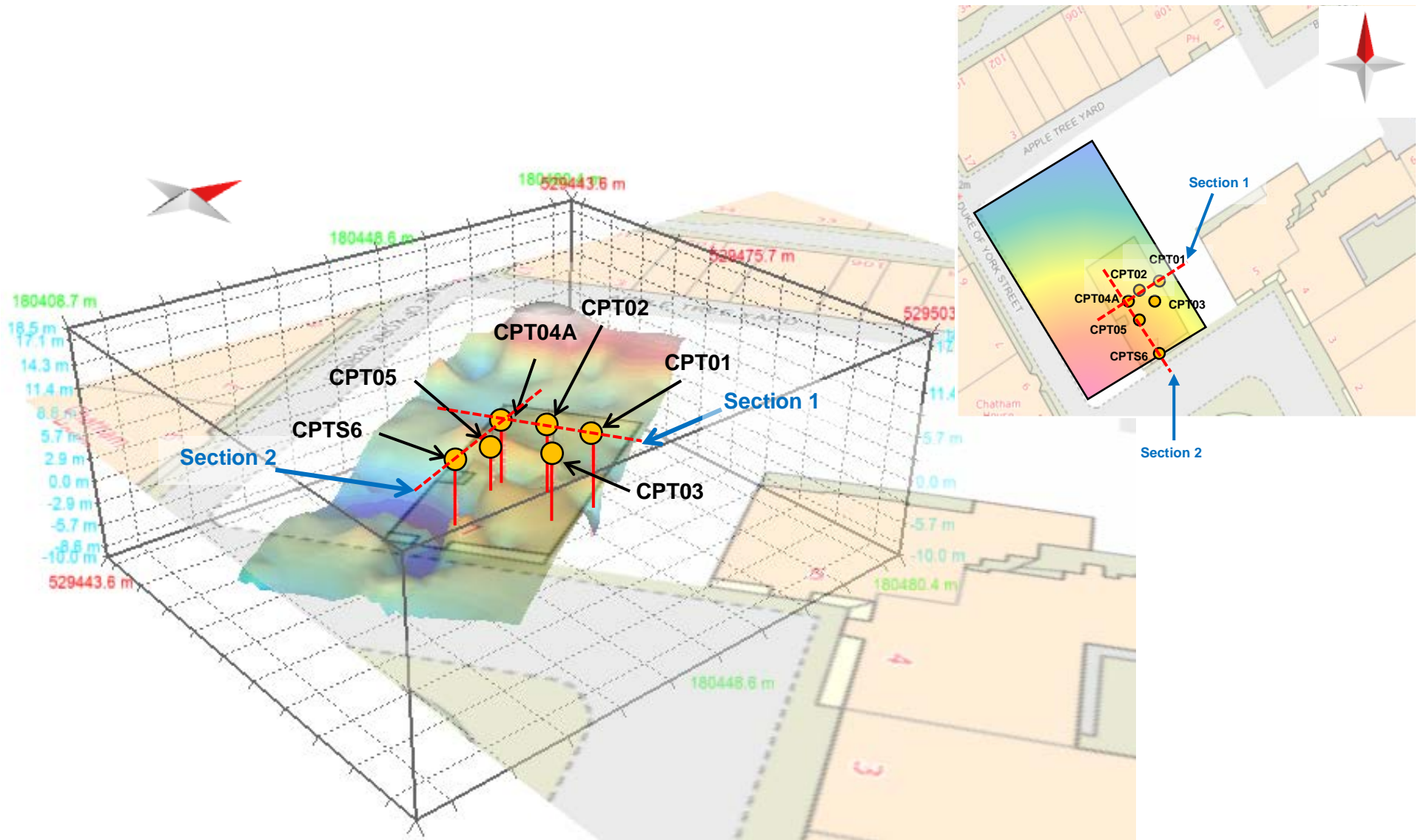


Figure 5.3: Piezocone tests and 3D ground model (Dixon, 2015) at St. James' Square.

5.3.1.1 Section 1

In Figure 5.4 and Figure 5.5, the top of London Clay reported by Dixon (2015) was indicated in each profile. These figures show the superimposed piezocone tests, belonging to section 1.

Finding a signature or pattern

There is a consistent layer of London Clay identified from the piezocone profiles, which agreed well with the depth of London Clay reported by Dixon (2015). This layer is on average at 3,0 m below the surface, and showed an increase in strength with depth.

In the identified layer of London Clay, q_c varies between 1,5 MPa and 4,0 MPa, with no abrupt changes. This pattern implies that this is potential natural, unworked or unsheared material. The relatively narrow range of q_t values with depth is even evident in the normalized plots (See Figure 5.5). The average of all piezocone profiles of section 1 is represented by the red line in Figure 5.4 and Figure 5.5.

Other relevant feature is the variation of fines content in London clay, which is captured by the changes in the measured pore pressures. For instance, one way to relate the piezocone data to soil type is by using the SBT index, as shown in Figure 5.6, where it is possible to differentiate between clay or and thin layers of clay and silty clay.

Last but not least, when pore pressure measurements are available, is possible to use the modified SBTn index proposed by Robertson (2016). The SBTn classify soils in terms of their dilative or contractive behaviour. This is of paramount importance because a reworked zone or sheared material may be contractive. In Figure 5.7, the London Clay is classified as “Clay-like-dilative” which could be interpreted as a uniform and sound strata, with no potential shear zones or contractive layers in between.

Comments about the inferred stratigraphy

In Figure 5.4 enclosed by the blue box, the overlying materials (Made ground, Langley silt, and River terrace deposits) showed a broad range of q_t , R_f (%) and u_2 (pore pressure) values. Their behaviour is completely different to that of London Clay, which helps to determine a boundary between these overlying layers and London Clay. On the other hand, London Clay is more coherent and well defined along the profiles.

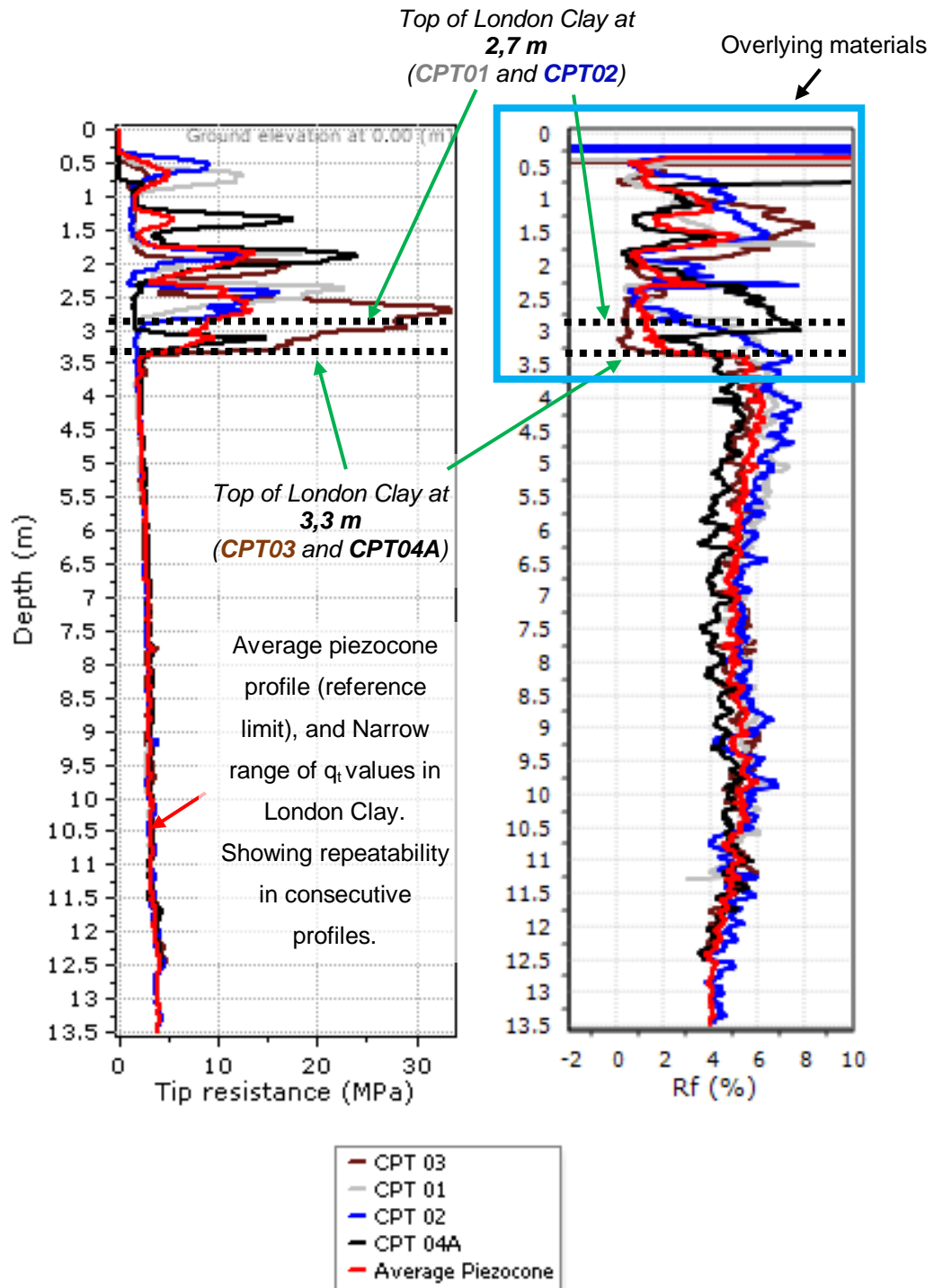


Figure 5.4: Superimposed profiles in terms of basic q_t , and R_f (%) parameters.

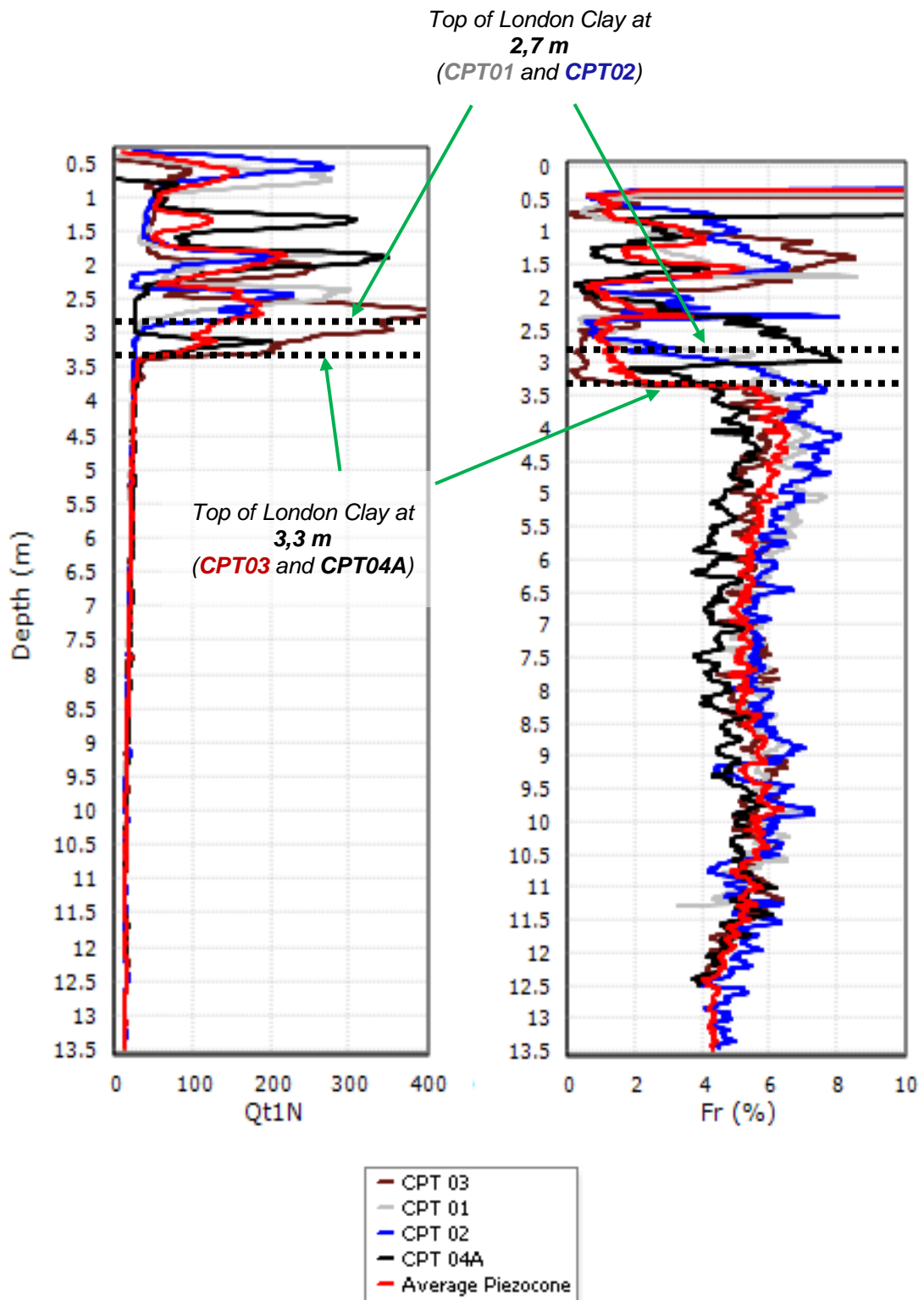


Figure 5.5: Superimposed profiles in terms of normalized Q_{t1N} , and F_r (%) parameters.

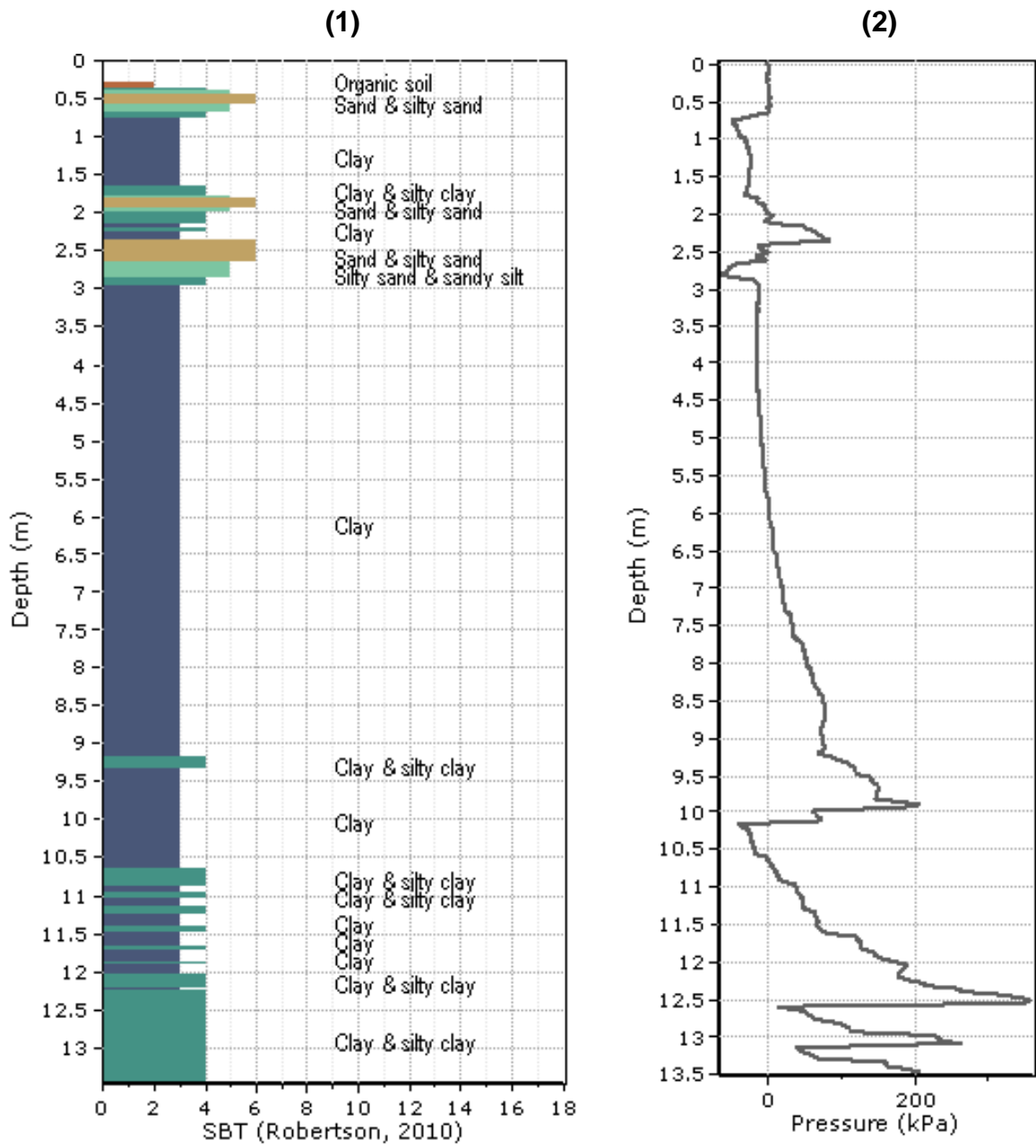


Figure 5.6: Soil behaviour type classification: (1) SBT, (2) pore pressure - CPT02.

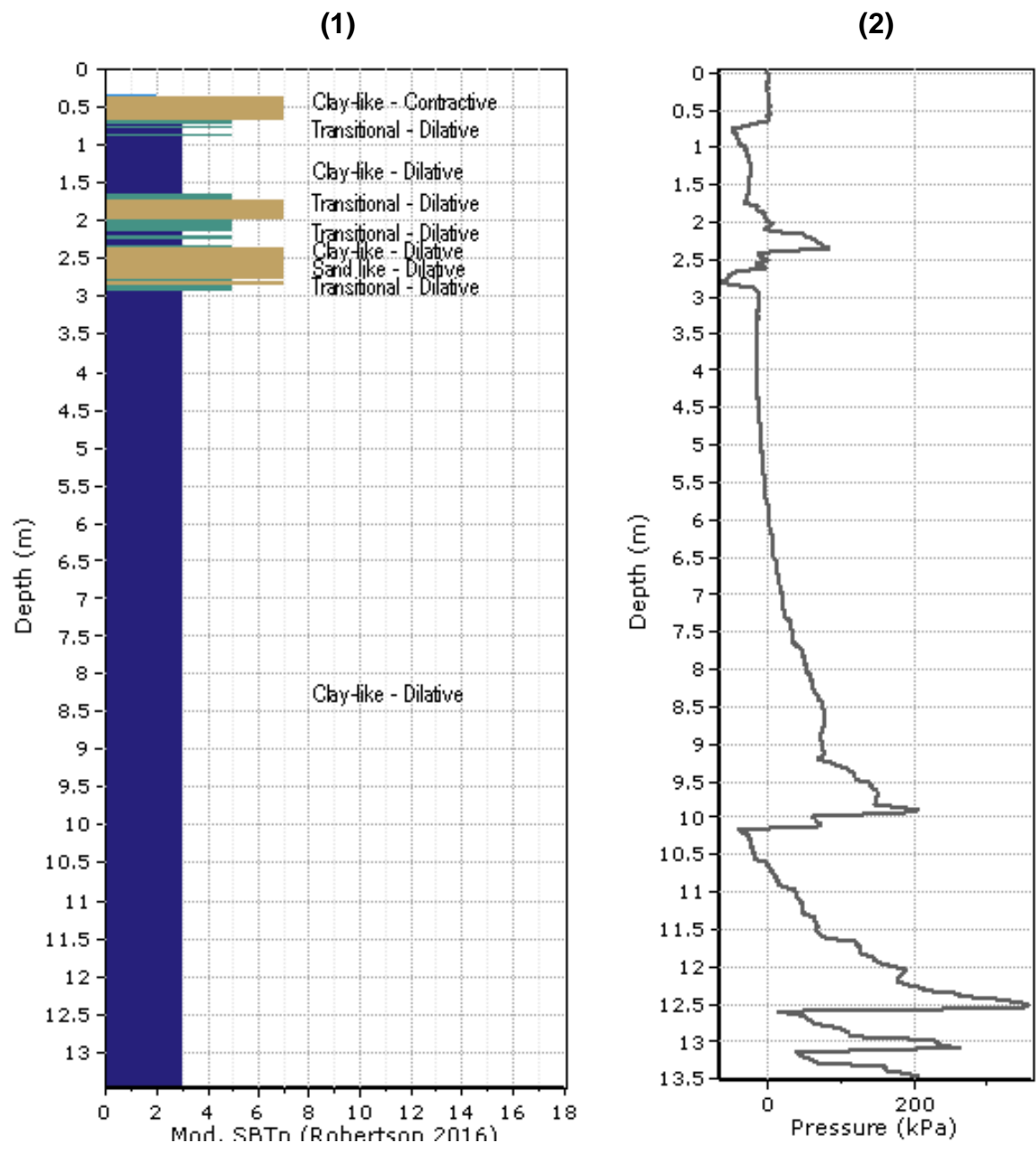


Figure 5.7: Normalized soil behaviour type classification: (1) SBTn, (2) pore pressure - CPT02.

Strength variability

London Clay showed similar q_t values between the analyzed piezocone tests (See Figure 5.4 and Figure 5.5). Overall the q_t values increased at a rate of 0,25 MPa/m. It is worth it to mention that CPT01, CPT02, CPT03, and CPT04A were executed close to each other, with the maximum distance being 10,0 m, measured between CPT01 and CPT04A. So, the identified similarities between these profiles in terms of their registered q_t values were expected.

The f_s and u_2 (pore pressure) profiles were superimposed in Figure 5.8, in order to confirm the limited spatial variability. In this figure enclosed by a red box, the f_s values have a narrow range, which confirms the uniformity in strength across the site, at least over the area covered by the analyzed piezocone tests. One characteristic of this figure is the superimposed pore pressure values. These have a broad range of u_2 , and this could be explained by the presence of closed fissures inhibiting flow or a decrease or increase of fines content.

Reference limit

The superimposed q_t , f_s and R_f (%) profiles were quite uniform and of limited variability, for this reason in this research the reference limit for q_t or f_s , could be represented by any of the analyzed piezocone tests or an average piezocone profile.

Identification of shear zones

No evidence of shear zones was found, apart from the broad ranges in pore pressures that may be associated to closed fissures or changes in fines content in London Clay.

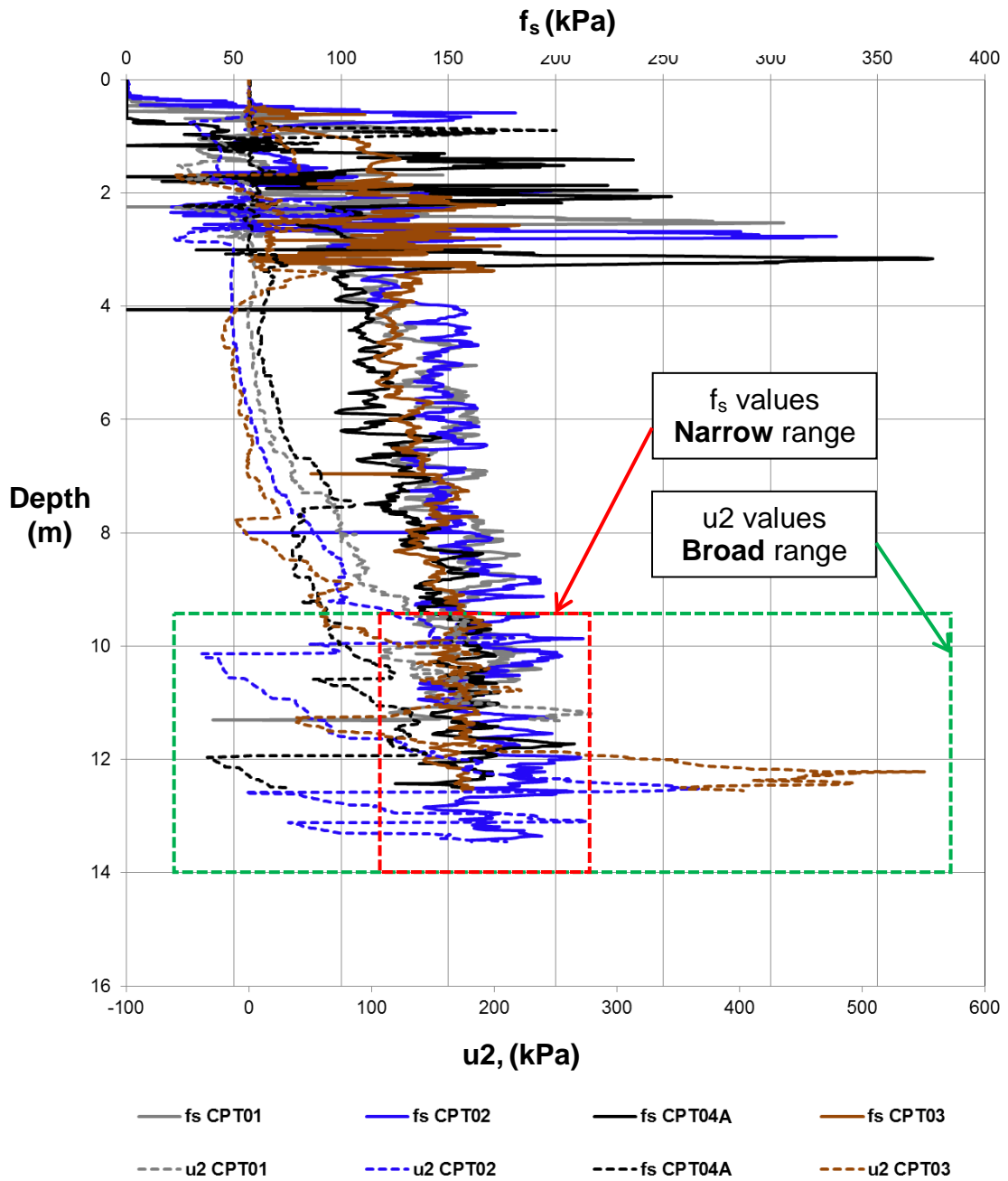


Figure 5.8: Superimposed f_s and u_2 profiles at St James's Square - Section 1.

5.3.1.2 Section 2

In Figure 5.9 and Figure 5.10, the top of London Clay corresponding to CPT04A was indicated. These figures show the superimposed piezocone tests, in terms of q_t , R_f (%), u_2 , and SBT index, belonging to section 2.

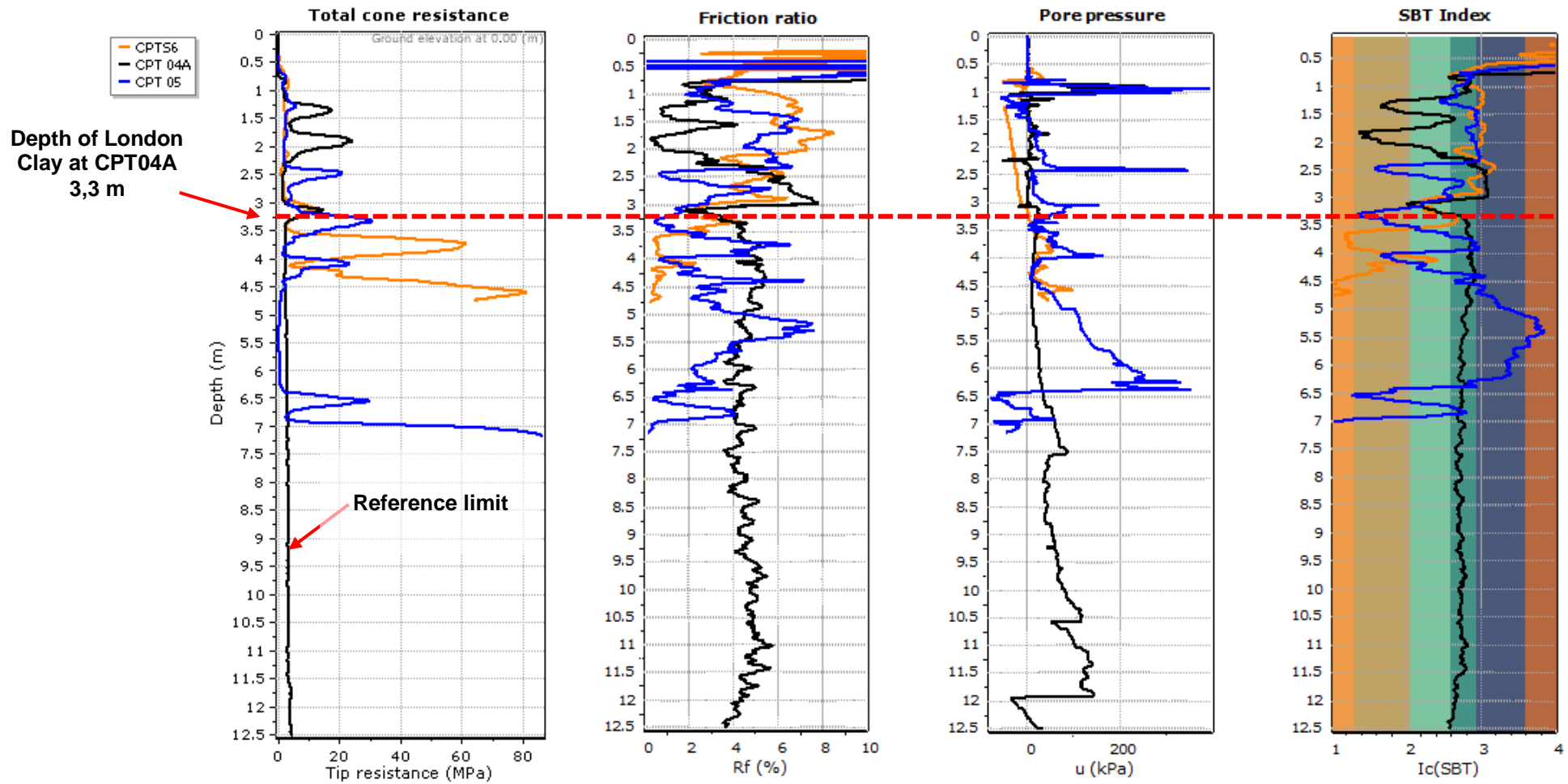


Figure 5.9: Basic plots of section 2, q_t , R_f (%), u_2 (pore pressure) and SBT index.

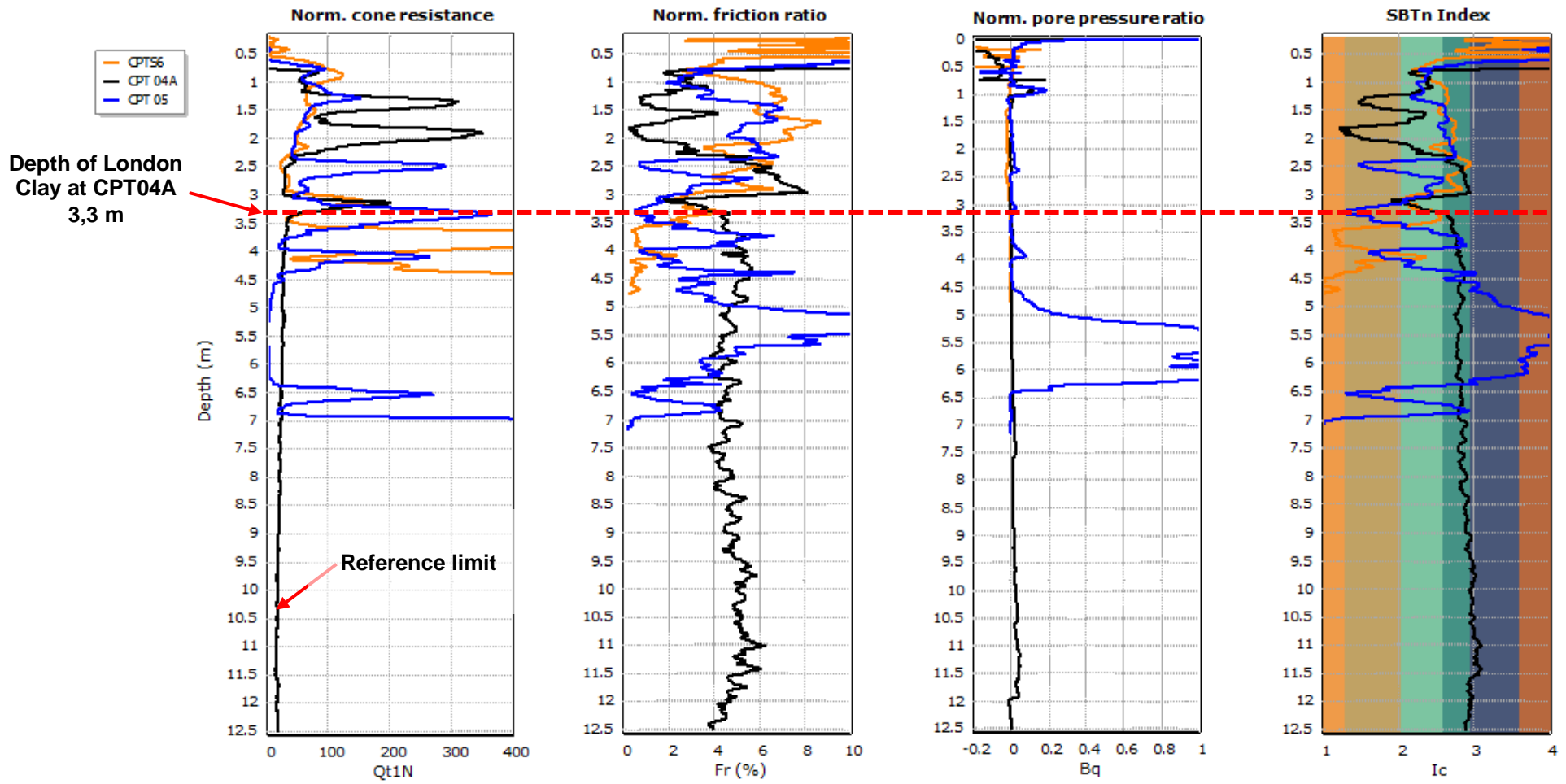


Figure 5.10: Basic normalized plots of section 2, q_t , R_f (%), u_2 (pore pressure) and SBT index.

Finding a signature or pattern

The q_t and R_f profiles of CPTS6 and CPT05 did not follow a evident pattern, compare to CPT04A. CPTS6 and CPT05 are located at 14,0 m and 5,0 m distance from CPT04A, which is a very short distance for the abrupt change in behaviour within London Clay, so no signature or pattern was defined from CPTS6 and CPT05.

Comments about the inferred stratigraphy

As in section 1, the overlying materials (Made ground, Langley silt, and River terrace deposits) showed a broad range of q_t , R_f (%) and u_2 (pore pressure) values. However, the boundary between the overlying layers and London Clay was obvious only in CPT04A.

Strength variability

The visual interpretation of the piezocone tests in section 2, within London Clay, showed very variable q_t values between the analyzed piezocone tests (See Figure 5.4 and Figure 5.5), also in CPTS6 very low values of q_t were recorded at a depth of 4,3 m compare to CPT04A, as shown in Figure 5.11.

Most importantly, the pattern of London Clay found in section 1 was a key feature to understand the strength variability in section 2. For instance in section 2, it was known in advance how London Clay behaves at this site. That is why CPT05 and CPTS6 were subjected to a further analysis, as these tests showed a different behaviour in just a short distance.

Reference limit

The q_t , u_2 , f_s and R_f (%) profiles of CPT04A were used as a reference limit, as it was the only piezocone test with a well-defined layer of London Clay, and because it was previously compared to other tests that showed the same pattern in section 1.

Identification of shear zones

In Figure 5.11 at a depth between 4,6 m and 6,3 m, enclosed by a red box, a potential shear zone was identified in CPT05. The f_s values and q_t values were lower than the reference limit set by CPT04A. The low f_s and q_t values, and the high pore pressures, were enough to conclude that this was indeed a potential shear zone at which a reworked or sheared material was located.

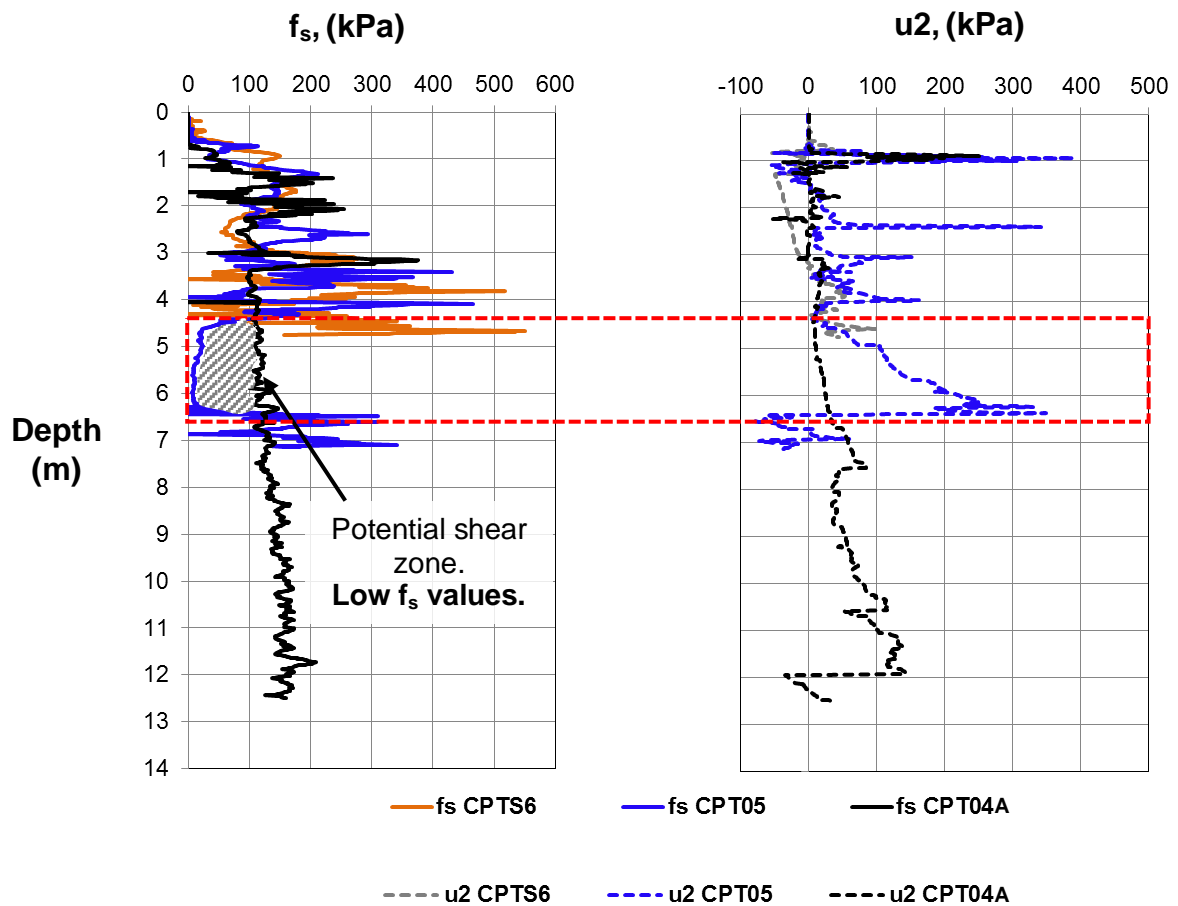


Figure 5.11: Superimposed f_s , and u_2 profiles - Section 2.

As it was pointed out in section 1, the use of the SBTn index, gives some insights into soil behaviour. So, the piezocone test CPT05 was classified according to SBTn index, as shown in Figure 5.12. In this figure, at the same depth, where the shear zone was identified, London Clay has a contractive behaviour. The latter, demonstrate that a contractive layer within London Clay is an evidence of reworked or sheared material.

Finally, the identified shear zone was interpreted to be the position of the fault. For this reason, the location of a strike-slip fault originally defined by Dixon (2015) was moved by about 5,0 m in a parallel direction, as shown in Figure 5.13, reflecting the new interpretation of the fault position. The proposed fault plane pass in between CPTS6 and CPT05, as precise extent of faulting is unknown, and more data is required.

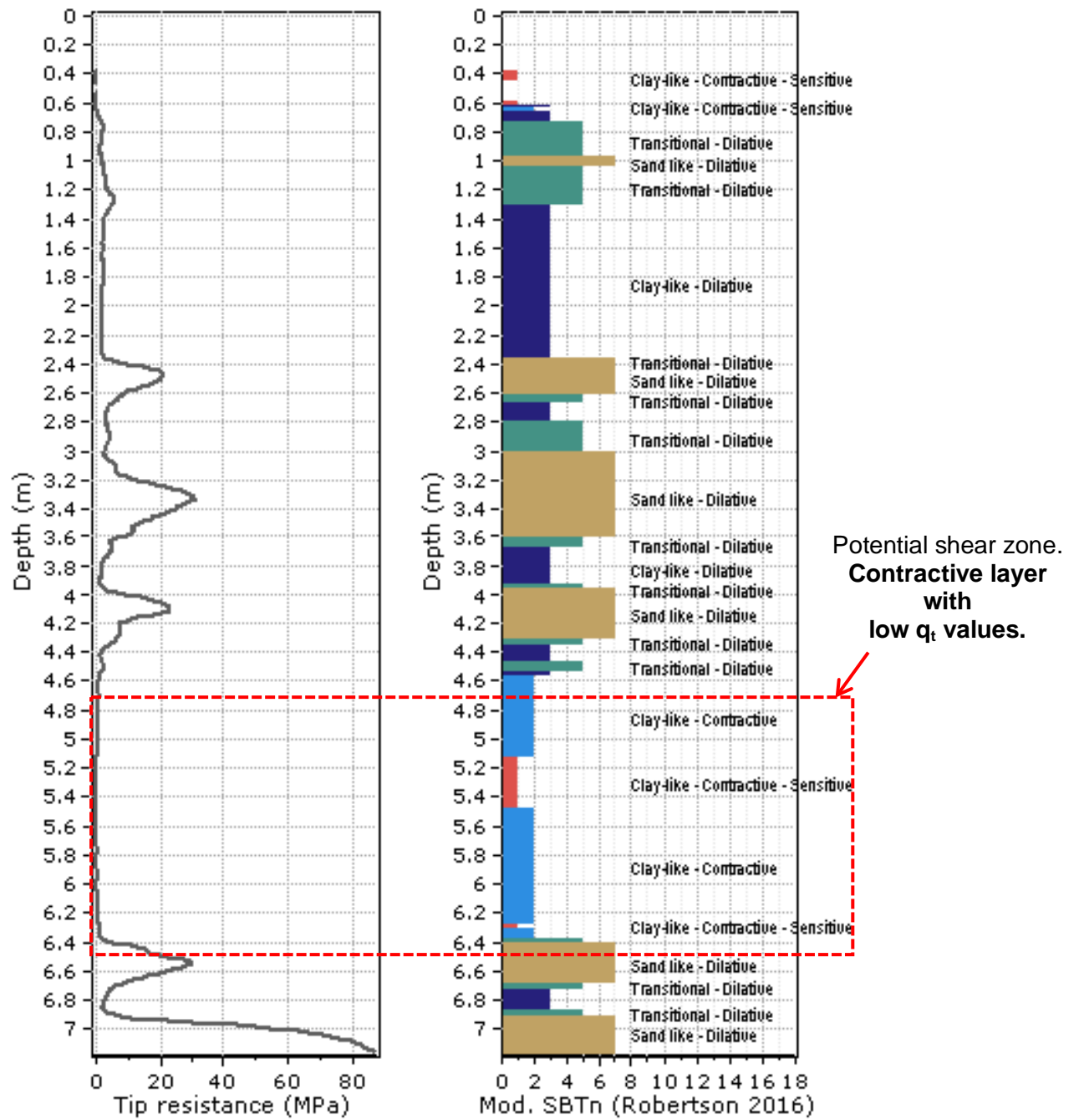


Figure 5.12: Cone tip resistance (q_t), and SBTn profiles - CPT05.

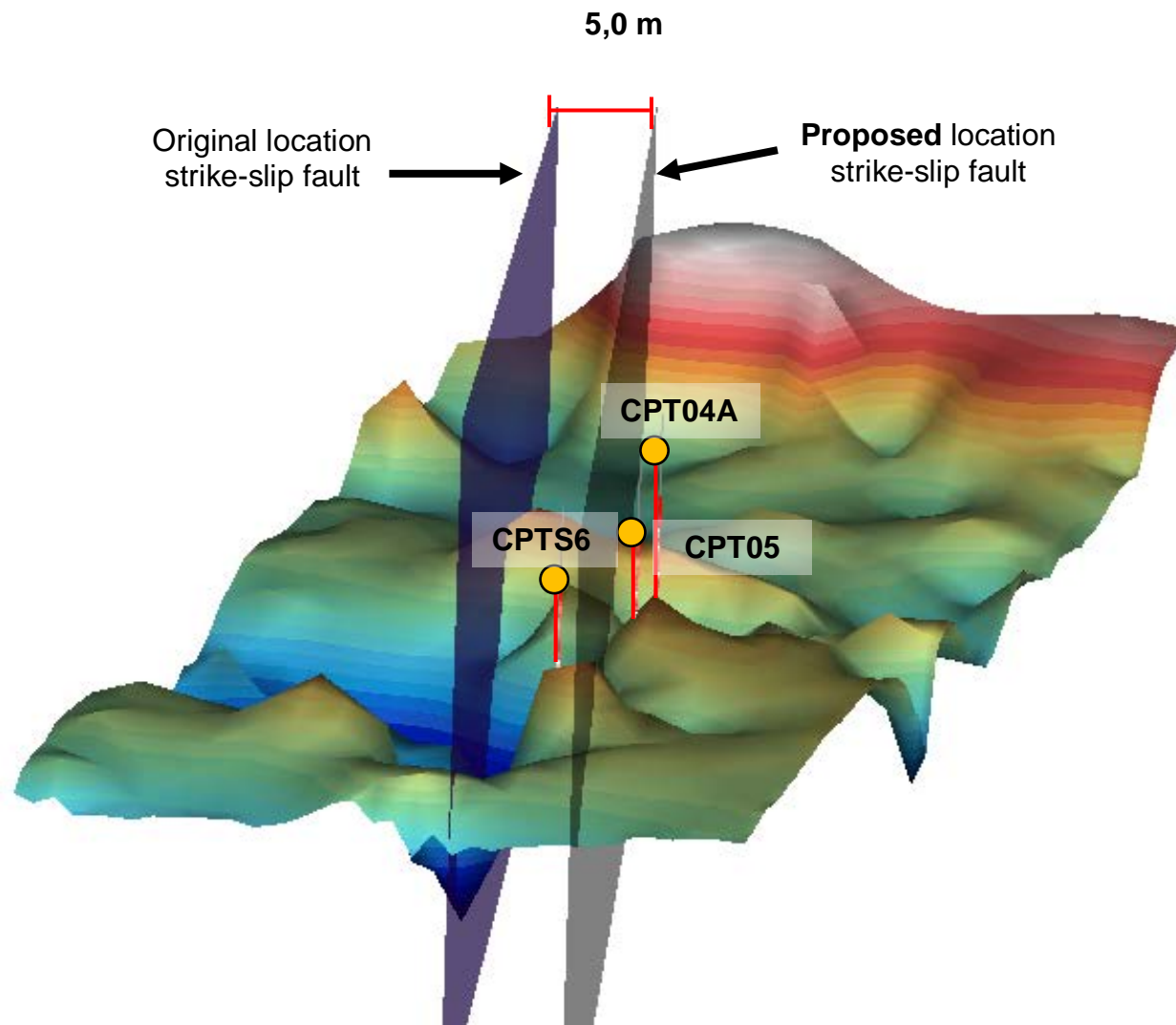


Figure 5.13: Proposed location of a previously identified strike-slip fault.

5.3.2 Results

The main findings of each section were summarized in Table 5.3.

Table 5.3
Main findings

| Section | Findings |
|---------|--|
| 1 | <p>Interpretation of ground profile</p> <ul style="list-style-type: none"> Overlying materials, showed a broad range of q_t, R_f (%) and u_2 (pore pressure) values. Below these materials, London Clay has a narrow range of q_t values, and was easy to identify. <p>Signature or pattern</p> <ul style="list-style-type: none"> A consistent layer of London Clay was identified from the piezocone profiles, also they agreed well with the depth of London Clay reported by Dixon (2015), and it is located at a depth of 3,0 m on average. According to the SBTn index, an intact layer of London Clay at this site has a dilative behaviour. <p>Strength variability</p> <ul style="list-style-type: none"> A limited spatial variability was identified from the superimposed q_t, R_f (%) and u_2 (pore pressure) profiles. This was expected as the piezocone tests were executed very close to each other. <p>Shear zones</p> <ul style="list-style-type: none"> No evidence of shear zones was found. |
| 2 | <ul style="list-style-type: none"> No clear interpretation of ground profile or pattern was possible from the data corresponding to CPT05 and CPTS6. Because their superimposed q_t, R_f (%) and u_2 (pore pressure) profiles were very different from each other. The only reference about how London Clay behaves was given by CPT04A, which was later used a reference limit. <p>Strength variability</p> <ul style="list-style-type: none"> A zone of weaknesses was identified in CPT05, and it was related to very low values of q_t, and f_s, at a depth of 4,3 m when compared to CPT04A (reference limit). <p>Shear zones</p> <ul style="list-style-type: none"> A potential zone of reworked and sheared material was located between 4,3 m and 6,3 m depth. To confirm the validity of the shear zone, the piezocone tests were plotted in the 3D ground model, along with the identified strike-slip fault prepared by Dixon (2015). As result, the original location of the strike-slip fault was changed by about 5,0 m (See Figure 5.13), and it was placed between CPTS6 and CPT05. The SBTn index was used to classify CPT05 in terms of the recorded response of London Clay. At the same depth of the shear zone, London Clay has a contractive behavior, which contradicts the natural dilative behavior found in section 1, and confirms that contractive layers within London Clay may be related to faulting. |

5.4 Heart of East Greenwich

At this site, 01 borehole, 56 CPTs and 9 Piezocone tests were analyzed according to the recommended procedure in chapter 4. To analyze the existing information, 3 sections were drawn, as shown in Figure 5.14. The interpreted sections and key findings are described in the following paragraphs. The CPT and piezocone profiles were reported by Savvidou, O. (2012).

5.4.1 Interpretation of CPT and Piezocone data

The CPTs belonging to each section were listed in Table 5.4.

Table 5.4
Sections and their corresponding CPTs - Heart of East Greenwich

| Section | CPTs |
|---------|---|
| 1 | • CPT03, CPT13, CPT23, CPT33, CPT43, and CPT53. |
| 2 | • CPT21, CPT22, CPT23, CPT24, CPT25, CPT26, CPT27, CPT28, CPT29, and CPT30. |
| 3 | • CPT06, CPT016, CPT26, CPT36, CPT46, and CPT56. |

Section 2 is perpendicular to section 1 and 3. The last two are parallel to each other. Section 1 was closer to the existing borehole, and it was used to relate the CPT profile with the ground profile, as shown in Figure 5.15. This helped to have an idea of the soil layering over the site and also to aid in the interpretation of section 2 and 3.

Section 1

In Figure 5.15, the ground profile from the existing borehole was drawn for comparison, and next to it the CPT profile in terms of q_c (cone resistance).

Comparison between borehole and CPT03

A good agreement between the existing borehole and CPT03 was found (See Figure 5.15). The river terrace deposits are quite variable with q_c values varying from 2,0 to 76,0 MPa. The upper boundary of Thanet sand formation does not coincide exactly with the limit determined from the CPT data, due to topographic irregularities or an uneven surface.



Figure 5.14: Plan view and cross sections 1, 2 and 3 at Heart of East Greenwich.

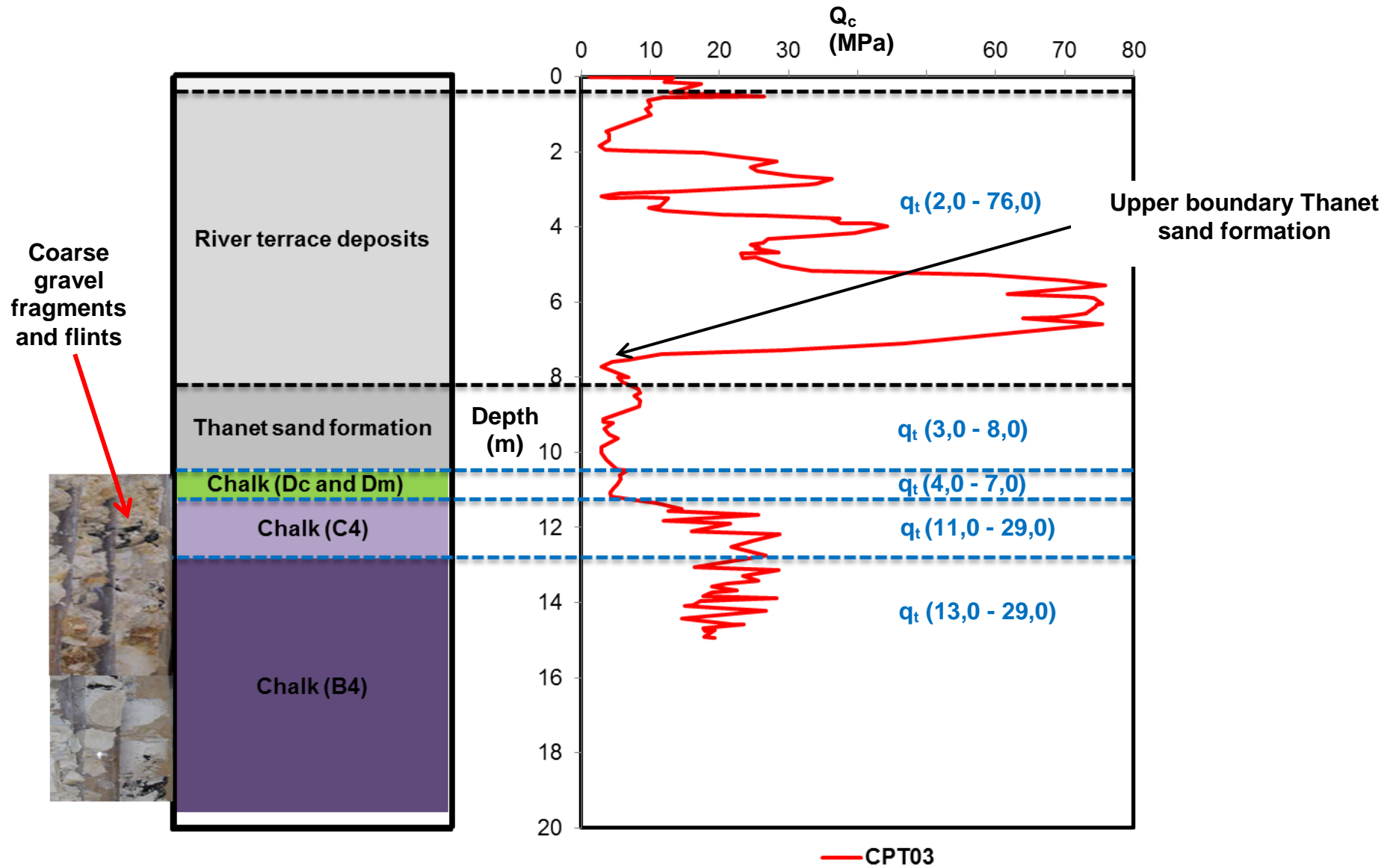


Figure 5.15: CPT03 (q_c profile) and borehole interpretation.

The Thanet sand deposit has a narrow range of q_c values varying from 3,0 to 8,0 MPa. Chalk underlies the Thanet sand deposit, from 10,5 m to 30,0 m depth, and it was divided into different grades: Dc, Dm, C4, B4, and A4/A3. q_c values in the Chalk stratum presents some peaks that may be attributed to the coarse gravel fragments and flints, which are randomly distributed as shown in the retrieved samples (See Figure 5.15).

Finding a signature or pattern

To determine a pattern, all CPT profiles belonging to section 1, were superimposed as shown in Figure 5.16. River terrace deposits were easy to identify as they shown a sharp decrease in q_c values when approaching the upper boundary of the Thanet sand formation. Thanet sand formation and Chalk (grades Dc and Dm), have similar q_c values, and both show peaks in q_c values due to gravel and flints within their matrices.

In this section, there is an increase in thickness of weaker layers between CPT03 and CPT53. However, the only borehole is next to CPT03, and at CPT53 is not possible to determine the boundary between Thanet sand formation and structureless Chalk, because of the similarities in q_c values registered in both stratums at CPT03 (See Figure 5.17).

Strength variability

In Figure 5.16 a decrease in q_c values from CPT03 to CPT53 was identified. For simplicity only CPT03 and CPT53 were plotted in Figure 5.17. It was observed a clear decrease in q_c values starting at a depth of 11,0 m. For instance at a depth of 12,5 m q_c decreases from 25,0 to 8,0 MPa.

Two possible interpretations were made: (1) An increase in thickness of Thanet sand formation which overlies Chalk, or (2) An increase in thickness of structureless Chalk. The thickness of the weak layer (either Thanet sand formation or structureless Chalk), increased by 4,5 m. The distance between CPT03 and CPT53 is 50 m. So, in just a short distance, the changes in q_c values were remarkable.

Reference limit

No reference limit was defined, as the CPT profiles displayed some peaks in q_c values, which make difficult the interpretation without requiring additional assumptions. The peaks were related to gravel fragments or flints.

Identification of shear zones

No shear zones were found, but a decrease in strength with distance was identified. The direction of weakening was oriented towards SE, almost parallel to a near off-site fault was identified by Mason et al. (2015) (as cited by Linde-Arias, 2017).

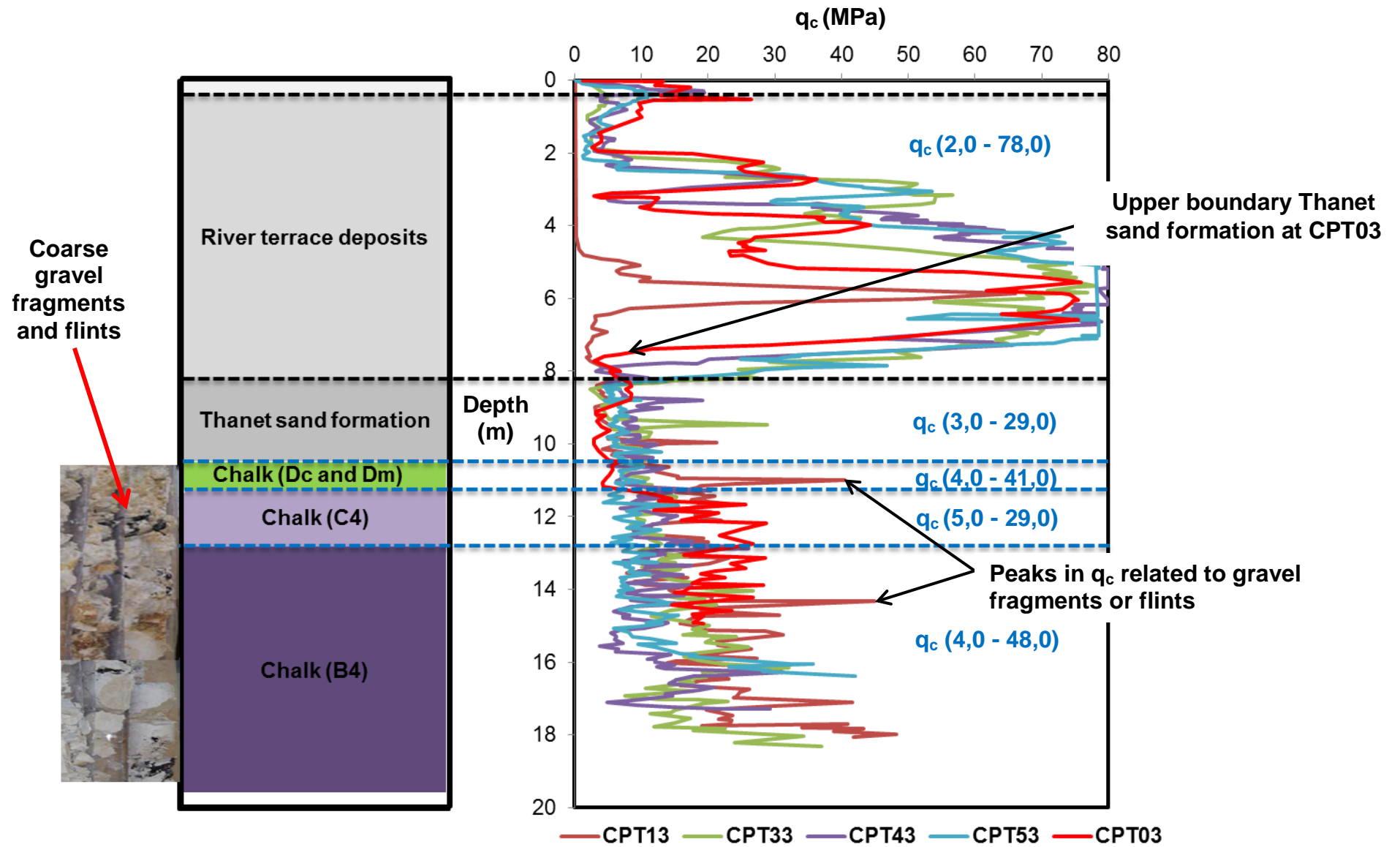


Figure 5.16: Superimposed CPT profiles in terms of q_c - Section 1.

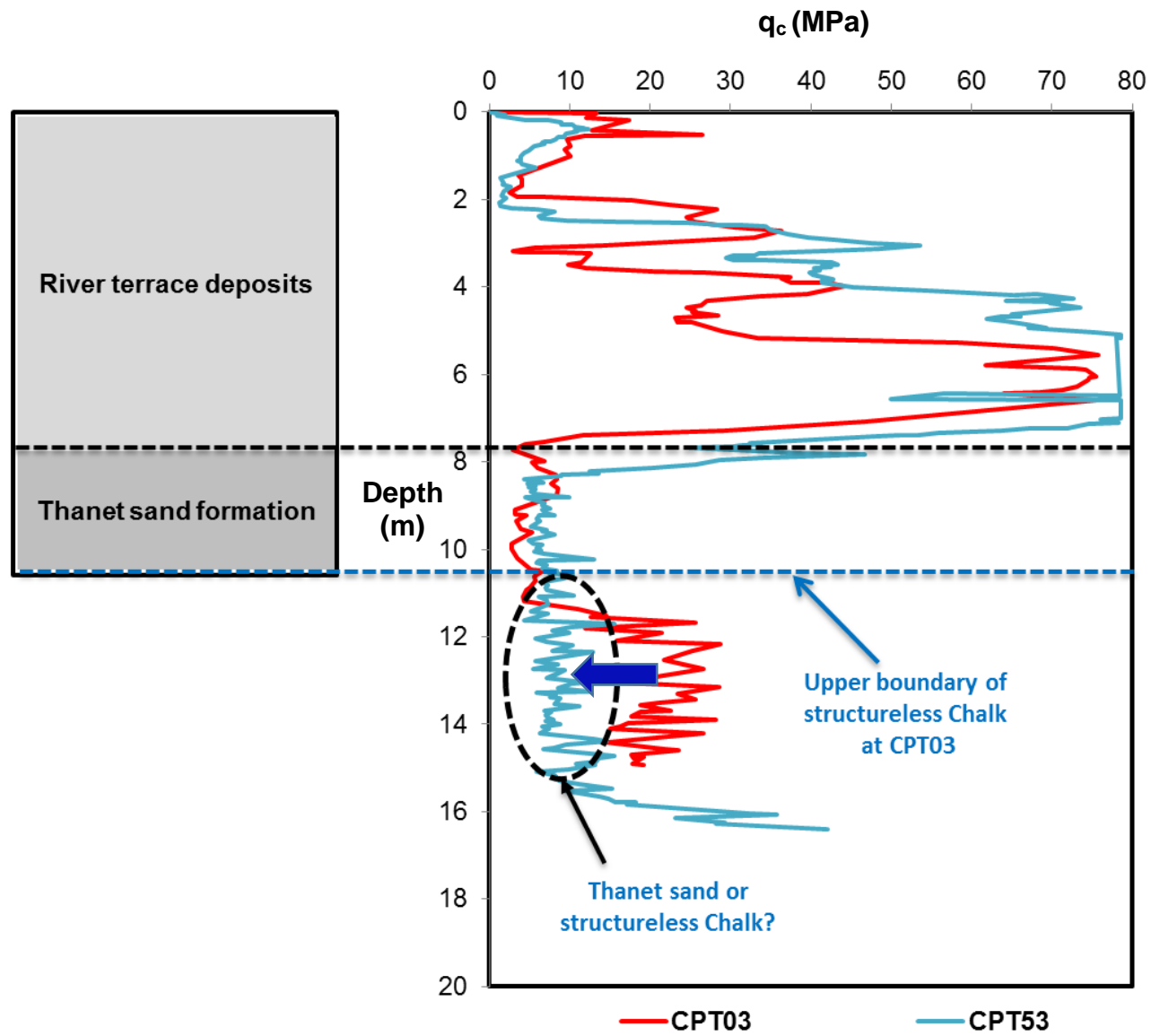


Figure 5.17: Strength variability between CPT03 and CPT53.

Section 2

In Figure 5.18 and Figure 5.19, an inferred ground profile was drawn and next to it the CPT profile in terms of q_c . The stratigraphy is made up of River terrace deposits, which overlies Thanet sand formation and Chalk. From q_c values was possible to accurately define the thickness of the River terrace deposit, whereas for the Thanet sand formation and Chalk, this process was difficult and it was preferred to just named the underlying layers as “Thanet sand formation and Chalk”.

Finding a signature or pattern

Consecutive CPT tests have similar q_c profiles. For instance, CPT24 and CPT25 (Figure 5.18) and CPT27 to CPT30 (Figure 5.19), showed a pattern. This highlights the repeatability that could be obtained in q_c profiles from CPT tests. Overall, in section 2 the pattern is made up of river terrace deposit overlying Thanet sand formation and Chalk.

Comments about the inferred stratigraphy

In Figure 5.20 the thickness of the river terrace deposit decreases by 1,4 m, from CPT24 towards CPT30. In CPT24 the lower boundary of the river terrace deposit is located at a depth of 8,2 m, and in CPT30 it is located at a depth of 6,8 m. This change in thickness is represented by an jump from 8,2 m to 6,8 m of the lower boundary of the river terrace deposit (See red arrow pointing upwards in Figure 5.20). There are also some peaks randomly distributed along the profile, related to gravel fragments and flints.

Strength variability

A continuous decrease in strength was identified from CPT24 towards CPT30. To aid in the discussion of this variability CPT24, CPT26 and CPT29 were plotted in the same graph as shown in Figure 5.21. In this figure, the strength decreases mainly inside the green square, where the green arrow represents this trend (from CPT26 to CPT29).

The highest reduction was about 20,0 MPa, however the presence of gravel fragments and flints within Chalk and Thanet sand formation made the interpretation difficult.

Reference limit

As in section 1, the CPT profiles displayed some peaks, which induced some broadening of q_c values and uncertainty of the real behavior of Chalk. So, no reference limit was defined.

Identification of shear zones

No shear zones were found, but a continuous decrease in strength from CPT24 to CPT30 was identified, this decrease was gradual with no abrupt or sharp changes, oriented towards NE, approximately perpendicular to a near off-site fault identified by Mason et al. (2015) (as cited by Linde-Arias, 2017).

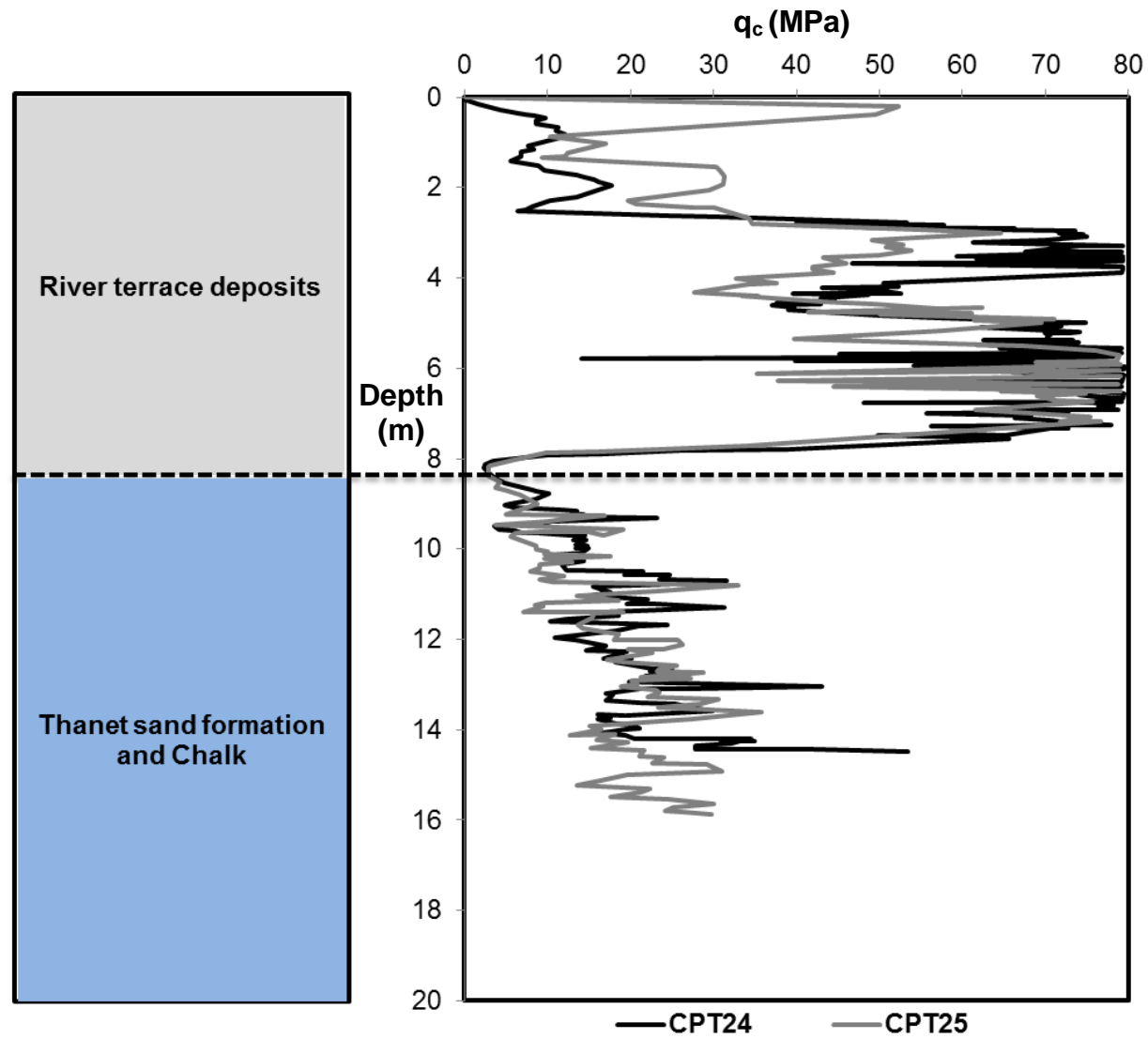


Figure 5.18: Inferred ground profile, and q_c profiles of consecutive CPTs (CPT24 and CPT25).

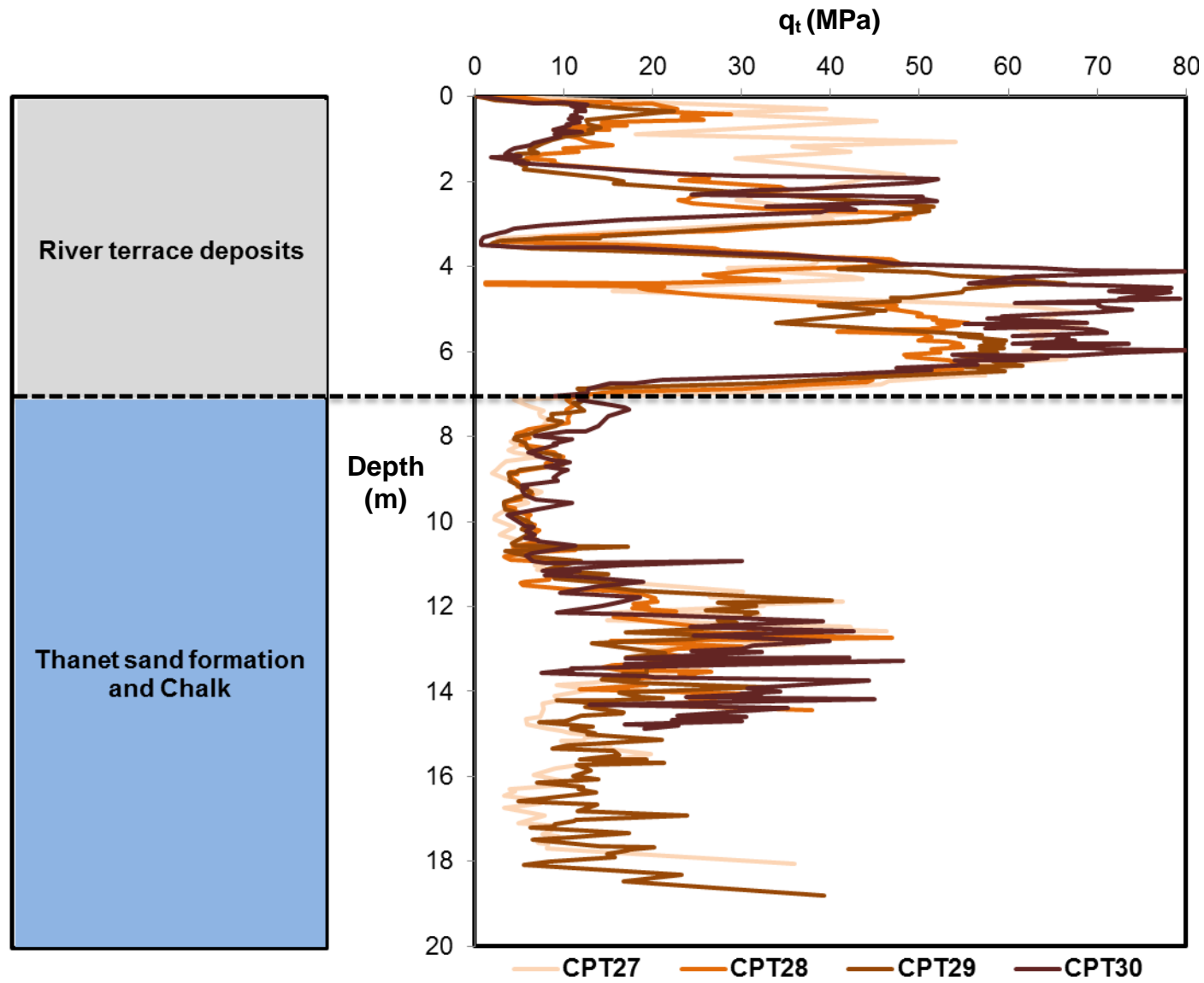


Figure 5.19: Inferred ground profile, and q_c profiles of consecutive CPTs (CPT27, CPT28, CPT29, and CPT30).

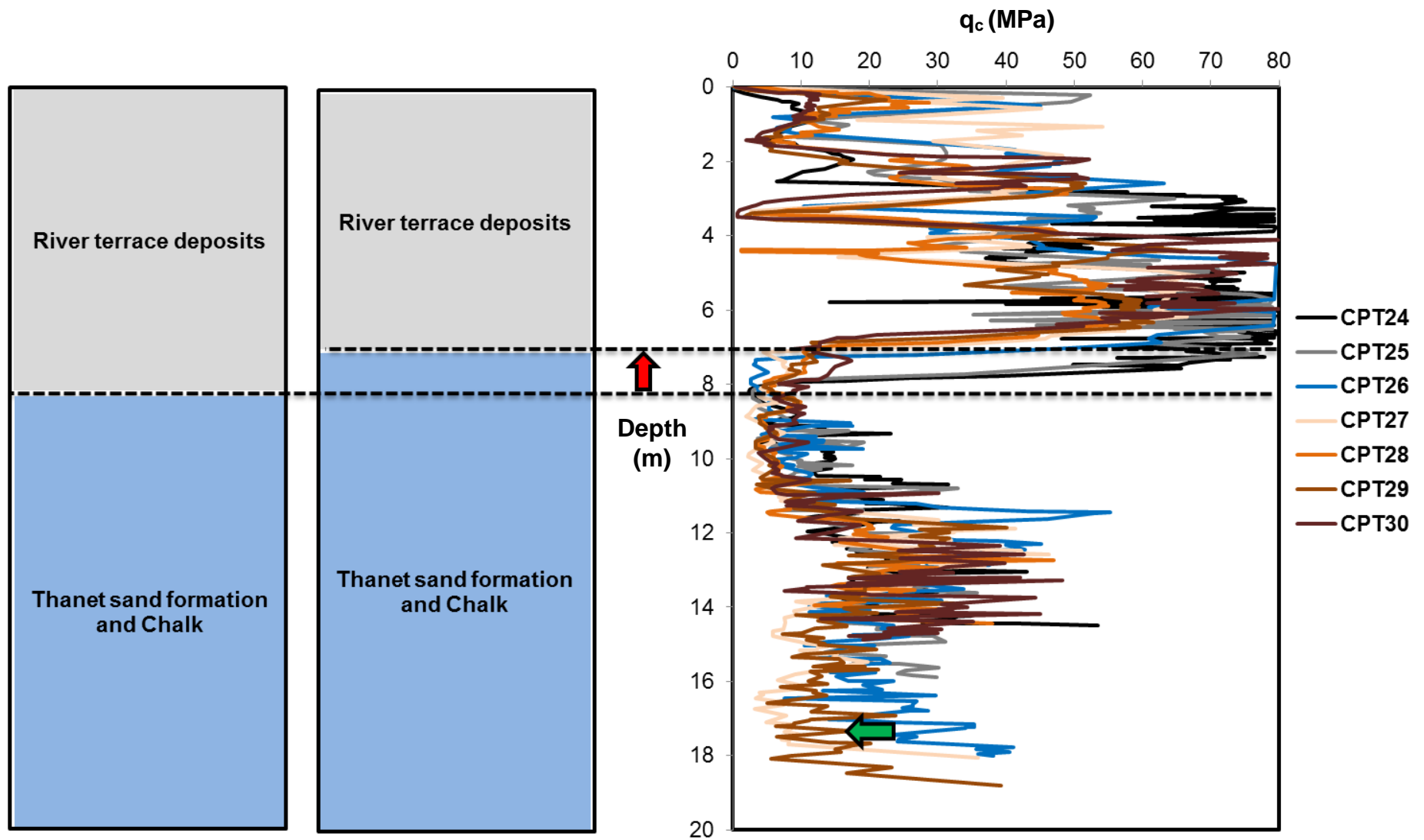


Figure 5.20: Superimposed CPT profiles - Section 2.

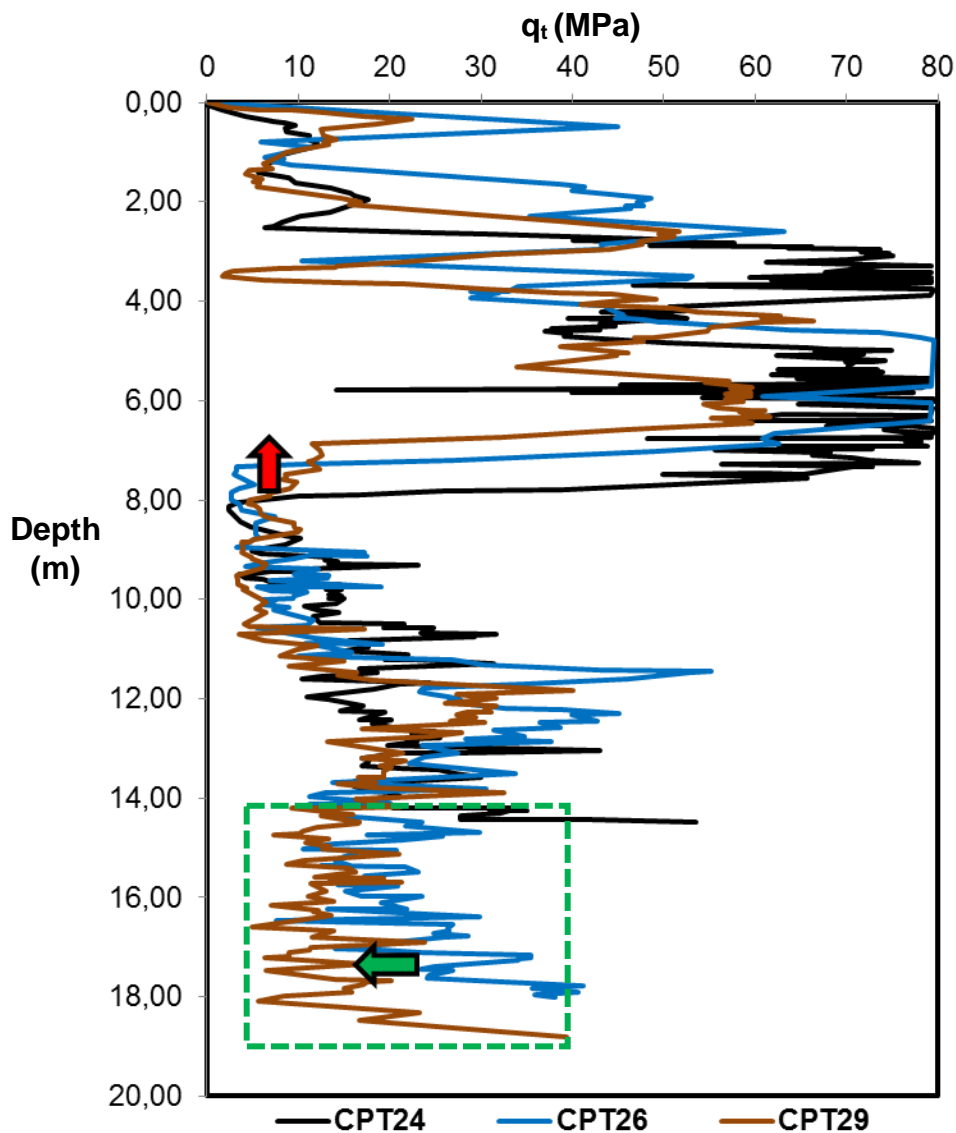


Figure 5.21: Strength variability between CPT24, CPT26, and CPT29.

Section 3

In Figure 5.22 and Figure 5.23, an inferred ground profile was drawn and next to it the CPT profile in terms of q_c . The stratigraphy is made up of River terrace deposits, which overlies Thanet sand formation and Chalk. From q_c values was possible to accurately define the thickness of the River terrace deposits, whereas for the Thanet sand formation and Chalk, this process was difficult and it was preferred to just named the underlying layers as “Thanet sand formation and Chalk”.

Finding a signature or pattern

As in section 2, when superimposed, consecutive profiles showed a pattern. In section 3, these are CPT06 and CPT16, and CPT46 and CPT56, respectively. q_c values within Thanet sand formation and Chalk were very close to each other, whereas the river terrace

deposit showed a higher variability, but still its thickness (i.e. lower boundary) could be identified easily.

Comments about the inferred stratigraphy

In Figure 5.24 the thickness of the river terrace deposit decreases by 0,4 m, from CPT06 towards CPT56. In CPT06 the lower boundary of the river terrace deposit is located at a depth of 7,7 m, and in CPT56 it is located at a depth of 7,3 m. This change in thickness is represented by an small jump from 7,7 m to 7,3 m of the lower boundary of the river terrace deposit (See red arrow pointing upwards in Figure 5.24). There are also some peaks randomly distributed along the profile, related to gravel fragments and flints.

Strength variability

A continuous decrease in strength was identified from CPT06 towards CPT56. This variability can be shown in Figure 5.24, where the strength decreases mainly inside the green square, and a the green arrow shows this trend,

The highest reduction was about 18,0 MPa, however there is a uncertainty induced by the presence of gravel fragments and flints which are the source of the peaks in q_c values, typical of Chalk and Thanet sand formation.

Reference limit

Similar to section 1 and section 2, the presence of peaks due to gravel or flints, does not allow a clear definition of reference limits, so rather than trying to find a reference limit, the analysis was focus on strength variability at specific depths and soil layering when possible.

Identification of shear zones

No shear zones were found, but apparently the mechanical weakening of these layers was oriented towards SE almost parallel to a near off-site fault identified by Mason et al. (2015) (as cited by Linde-Arias, 2017). In this section, the zone at which the strength reduces was located between 11,0 m and 13,0 m depth (See green box in Figure 5.24).

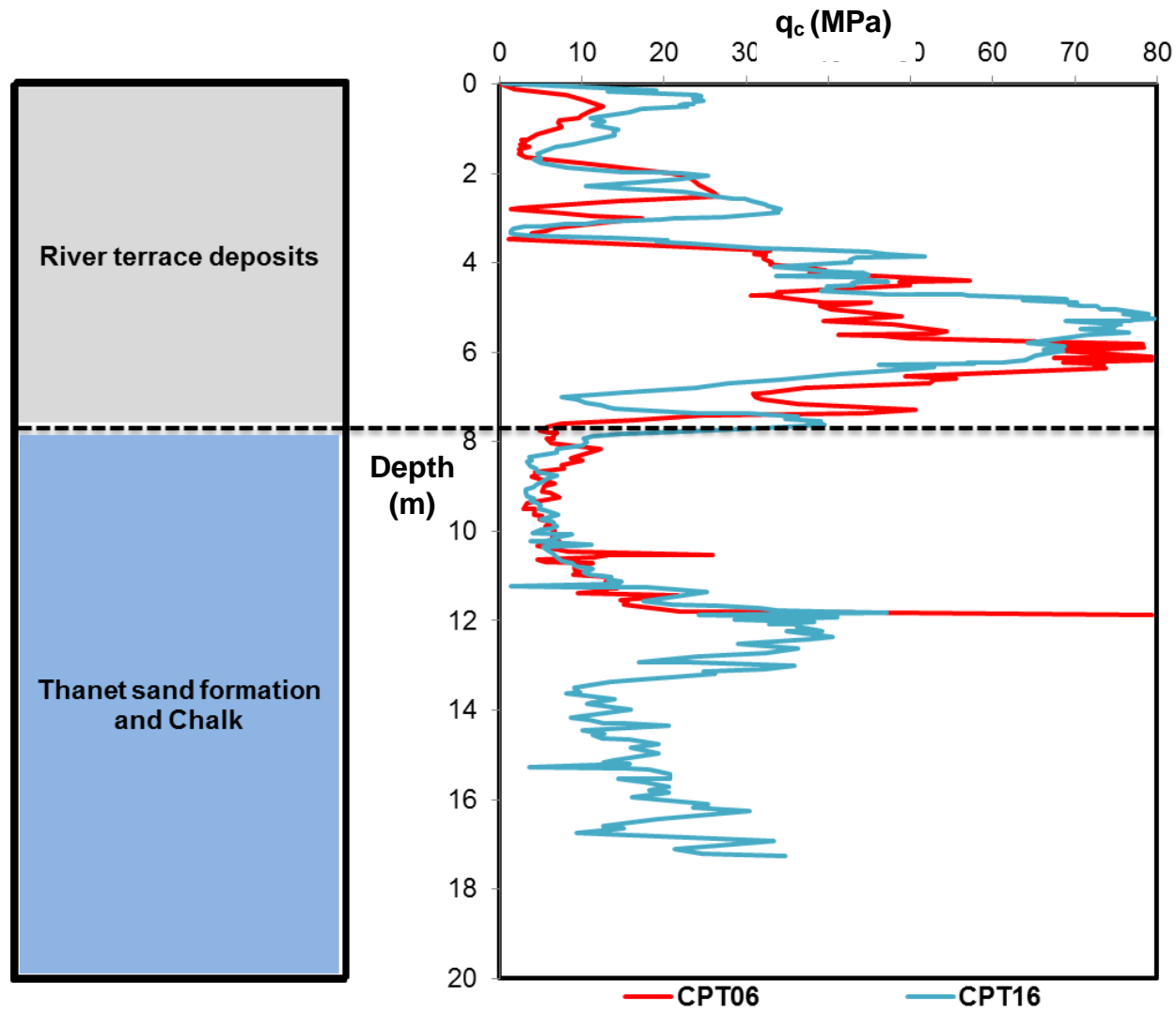


Figure 5.22: Inferred ground profile, and CPT profiles of consecutive CPTs (CPT06 and CPT16).

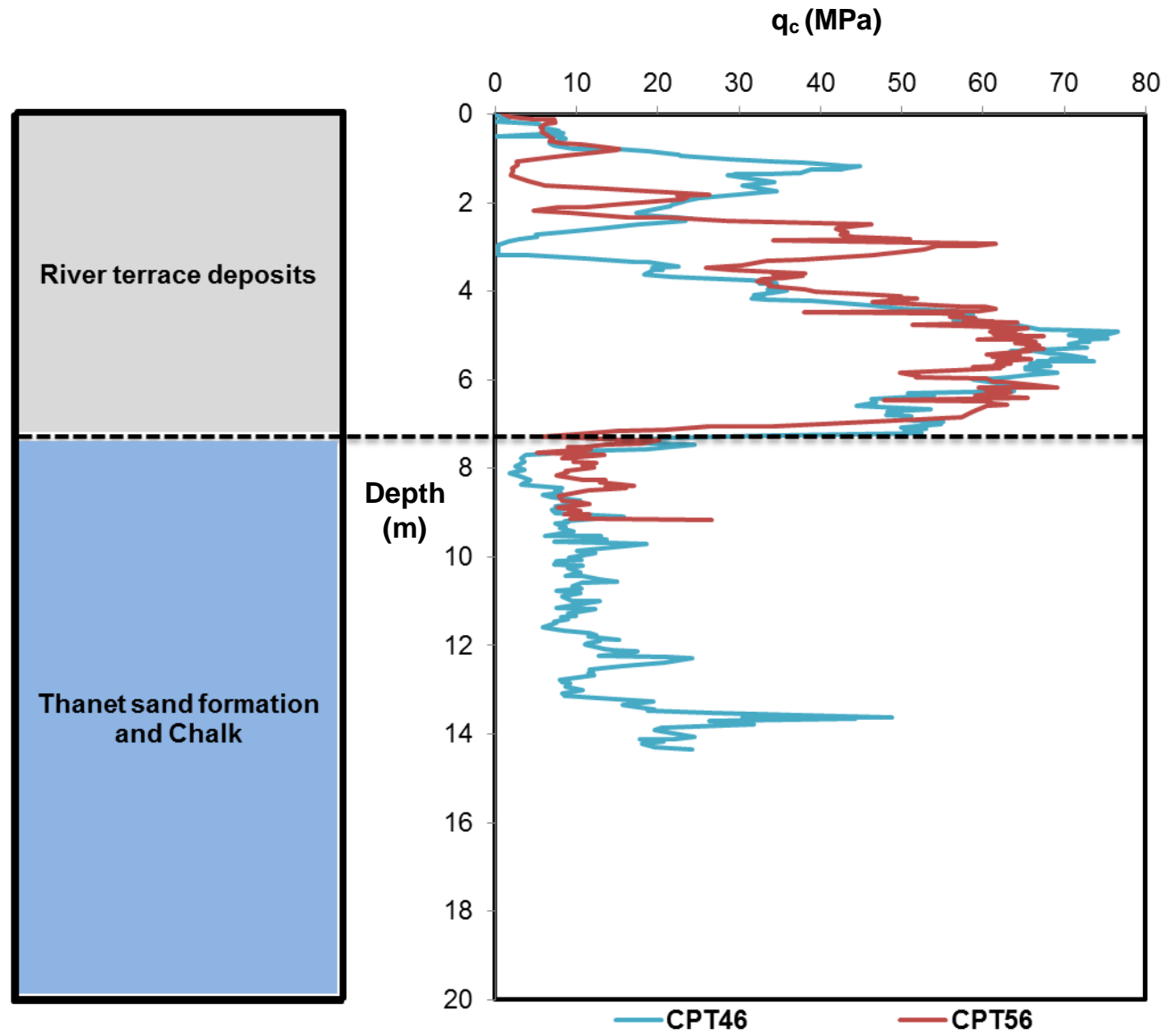


Figure 5.23: Inferred ground profile, and CPT profiles of consecutive CPTs (CPT46 and CPT56).

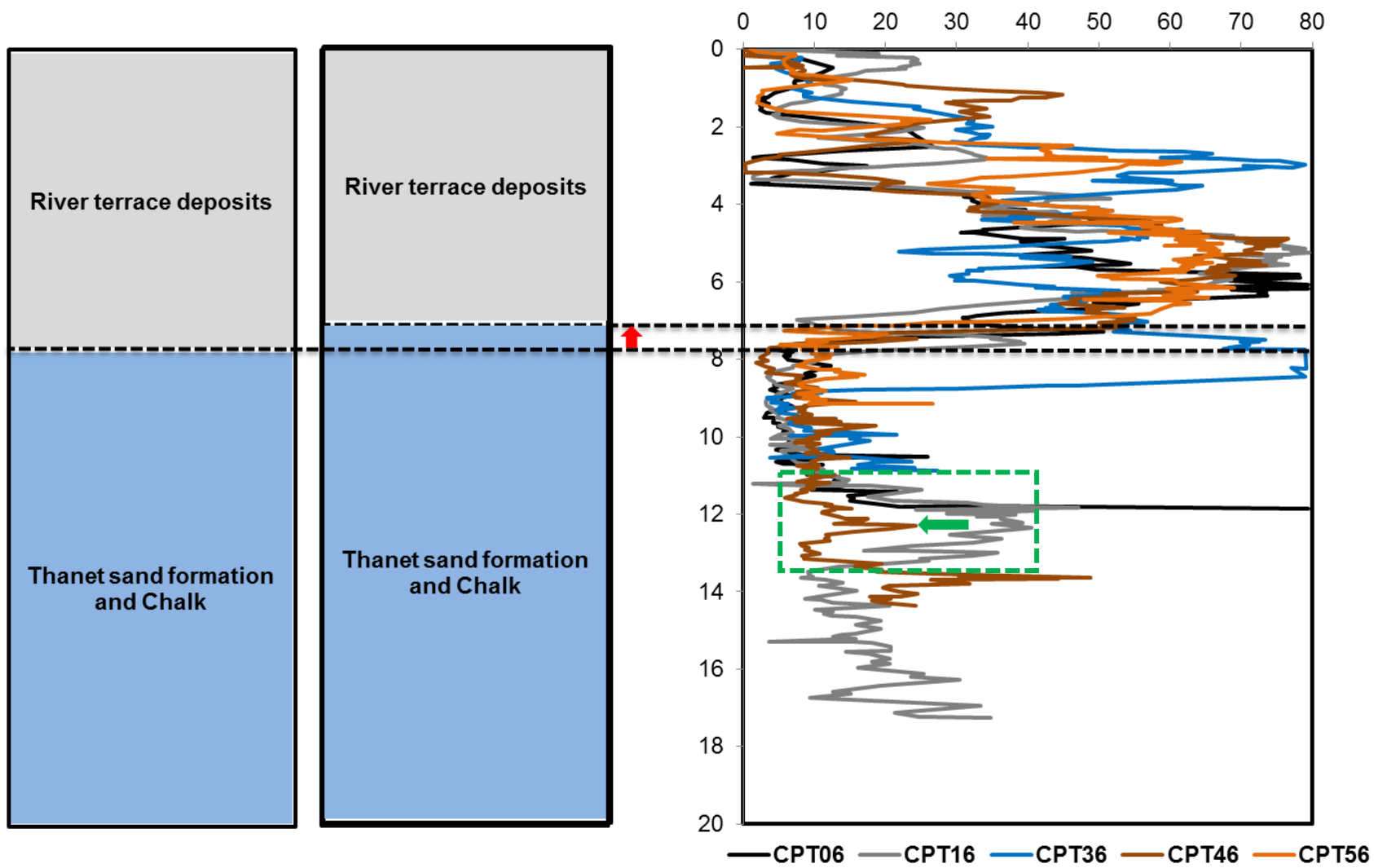


Figure 5.24: CPT profiles, projection of CPTs belonging to section 3.

5.4.2 Results

The main findings at the Heart of East Greenwich were summarized in Table 5.5.

Table 5.5
Main findings

| Findings |
|--|
| <p>Strength variability</p> <ul style="list-style-type: none">• The presence of gravel fragments and flints influences greatly in the interpretation of the CPT profiles.• The strength decreases gradually with no abrupt or sharp changes. <p>Repeatability</p> <ul style="list-style-type: none">• When plotted together, consecutive tests showed similar q_c profiles, with a minor difference (2,0 or 4,0 MPa). This highlights the repeatability that could be obtained with the CPT test. <p>Shear zones</p> <ul style="list-style-type: none">• The identified decrease in strength was oriented towards SE in sections 1 and 3, and towards NE in section 2. With a resultant direction of mechanical weakening towards SE. Approximately to the direction at which a near off-site fault was identified by Mason et al. (2015) (as cited by Linde-Arias, 2017). From this information a strength variability map was prepared as shown in Figure 5.25. |

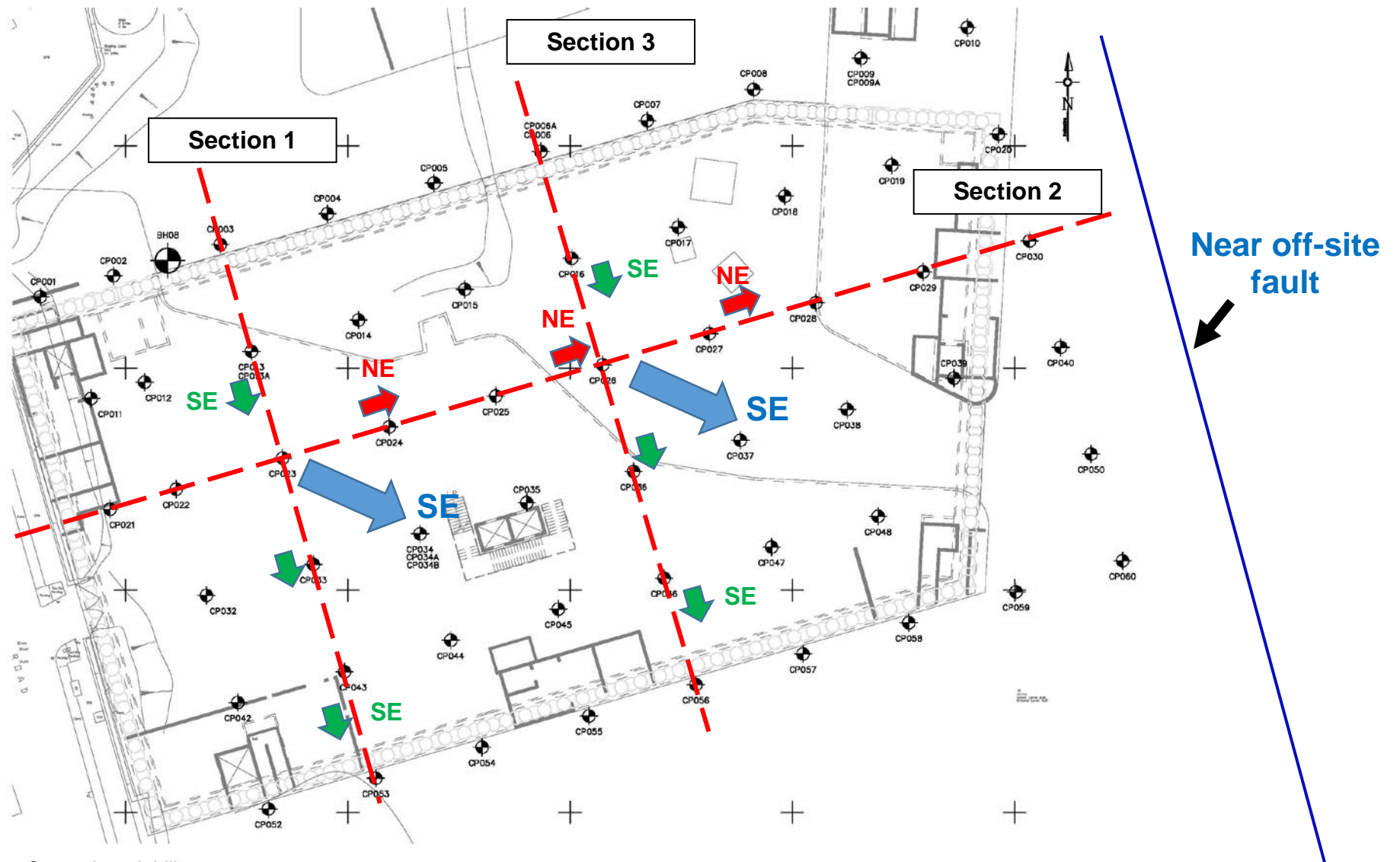


Figure 5.25: Strength variability map

6 DISCUSSION

In this chapter some key findings revealed by the results of the analysis carried out in the present research project, are discussed. In general, these findings are related to the fact that faulting has a clear and traceable impact on shear strength properties, and in situ tests such as: CPT and piezocone tests can confirm this.

One promising finding is that CPT and piezocone tests performed well by showing consistent and repeatable strength profiles, which provided an insight on how strength varies spatially, so, even if no faults are found, a potential direction of weakening towards existing damaged zones could be identified (for off-site faults).

Findings at St. James' Square

The results obtained at St. James' Square exposed an abrupt change in strength within London Clay, which contradicts the assumption that London Clay formation is uniform. This abrupt change was spotted by direct interpretation of piezocone tests.

2 out of 6 piezocone tests, showed totally different profiles compared with a reference limit. By analyzing the data of these tests using the proposed methodology, a potential zone of reworked and sheared material was identified. This was interpreted as the position of the fault. To confirm its validity, the piezocone tests were plotted in a 3D ground model representing the top of London Clay, along with a strike-slip fault reported by Dixon (2015).

The position of a strike-slip fault originally inferred by Dixon (2015) was moved to reflect the new interpretation of the fault position. The fault plane was placed along these piezocone tests, and the extent of the damage zone corresponds to the thickness of the shear zone.

Findings the Heart of East Greenwich

In all CPT profiles at the Heart of East Greenwich, the registered q_t , f_s , u_2 (pore pressure) values were highly influenced by the presence of gravel fragments and flints. These obstructions were present mainly in Thanet sand formation and Chalk. Because of the uncertainties induced by these obstacles, 3 different interpretations about the ground conditions arose:

1. An increasing thickness of Thanet sand formation with distance oriented towards SE.
2. An increasing thickness of structureless chalk with distance oriented towards SE.
3. Off-site presence of fault.

As evidence about an off-site fault near the area of study was available, the 3rd interpretation was the most likely. Also, the analysis of CPT tests suggested that the resultant vector of decrease in strength of the site, was oriented towards south east, where a sinistral wrench fault was identified by Mason et al. (2015) (as cited by Linde-Arias et al., 2017).

Although, the other interpretations could be valid as well, more boreholes and sampling are required. The boreholes may confirm the presence or absence of shear zones, as polished surfaces can be identified by visual inspection. In addition the changes in thickness and lithology could be determined.

The previous paragraph highlights the main limitation of the performed CPT tests, which is the lack of borehole sampling; however, the registered values provided an insight into the spatial variability of strength across the site. If additional boreholes were carried out, samples could be retrieved at specific depths to further confirm the cause of the strength decrease.

7 CONCLUSIONS AND RECOMMENDATIONS

The research carried out in this dissertation has led to the following conclusions and recommendations:

Conclusions:

St. James' Square

Shear zones: The piezocone tests performed at St. James' Square showed a definite boundary between London Clay and the overlying layers. This was expected, because of the additional resolution provided by the measured pore pressures. Moreover, 4 out of 6 tests displayed a relatively consistent and uniform layer of London Clay, which was assumed as a reference limit. The other 2 tests were compared with the established reference limit. As result a shear zone was identified, which amended the position of a fault plane originally reported by Dixon (2015), reflecting the new interpretation of the fault position.

Limitations: The extent of the damage zone is unknown, and more data is required, in order to precisely define the impact of the mechanical weakening induced by faulting on the London Clay formation. However, with the available information was possible to determine to some degree a reworked zone and this could be considered as a preliminary assessment.

Heart of East Greenwich

Shear zones: No shear zones were identified. CPT profiles showed strength variability with no abrupt changes, but with a common weakening orientation towards an interpreted off-site fault. This shows the successful application of CPT tests not only in identifying layering, but also in the assessment of natural spatial variability of strength or the mechanical impact on strength due to faulting.

Repeatability: Consecutive CPT profiles were very similar, highlighting CPT repeatability. This is a key feature when trying to assess strength variability. If there is a sharp or abrupt change in properties over a short distance, then there is the likelihood to find a fault or reworked material within a damaged zone. In the standard CPT configuration, q_c values are often more reliable than f_s values, as the sleeve is positioned in a zone of high stress gradient. So, q_c values were used to assess repeatability.

Lithography: At the Heart of East Greenwich, the boundary between Thanet sand and Structureless Chalk cannot be determined only from CPT data. The main reason is that those layers showed similar q_c , f_s values, as well as peaks in q_c and f_s , due to gravel and flints, randomly distributed along all the CPT profiles. So, the analysis is considered as preliminary, unless laboratory data and soil sampling is available. Because of this, the ground profile was interpreted as river terrace deposits, and the underlying layers were named just as “Thanet sand formation and Chalk”.

Limitation: The CPT profiles in Chalk and Thanet sand were highly affected by the presence of gravel fragments and flints. The profiles showed peaks in strength related to the identified obstacles, resulting in a broadening of the registered CPT data and not allowing an accurate interpretation.

Because of the obstructions present in Chalk and Thanet sand, CPT data led to several interpretations. The possible interpretations were 1) an increasing thickness of Thanet sand, 2) an increasing thickness of structureless chalk or 3) mechanical impact associated with an off-site fault. The most like was the third interpretation, as the orientation of weakening was pointing towards an existing fault. So, CPT in conjunction with a local geology and off-site or on-site discontinuities or periglacial features will enhance CPT data interpretation and effectiveness.

Recommendations:

A CPT Dataset of CPT tests carried out in London Clay should be prepared. According to the results of this research, CPT data in London Clay was quite uniform and a reference limit was easy to establish. But, in projects where just one or two CPT tests are available in London Clay, a dataset (used as a reference) limit may aid in the interpretation of CPT data to assess the potential presence of faulting. A dataset for Chalk will be worthwhile, to some degree, as it presents in general some peaks associated to flints, and it may require sampling to discard the presence or not of flints. In addition, laboratory tests could determine its density, and water content, to better classify its natural state to assess the changes in properties induced by faulting.

Geophysical tests such as electrical resistivity to determine the location and geometry of faults, and stratigraphic mapping (variation in thickness and bedrock depth). Especially in chalk where flints are a source of uncertainty when interpreting CPT data, as this test

does not involve penetration. Another alternative to complement CPT data is a microgravity survey, to determine the lateral extent of a damage zone over a specific area, by relating low density to a weakened ground.

One last geophysical test could be seismic refraction to register p and s waves. This is already applied to characterize the subsurface geologic conditions. If a series of CPT tests are aligned parallel to a seismic survey line, a better appreciation of the strength variability and layering could be obtained without further assumptions. What it is more, the number and position of boreholes will be optimized, if performed in areas where anomalies or shear zones are located.

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