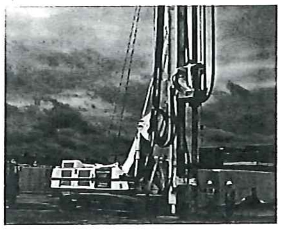

Innovation in the design of continuous flight auger and bored displacement piles

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INNOVATION IN THE DESIGN OF CONTINUOUS FLIGHT AUGER AND BORED DISPLACEMENT PILES

By

David James Baxter

A dissertation thesis submitted in partial fulfilment of the requirements for the award of the degree Doctor of Engineering (EngD), at Loughborough University

[June 2009]

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ACKNOWLEDGEMENTS

This research was made possible through funding from the Engineering and Physical Sciences Research Council and from Rock and Alluvium Ltd, for this I would like to express my sincere gratitude.

I would like to thank my Academic Supervisors at Loughborough University, Dr Paul Fleming and Professor Neil Dixon, for their generous and plentiful support, guidance and advice throughout the research period. Thanks are also extended to Dr Ashraf El-Hamalawi who regularly provided a fresh perspective.

I would also like to thank all of the staff, past and present, at Rock and Alluvium who have helped me throughout this research. In particular I would like to show my appreciation for the efforts of my Industrial Supervisors Steve Hadley and Ken Cromwell, who have provided direction, advice and practical insight as well as the time and resources necessary to follow my research interests.

The support of the Centre for Innovative and Collaborative Engineering has also been much appreciated. I would particularly like to acknowledge the guidance received from Professor Dino Bouchlaghem and Professor Chimay Anumba and the assistance of Jo Brewin, Colette Bujdoso, Sara Cowin, and Nadine Smith.

The time and efforts of the examiners, Dr Matthew Frost, Dr Charles Hird and Dr Matthew Raybould are greatly appreciated, as is the feedback they provided.

There are numerous friends and fellow Research Engineers who have, whether they know it or not, helped me to stay the course.

Finally my deepest gratitude goes to my wife, Jenny, for her patience, understanding, support and encouragement and of course to my children, Sophia and Thomas, who have made me smile every day no matter what.

ABSTRACT

The field of pile design and construction in the United Kingdom is currently in a period of change. Not only are new processes and techniques being developed but also the legislative landscape is changing with the introduction of new Europe wide normative standards (BSI, 2004, 2007).

This project sought to optimise pile design through better understanding and interpretation of ground conditions and of the pile-soil interaction for two pile types, continuous flight auger and bored displacement piles.

Quantitative methods for interpreting and summarising previous knowledge and experience have been developed; the strata are divided into discrete bands and properties are represented with summary statistics. Experience and previous knowledge relating to the shear strength of London Clay have been quantified and presented using this approach. Furthermore, a straightforward tool has been provided for the implementation of such data into design; the previous knowledge and new site specific data are combined using Bayesian updating. Through use of this technique, the uncertainty associated with interpreting ground conditions from site data has been demonstrated to be reduced. The techniques described have been adopted into design practice within the sponsoring company.

Bored displacement piles are a relatively new pile type. There is little published data or scientific understanding of the processes undergone by the soils during and after construction and the effect that these have on pile performance. This research identified the need for, and developed, a unified framework of descriptors for the various types of bored displacement pile and investigated the installation energy and performance of bored displacement piles in London Clay. The energy to construct the pile was observed to be highly variable and not directly related to capacity. Performance of bored displacement piles was observed to be similar to continuous flight auger piles of similar dimensions; typical values for the adhesion between bored displacement piles and the surrounding soil were established and these were comparable to those achieved by continuous flight auger piles.

In addition, to enable the analysis of the probability of failure of a pile, the sources of variation have been investigated and the variability quantified. Besides the soil conditions, the dimensions of the pile, notably pile diameter, were found to be a significant source of variation.

KEY WORDS

Piles and piling, design methods, ground modelling, London Clay, bored displacement piling, continuous flight auger piling, case histories.

PREFACE

This thesis presents the research conducted from 2005 to 2008 to fulfil the requirements of an Engineering Doctorate (EngD) at the Centre for Innovative and Collaborative Engineering (CICE), Loughborough University, United Kingdom. An industrial context for the research was provided through collaboration with Rock and Alluvium Ltd, a UK based specialist piling contractor. The research was also funded by the Engineering and Physical Sciences Research Council (EPSRC).

The core of the EngD is the solution of one or more significant and challenging engineering problems within an industrial context. The problems are identified with the industrial partner such that the research has direct relevance to the business and provides demonstrable benefit. The research must not only innovate but also demonstrate implementation of that innovation.

This thesis provides the aim, objectives, methodology, findings and industrial impact and implications of the research. The collection of published papers, which is appended to this thesis, forms an integral part of the research and these should be read in conjunction with the main text. The papers are referenced from within the discourse and provide further in-depth technical detail.

USED ACRONYMS / ABBREVIATIONS

A_b	area of the base of the pile
A_s	area of the pile shaft in contact with the soil
BD	bored displacement
c_a	average adhesion (between soil and pile)
C_b	undrained shear strength at base of pile
$\bar{c}$	average undrained shear strength
CBD	Chessington BD test pile.
CFA	continuous flight auger
CRP	constant rate of penetration pile load test
c_u	undrained shear strength
d	depth (below ground level)
E	Expected Frequency
E_s	screwing in energy
F	factor of safety
F^+	factor of safety with a parameter decreased by 1 standard deviation
F^-	factor of safety with a parameter increased by 1 standard deviation
$\bar{F}$	mean factor of safety
F_{MLV}	factor of safety associated with most likely value
I_p	plasticity index
M_i	torque applied to an auger
ML	maintained load pile load test
MLV	most likely value
N	standard penetration resistance (SPT blow count)
n	number of sample points
N_c	bearing capacity factor
N_d	vertical thrust of an auger
n_i	rotational speed of an auger
O	Observed frequency

P_f	probability of failure
$Q_{theoretical}$	theoretical, calculated capacity
Q_{actual}	actual observed capacity
Q_b	base/end bearing resistance
q_{c1N}	cone resistance
Q_s	frictional shaft resistance
Q_u	ultimate pile resistance
R^2	correlation coefficient
SPT	standard penetration test
s	standard deviation of a sample
v_f	coefficient of variation of factor of safety
v_i	vertical speed of an auger
v_x	coefficient of variation
WBD1	Wimbledon BD test pile number 1
w	Moisture content
w_L	Liquid limit
WBD2	Wimbledon BD test pile number 2
WCFA	Wimbledon CFA test pile
x	sample value
$\bar{x}$	mean of a sample
x_k	characteristic value
x_m	mean shear strength within a depth band
α	adhesion factor
ΔF	change in factor of safety
μ	mean of a population
μ'	mean of prior distribution
μ''	mean of posterior data
σ	standard deviation of a population
σ'	standard deviation of prior data
σ'''	standard deviation of posterior distribution
σ_F	standard deviation of factor of safety
σ^2	variance
ν	Degrees of freedom

χ^2_{test}	Chi squared test statistic
$\chi^2_{5\%}$	Chi squared distribution and 5% significance
Ω	area of the borehole
i	refers to the screwing in phase

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LIST OF PAPERS

The following papers, included in the appendices, have been produced in partial fulfilment of the award requirements of the Engineering Doctorate during the course of the research. These are referenced throughout the main text of this thesis.

Appendix A	Paper 1	BAXTER, D.J., DIXON, N., FLEMING, P.R. AND CROMWELL, K.A., 2008, Refining characteristic values of shear strength using experience, *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering*, **161**, pp247-257.
Appendix B	Paper 2	BAXTER, D.J., DIXON, N., FLEMING, P.R. AND CROMWELL, K.A., 2008, Using Experience And Case History Data To Enhance The Design Of Piled Foundations And Predict Behaviour Characteristics*, Proceedings of the 6th International Conference on Case Histories in Geotechnical Engineering,* Arlington VA, 11th-16th August
Appendix C	Paper 3	BAXTER, D.J., DIXON, N., FLEMING, P.R. AND HADLEY S.H., 2006, Bored Displacement Piles – A United Kingdom Perspective, *Proceedings of the 10th International conference on Piling and Deep Foundations*, Amsterdam, 31st May – 2nd June, pp210-218
Appendix D	Paper 4	BAXTER, D.J., FLEMING, P.R., DIXON, N. AND CROMWELL, K.A., 2008 Bored Displacement Piles In Stiff Clay: construction and performance, *Proceedings of the 2nd BGA International Conference on Foundations,* Dundee, 24th -27th June.

1 INTRODUCTION

This chapter provides an introduction to the general subject area. The context of the research within the piling industry is described and the Industrial Sponsor, Rock and Alluvium, is introduced. The aim of the research is stated along with its supporting objectives and a description of structure of the thesis is presented.

1.1 BACKGROUND TO THE RESEARCH

1.1.1 PILED FOUNDATIONS

Piles have been used to provide foundations for over 4000 years and were widely used by the Ancient Greeks and Romans (Fleming *et al.*, 1992). Modern piling techniques are diverse and the term 'pile' is used to describe a wide variety of columnar elements in a foundation which transfer load from the superstructure through weak or compressible strata to more competent soils or rock (Tomlinson, 1994).

Piles may be crudely classified as either displacement or replacement types, according to their method of construction. Each of these major categories can then be further divided. Figure 1.1 shows a classification of pile types (after Weltman and Little, 1977). Fleming *et al.* (1992) illustrated the main differences and pile properties, which can be summarised as follows.

Displacement piles displace soil radially as the pile is driven or jacked into the ground. This compacts coarse grained soils and can lead to enhanced soil strength and pile bearing capacity. Fine grained materials may heave (particularly if saturated) as there is little possibility of immediate volume change. Replacement piles remove soil from the pile location to allow construction of the pile. Construction of replacement piles reduces lateral stress in the ground. The problems of noise and vibration associated with displacement piles may be avoided by using replacement piles, although this is at the cost of the loss of the benefit from soil compaction.

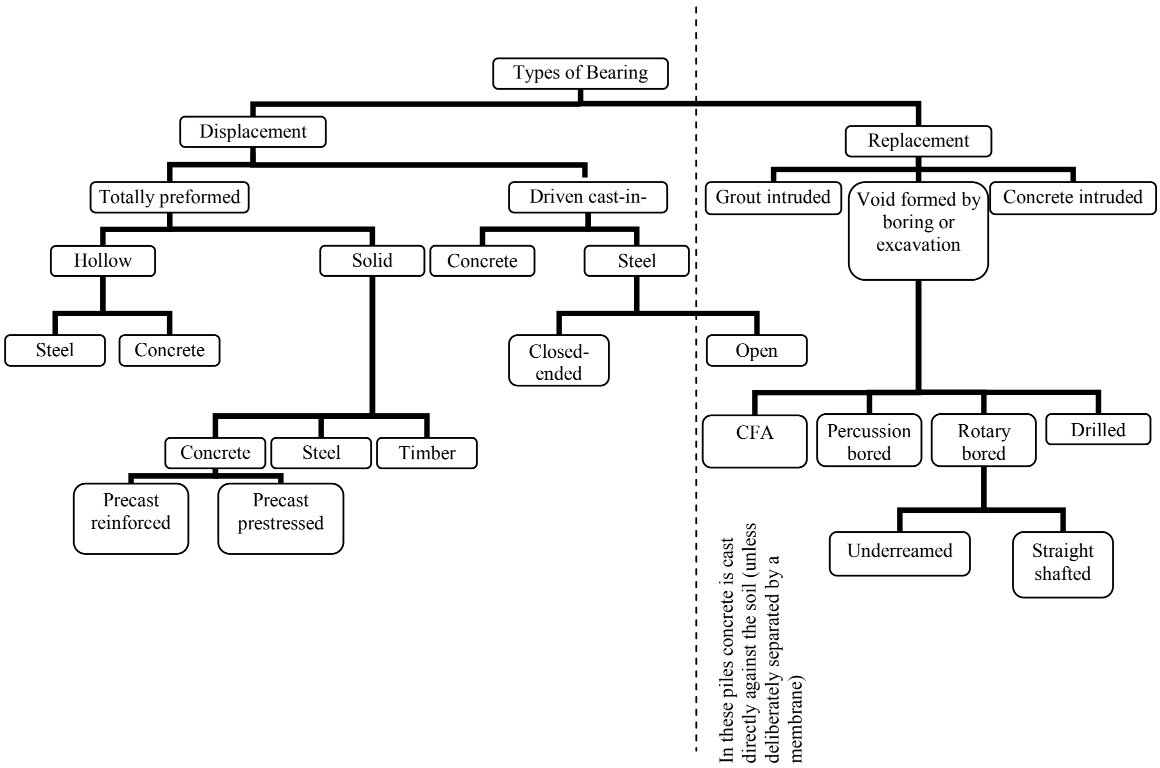

Figure 1.1. Classification of pile types (After Weltman and Little (1977)).

The focus of this research is on continuous flight auger piles (CFA) and bored displacement (BD) piles. This focus stems from the interests of the Industrial Sponsor, Rock and Alluvium. CFA piles are a subset of bored replacement piles formed by drilling an auger with continuous helical flights into the ground to the required depth and pumping concrete through the hollow auger stem as it is subsequently withdrawn. BD piles are a novel, hybrid pile which use specially designed rotating augers to displace (or partially displace) the soil before the introduction of concrete in a similar manner to CFA piles. The construction methods of CFA and BD piles are described further in sections 2.4.1 and 2.4.2 respectively. The market for these pile types and industry trends are described in section 1.2.1.

1.1.2 PILE DESIGN

There are four generally accepted approaches to determine the maximum applied load that can be resisted by a pile. These are used to a lesser or greater extent depending upon national standards, local practices, soil type and the site investigation data available. These are the total stress approach, effective stress approach, empirical correlation with *in situ* tests and the application of energy methods.

The calculated ultimate resistance of a pile, Q_u, is the sum of two terms

$$Q_u = Q_b + Q_s \qquad \textbf{Equation 1.1}$$

where Q_b is the base or end bearing resistance; and Q_s is the frictional shaft resistance. This is shown in Figure 1.2.

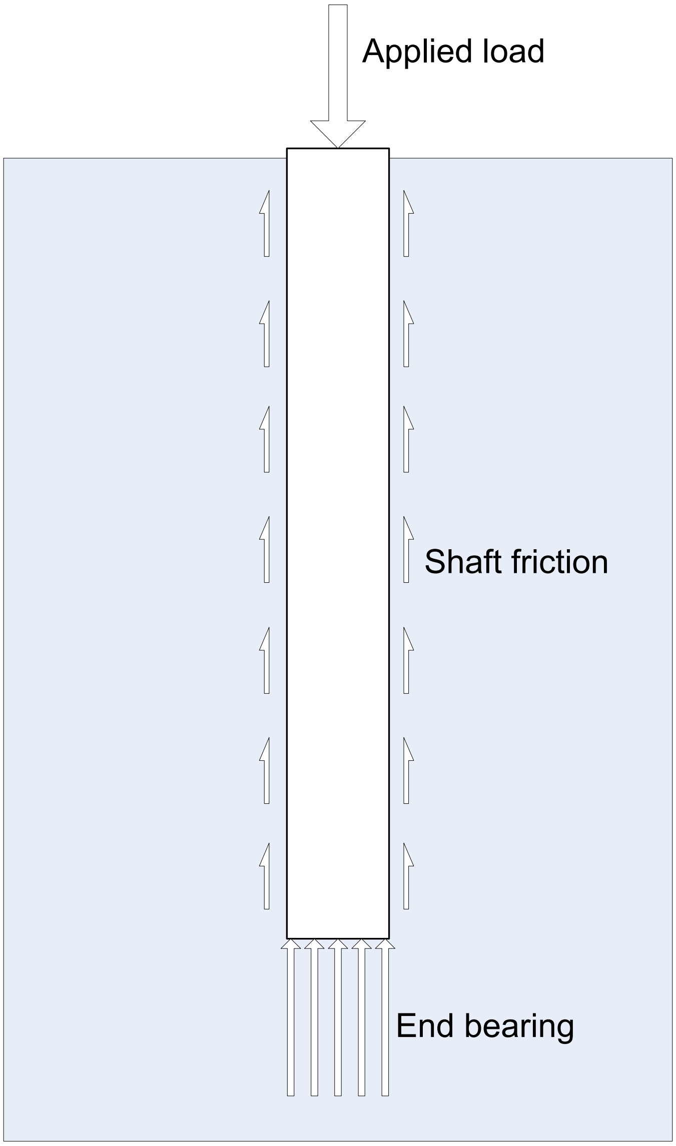

Figure 1.2. Calculation of the capacity of a pile.

In this research, piles constructed in fine-grained soils using CFA or BD piling techniques are considered. Design of such piles would typically follow the total stress approach, described below.

The end bearing capacity, Q_b, is given by

$Q_b = A_b N_c c_b$ (kN) **Equation 1.2**

where A_b is the area of the base of the pile, N_c is a bearing capacity factor and c_b is the undrained shear strength of the clay at the base.

The shaft capacity is given by

$Q_s = A_s \cdot c_a$ (kN) **Equation 1.3**

where A_s is the area of the shaft in contact with the soil and c_a is the average adhesion, or frictional shear stress, between the clay and the pile shaft. The average adhesion is some fraction of the clay strength and can be written

$c_a = \alpha \cdot \overline{c}$ (kN/m^2) **Equation 1.4**

where the adhesion factor α, which has been developed empirically, is less than unity, and not necessarily a constant, and $\overline{c}$ is the average undrained shear strength of the clay along the length of the pile. The adhesion factor, α, is a function of the pile soil interface and the method of pile installation. Skempton (1959) suggests that α will be less than one because of the increase in moisture content of the soil during the pile construction and hence the lowering of the clay strength. Skempton proposes four explanations for the softening of the soils; these are a) water flowing out of the clay during boring, b) migration of water towards the area of stress relief, c) water added during boring, and d) water derived from concrete. In any pile construction these four factors may vary.

Skempton (1959) demonstrated N_c to be well approximated as 9; α is typically in the range 0.4 to 0.6 for bored piles in London Clay (Patel 1992).

The variables in pile design are listed in Table 1.1 with a brief description of the factors which control them. From this table it is possible to identify the critical nature of the interpretation of ground conditions.

Table 1.1. Variables in pile design.

Variable	**Factors affecting variable**
Area of the shaft, A_s	This may be larger than expected due to over drilling, or smaller than expected due to partial collapse of the bore (necking).
Area of the base, A_b	This may be larger than expected due to over drilling
Undrained shear strength at the base, c_b	The method of measurement, number of tests, interpretation of test results. Discussed further in section 1.1.3
Average undrained shear strength along the shaft, c_a	The method of measurement, number of tests, interpretation of test results. Discussed further in section 1.1.3
Bearing capacity factor, N_c	Ground conditions, pile shape
Adhesion factor, α	Ground conditions, presence/migration/addition of water during construction, stress conditions during construction, pile installation technique, method and time taken Interpretation of soil tests and of pile load tests used in back analysis.

1.1.3 GROUND CONDITIONS

Geotechnical analysis and design is dependent upon reliable knowledge of the ground conditions present at a site and how the proposed structure will interact with the soil. Information relating to the ground conditions is obtained from a site investigation. The design, execution and interpretation of the investigation is critical to the design process. The cost and disruption caused by unforeseen ground conditions following inadequate investigation far outweigh the additional costs associated with a competent investigation (Egan, 2008).

Site investigation consists of a primary consideration of existing data (the ‘desk study’) and carefully planned intrusive investigation (the ‘Ground Investigation’) followed by analysis and interpretation. The standards for site investigation in the United Kingdom are described by the British Standards Institute in BS 5930:1999 (BSI, 1999) and BS EN1997-2:2007 (BSI, 2007).

For pile design it is necessary to have a measure of the resistance that can be provided by the soil. This would typically be the shear strength (for fine grained soils) or the angle of shearing resistance (for coarse grained soils).

Ground investigation is by its nature restricted to only a finite number of locations and depths. Test results are specific to the location of the test or sample. Interpolation of the test results is necessary to interpret the ground conditions across the site. This should take account of the variation that occurs and the likely deviation from the test results. Experience and existing knowledge can be incorporated into the geotechnical interpretation, a process which is often carried out in a subjective manner. In a typical example, strengths are plotted against depth and a line is selected to represent a conservatively chosen average for the site. An example is shown in Figure 1.3.

The degree of conservatism applied in selecting the average line is subjective and can be based on local experience. The line does not necessarily represent an arithmetical average but may be manually selected by eye (LDSA, 2000).

In this research, the characterisation process, with the allowance for inclusion of existing knowledge and experience is considered in an objective, quantitative manner within the framework of BSEN1997-1:2004 (BSI, 2004). This is described in section 2.2.

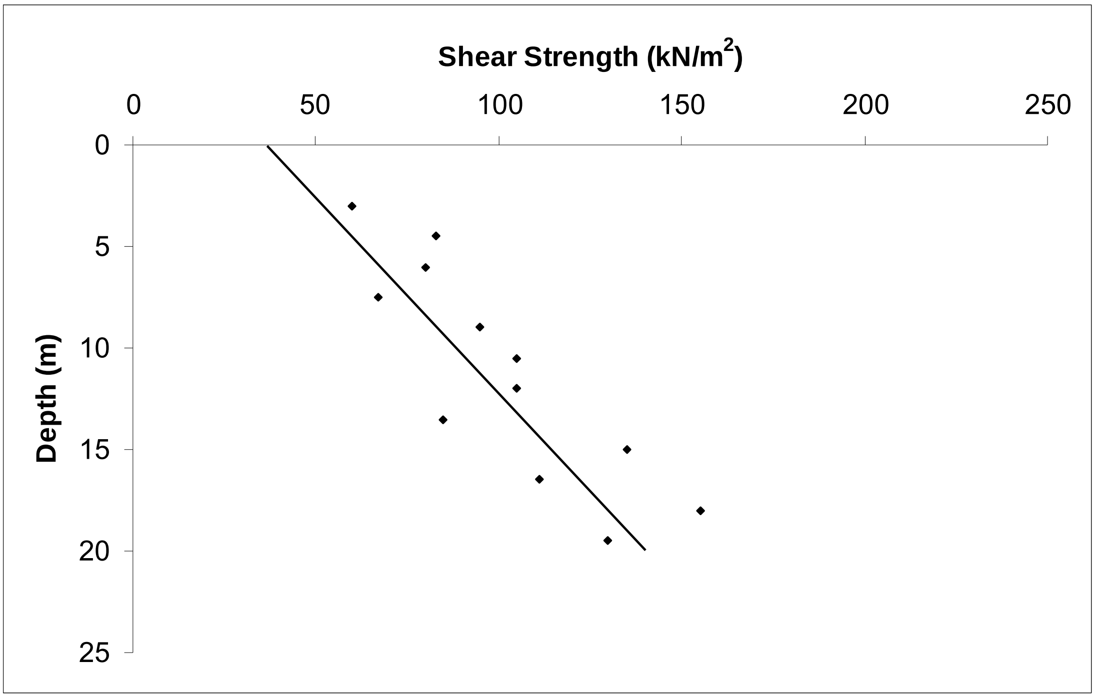

Figure 1.3. A typical plot of shear strength versus depth with a conservatively chosen mean design line.

1.2 THE CONTEXT OF THE RESEARCH

1.2.1 MARKET TRENDS

The market for piles was estimated to be worth £403.9M in 2007 (MDB Industrial, 2008) and has shown consistent growth over recent years (8% in 2007). The market can be sub-divided into large (>600mm) and small diameter piles. Rock and Alluvium's operations are predominantly in the small diameter sector which was worth £187.9M in 2007; this sub-sector is dominated by CFA piles which accounted for £157M.

The consistent sustained growth in this market makes it an appealing and hence competitive sector in which to operate. Contractors operating in this area therefore have an interest in research and development which can lead to a competitive advantage. Small margins have traditionally limited research and development operations in this sector but it is anticipated that this will increase in forthcoming years (MDB Industrial, 2008). This is in line with an anticipated workload increase reported by 61% of geotechnical companies (Ground Engineering, 2008).

There is also increasing interest in novel piling techniques such as BD piles because of their low vibration and low spoil production. These techniques are now offered by several contractors, all of whom carry out their own research and development to advance their own proprietary systems. The use of BD piles has increased such that they are now specifically included in the latest revision of the Institution of Civil Engineers (ICE) Specification for Piling and Embedded Retaining Walls (ICE, 2007). Furthermore, there is growing awareness of environmental concerns and the sustainability of geotechnical engineering solutions. An example of this would be the total energy used to construct a foundation or the carbon released by this process, including the associated energy expenditure through transportation and materials production.

1.3 THE INDUSTRIAL SPONSOR: ROCK AND ALLUVIUM

The industrial partner and sponsor for this research was Rock and Alluvium. Rock and Alluvium is a specialist piling contractor which has been operating since 1963. In 1989 Rock and Alluvium became a part of the Galliford group (later Galliford Try). Galliford Try is a leading construction company which is listed on the FTSE 100 index of companies, and in 2008 was the third largest civil engineering contractor (New Civil Engineer, 2008) with an annual turnover of £1066M.

Rock and Alluvium operates as an independent business unit; with turnover in 2008 of £21M. It is the 15th largest geotechnical contractor in the United Kingdom (Ground Engineering, 2008).

Rock and Alluvium employs a team of geotechnical engineers and construction specialists that enables it to offer design and construct services for piled foundations. Due to the complex nature of pile design, and due to the influence of the construction process on the pile performance (discussed in sections 1.1.2 and 2.4 respectively), many specialist piling contractors provide services on a 'design and construct' basis. This allows the design to be carried out with the knowledge of the actual installation process that will be used and its effect on performance.

Rock and Alluvium offers piled foundation solutions using either CFA or BD piling techniques as well as secant and contiguous piled retaining walls. The design of piles is dependent upon an understanding of the pile construction process and the interpretation of soil conditions. The incentive of better understanding of these processes is consistent with the motivations and ambitions of Rock and Alluvium.

1.3.1 MOTIVATION OF THE SPONSORING COMPANY

Rock and Alluvium are committed to research and development of new processes and techniques which lead to better provision for the Customers' needs and increased Client satisfaction (Rock and Alluvium, 2008). Involvement in such research also provides an opportunity for Rock and Alluvium to obtain a competitive advantage over other specialist piling contractors. The CFA market is highly competitive and Rock and Alluvium have identified the need to be able to provide efficient design as key to winning work and hence maintain their market share and to achieve their target for expansion. This commitment to research and development has led to an interest in topics relating to the refinement and improvement of the pile design process (particularly of CFA and BD piles) and the development and advancement of piling techniques (principally the novel BD techniques). The market for BD piles is specialised but has been identified by Rock and Alluvium as an area of growth and opportunity.

1.3.2 BENEFITS TO INDUSTRY

There are general benefits to the wider industry of this research programme. The industry drivers that have been addressed in this research are;

- The need for quantitative, objective methods for incorporating experience in designs that are compatible with BS EN 1997-1:2004 (BSI, 2004).
- The state of confusion regarding types, benefits, capabilities and limitations of BD piles and lack of common language for discussion or comparison of such.
- The recognised ambiguity in the allocation of uncertainty in pile design which is further hindered by a lack of understanding of the sources and magnitudes of variability in pile design and construction.
- The need for design solutions that can provide for probabilistic analysis and comparison of solutions are increasingly required.
- The need for efficiency in pile design which is exacerbated by increasing materials costs.

The field of pile design and construction is currently in a period of change as not only new processes and techniques are developed (for example BD piles to solve some of the problems presented by contaminated brownfield sites) but also as the legislative landscape develops with the introduction of BS EN 1997-1:2004 (Geotechnical design: General Rules)(BSI, 2004) and BS EN 1997-2:2007 (Geotechnical Design: Ground Investigation and Testing) (BSI, 2007) commonly known collectively as 'Eurocode 7' and which introduce different design philosophies and notably load and resistance (partial) factored design. The design of piled foundations involves modelling of the soils and an application of engineering judgement. Within BS EN 1997-1:2004 (BSI, 2004) there is the mechanism for this to be conducted in an objective manner. Design approaches are required that can be used within this framework. There is an increasing requirement to quantify the risk, or probability of failure, associated with design thereby enabling a probabilistic evaluation of project costs; quantitative, objective design approaches and methods are therefore called for.

Bored displacement piles are in their infancy by comparison with other piling techniques, yet are in increasing demand from Clients seeking to reduce spoil handling and haulage (especially where contamination is present). There is little published data or scientific understanding of the processes undergone by the soils during and after construction and the effect that these have on pile performance. Discussion and evaluation of BD techniques and the performance of piles constructed in this manner is needed within the industry such that progression can be achieved.

1.4 AIM AND OBJECTIVES

The aim of this research is to enhance CFA and BD pile design techniques through better characterisation of ground conditions and increased understanding of the sources and magnitude of variability in pile construction.

This aim is met by fulfilling a number of objectives which are listed below.

1. To establish the state of the art in the design of CFA and BD piles through a review of the literature.
2. To develop quantitative methods for interpreting site investigation data for CFA and BD pile design in light of previous experience, hence allowing existing knowledge to be incorporated into design in an objective manner and to optimise design.

3. To develop a language and terminology for the classification of BD piles, to enable dialogue, discussion and comparison of different types of BD pile.
4. To investigate BD pile performance and capacity
5. To analyse the sources of variability in CFA pile design and installation and the influence on pile design using probabilistic comparison.
6. To investigate the energy to construct BD piles with comparison to CFA piles

Completion of these objectives allows recommendations to be made for industry practice regarding CFA and BD pile design.

1.5 STRUCTURE OF THESIS

The structure of the thesis is described below, describing the content of each chapter.

Chapter 1 presents the background to the research and its context. The Industrial Sponsor is introduced and the aim of the research and its supporting objectives are presented.

Chapter 2 is a review of the literature relevant to the research to define the state of the art in CFA and BD pile design and installation. Seven areas are reviewed; these are site investigation requirements, how site conditions are modelled and represented, London Clay material properties, the construction of piles, pile load testing, variability in the piling process and the application of statistical techniques to pile design.

Chapter 3 contains a review of research methodologies and discussion of those selected for this research with respect to how they met the research objectives.

Chapter 4 is a summary of the research activities undertaken, including a quantification of existing knowledge relating to shear strength of London Clay and subsequent application of this information to design through Bayesian updating, investigation into the variability of factors in pile design, development of a classification system for BD piles and assessment of the performance of and energy to construct BD piles

Chapter 5 presents the main findings and implications of the research. Contribution to existing practice, the implications for the industry and the impact on the Industrial Sponsor are discussed. There is also a critical evaluation of the research and recommendations for further research.

Appendices A-D contain the four academic papers. These are referred to directly throughout the thesis.

The various research activities and outcomes can be viewed in relation to the objectives set out above in Figure 1.4 as can the published academic papers (appendices 1-4).

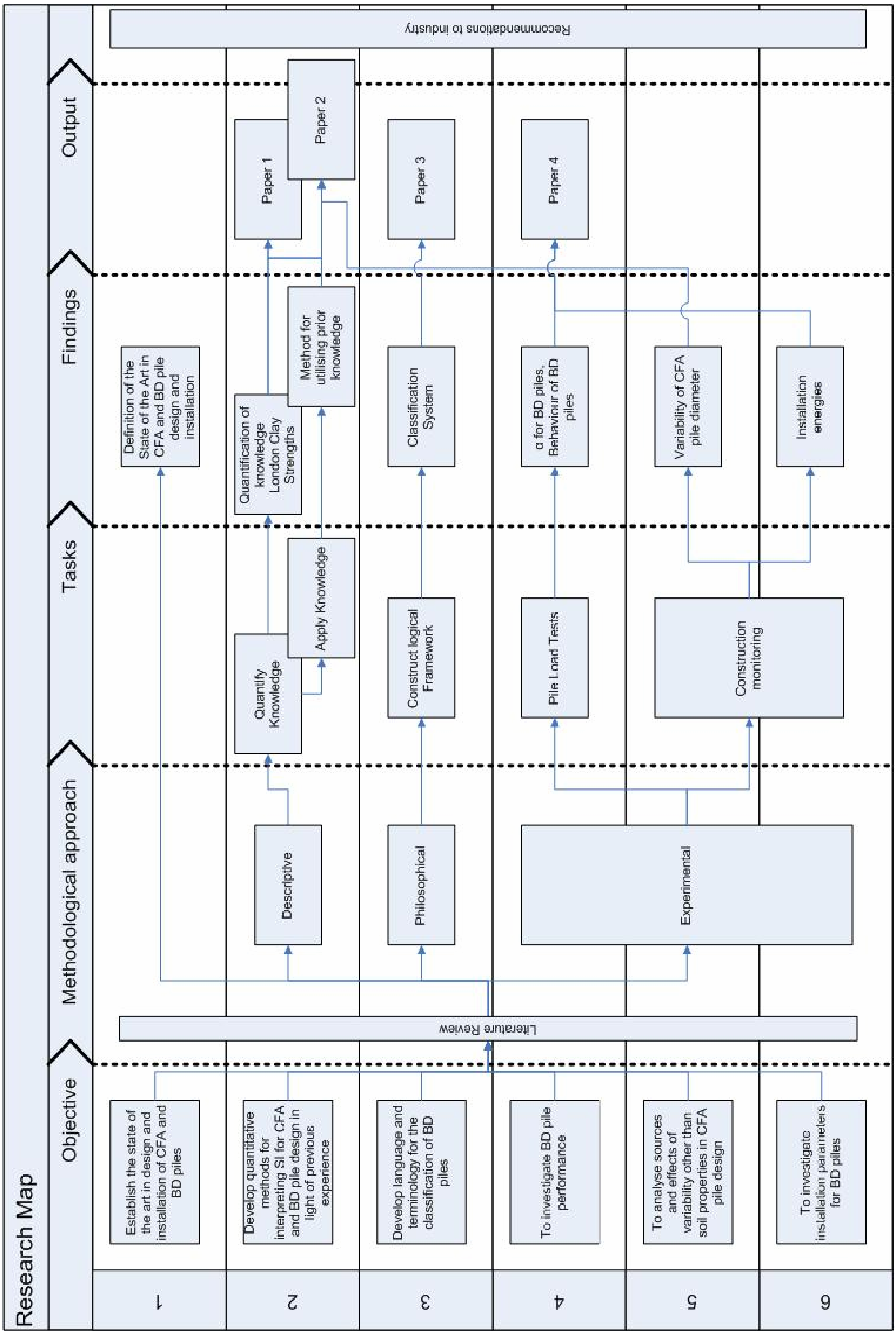

Figure 1.4. Research map.

2 LITERATURE REVIEW

The purpose of this literature review is to define the current state of the art in pile design and construction and to identify gaps in the knowledge worthy of further investigation. The direction of this research was influenced by the requirements of Rock and Alluvium, the Industrial Sponsor. The literature review therefore concentrated on CFA and BD piles, the main products of Rock and Alluvium. Furthermore, Rock and Alluvium have carried out a large amount of their construction work in London and the South East of England and expect this volume of work to continue in the future; much of the area of interest is underlain by London Clay. Historical data was available and considered suitable for detailed analysis. The literature review has therefore concentrated on the design and construction of CFA and BD piles in London Clay and part of the review concentrated on the lithology of London Clay. This chapter addresses the objective to establish the state of the art in the design and installation of CFA and BD piles in London Clay. The relevant areas of interest presented here are: site and ground investigation requirements for piling; the interpretation of site conditions from site investigation data for design; the presence of trends and variation in strength of London Clay; pile classification, the construction of piles using CFA and BD techniques; pile load testing and definition of capacity, and finally statistical techniques that can be used in the site modelling process.

2.1 SITE INVESTIGATION REQUIREMENTS

The design of piles requires the selection of suitable values for the properties of the soil. Determination of these values requires site specific knowledge of the soils. This is found through intrusive ground investigation, which forms part of the site investigation. Standards for site investigation are described in BS 5930:1999 (BSI, 1999) and BS EN1997-2:2007 (BSI, 2007). The density of sample points is not specified but becomes a function of the size, layout and nature of the site. Freedom is afforded to the designer of the ground investigation to allow for the degree of previous knowledge and anticipated variability of the soils.

It has been reported, however, that 30% of site investigations were described by contractors as ‘poor’ or ‘very poor’ (Egan, 2008). This is reflected in design codes (BSI, 2004) and guidance documents (LDSA, 2000) which require higher factors of safety to be incorporated into design, or more conservative selection of design parameters, where the ground investigation is recognised as being less extensive.

To apply the total stress approach to pile design, a measure of the undrained shear strength of the soil is required. This may be found in one of several ways including triaxial compression tests, standard penetration tests (SPTs), shear vanes and penetrometers (BSI, 1999). The preferred method is the triaxial compression of undisturbed samples (LDSA, 2000).

SPTs are often used to supplement triaxial compression test data. Where they are used a correlation is required between the blow count (N) and shear strength (c_u). Stroud (1974) recommend that a ratio of shear strength to blow count of between 4 and 6 is used depending upon the plasticity of the soil. Figure 2.1 shows how the ratio varies with increasing plasticity index. This relationship has been derived from a scatter plot which displays quite significant variation. This leads to uncertainty and the introduction of errors when correlations are used.

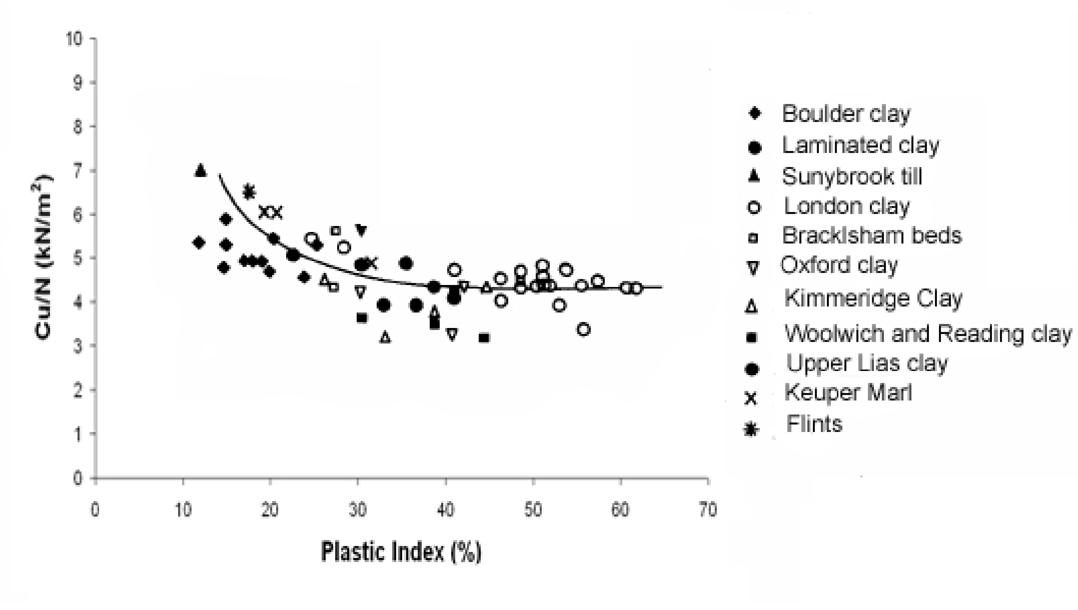

Figure 2.1. Correlation of shear strength with standard penetration resistance *(After Stroud, 1974).*

2.2 MODELLING SITE CONDITIONS

The process of building a model for the ground conditions and applying an interpretation is set out in BS EN1997-1:2007 (BSI, 2007) and has been mapped by Orr (1993) and Frank *et al.* (2004). Figure 2.2 shows the steps necessary to establish ground properties that can be used in design. The steps are described below and follow the development of soil property values through four stages – measured values, derived values, characteristic values and design values.

Ground properties are established primarily from ground investigation through tests carried out *in situ* and on samples (disturbed or undisturbed) taken for laboratory testing (e.g. undrained triaxial test to find undrained shear strength). These are referred to as the 'measured values' and represent the test results (following the application of any test related corrections, which are independent of further analysis). Where test results do not return a value which can be used directly in design, these need to be converted to 'derived values' by applying theory, empiricism or correlations such as the correlation of standard penetration resistance to undrained shear strength through the relationship proposed by Stroud (1974). The 'geotechnical parameter value' is an intermediate stage, defined by Frank *et al.* (2004). It allows for an assessment of the influence of the test such as a conversion from a property at a point to allow for the bulk behaviour of the soil. This is also the point at which refinements can be made based upon published data and general experience or prior knowledge.

The derived values (and geotechnical parameter values) are only representative of the sample and not of the overall distribution of values on the site. The values which describe the properties of the ground at the site, its associated variation, the nature and behaviour of the soil mass and its interaction with the structure are the 'characteristic values'. This characterisation of the site and the geotechnical problem is a vital step in the design and is where the engineer's skill and judgement is applied. The characteristic values are subsequently developed into to 'design values' by the application of safety factors.

The characteristic value is defined in BS EN 1997-1 (BSI, 2007) as a cautious estimate of the value affecting the occurrence of the limit state. The characteristic value must take account of the inherent variability of the soil, measurement errors and the extent of the zone governing behaviour (Dixon *et al.,* 2002). There are numerous sources of variation, these have been summarised by Uzielli *et al.* (2005) who concluded that variability can be divided into three classes; inherent soil variation, measurement error, and transformation uncertainty. These deal with the variability of the soils and the calculations applied to soil properties. Beyond this it is necessary to quantify the variability in the pile construction. Characteristic values are considered to be equivalent to those referred to elsewhere as "moderately conservative values" (i.e. Padfield and Mair, 1984) or "representative values" (i.e. BSI, 1994) (Simpson and Driscoll, 1998).

Determination of the characteristic values requires site specific information, which is found through the ground investigation. There is also the scope for inclusion of previous experience and knowledge of the strata. This is usually currently done in a subjective manner. In order to facilitate quantitative or probabilistic design an objective approach is required and a clear need for further research is apparent. This need is addressed in this research. Furthermore, for such an approach to be applied, a quantification of the previous knowledge is required, in the case of this research this means quantifying the existing knowledge relating to the shear strength of London Clay.

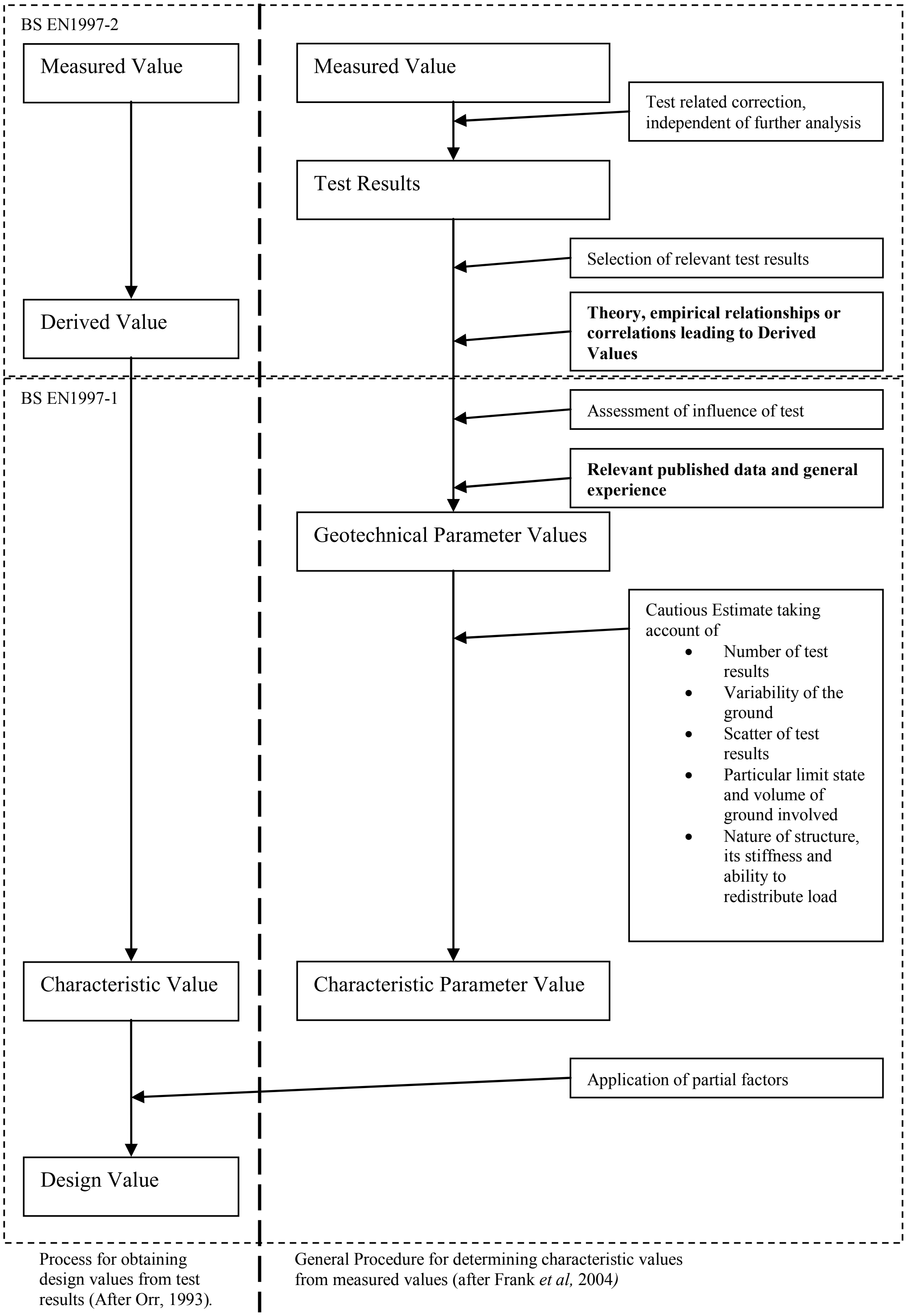

Figure 2.2. Steps in determining characteristic and design values.

2.3 LONDON CLAY

London Clay is a geological deposit which occurs beneath London, from Reading in the West and into Kent and Essex in the East as shown in Figure 2.3. London Clay is an overconsolidated, stiff, fissured clay of Eocene age. It is a blue grey clay that weathers to brown and contains fossils resembling modern warm water forms and can contain bands of concretions as well as pyrite or selenite crystals. It is reasonably uniform and its properties have been extensively investigated (Blyth, 1960). Some typical, indicative values of engineering properties of London Clay are shown in Table 2.1.

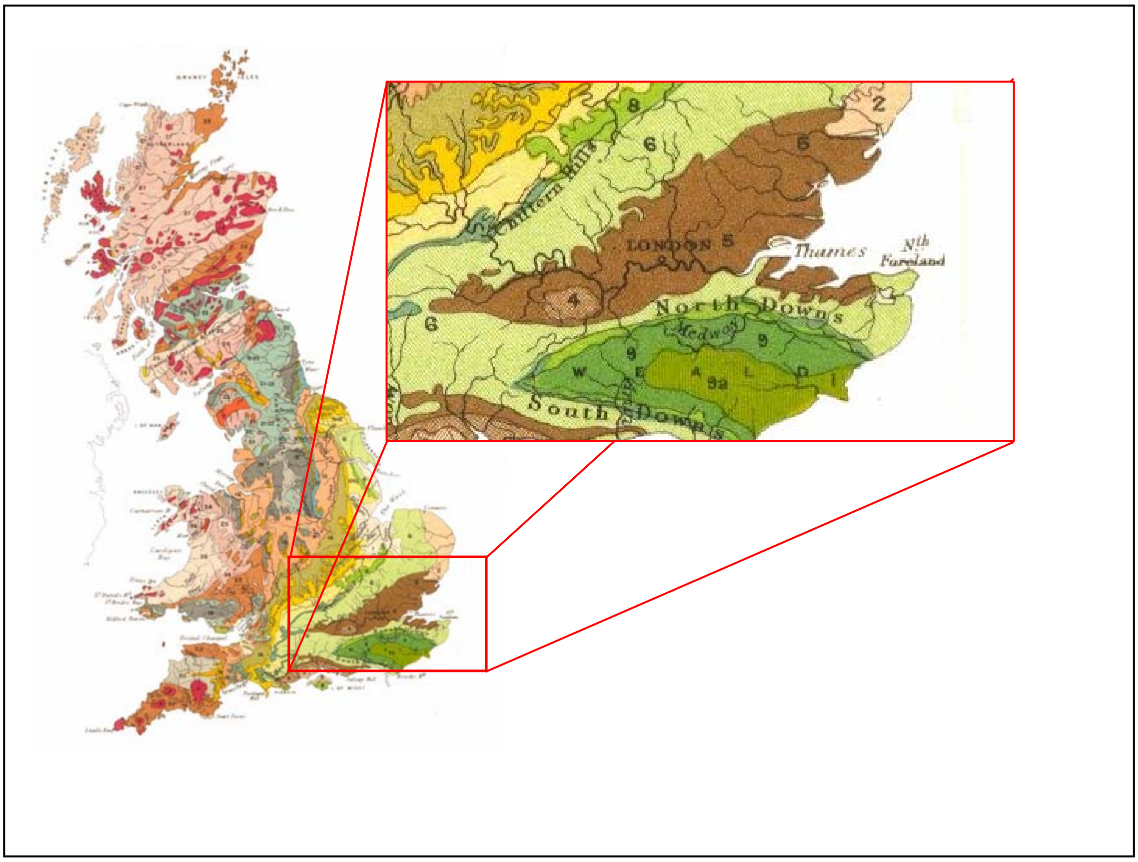

Figure 2.3. Geological map of the United Kingdom showing detail of the extent of the London Clay in the London Basin. (Map adapted from commons.wikimedia.org (2005)).

Skempton (1959) analysed pile load tests at ten sites in London clay to investigate the adhesion that could be mobilised between a loaded pile and the soil, this is described in further detail in section 2.3.2. Skempton also provided extensive data relating to the shear strength of London Clay. Figure 2.4 shows Skempton's interpretation of the relationship between strength and depth. Skempton (1959) observed some softening of the clay where there had been erosion and subsequent re-covering with alluvial material and made a correction to sample depth for samples from such areas based on the relative density of overlying materials but only to depths of about 15m below ground level. Conversely, Patel (1992) used only straight line regressions for mean shear strength profiles, this work is described in more detail in section 2.3.2.

Table 2.1. Typical engineering properties of London Clay.

Property	Typical value
Undrained shear strength, c_u (kN/m^2)	50-300 (increasing linearly with depth)
Plasticity Index, I_P(%)	30-50
Liquid Limit w_L (%)	60-70
Moisture content, w (%)	20-30

The results of Skempton (1959) are based on triaxial compression tests on 38mm diameter undisturbed samples. The average strength is derived from a scatter plot. There is variation around this average value which can lead to uncertainty. Variation is explored further in section 2.3.1. Modern site investigations utilise 100mm diameter undisturbed samples because of the increased likelihood of intercepting a natural fissure and hence obtaining a more representative, fissured, shear strength. Patel (1989) found the strength measured using 38mm diameter samples to be approximately 30% higher than when 100mm diameter samples are used.

2.3.1 VARIATIONS AND VARIABILITY

The strength of London Clay varies between sites and this leads to a need for individual consideration and site specific investigation (Patel, 1992). These differences can be partially explained by considering the history of the strata, the sequence of sedimentation, erosion, loading and weathering.

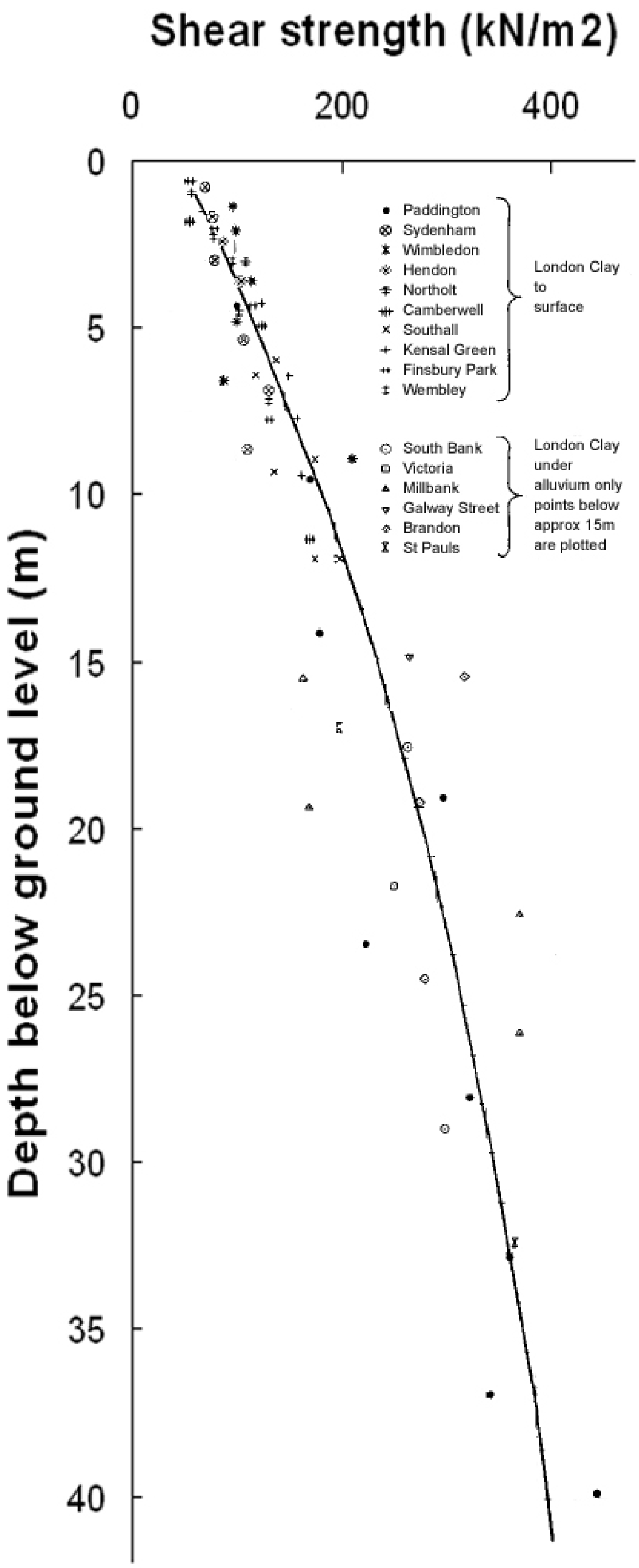

Figure 2.4. Average strength with depth of London Clay (modified from Skempton 1959).

King (1981) describes how the marine sedimentation of the London Clay was affected by changes in sea level during the deposition, a fall in sea level being associated with coarser grained material being deposited. The London Clay can be divided into five successive lithological units which reflect the cyclical changes in the depositional environment. Engineering properties can be discontinuous across the boundaries between the lithological units; hence there is merit in knowing the relative positions of the divisions and interpreting property profiles accordingly. In the case of undrained shear strength however there is no discernable discontinuity (Hight *et al*, 2002).

Furthermore, there is an increase in the thickness of deposit and a decrease in grain size from west to east consistent with being further from the historical coastline which is shown in Figure 2.5.

Post deposition, the London Clay has been subjected to a complex loading regime. Continuing deposition led to the London Clay being deeply buried (by up to 400m). Subsequent erosion of these deposits and of the London Clay during the late Tertiary and Pleistocene times has led to the surviving clay being heavily overconsolidated (over consolidation ratio of London clay can be greater than 20). The extent of this erosion has been estimated to be a thickness of between 150 and 300m (Skempton, 1961; Bishop *et al*, 1965; Chandler, 2000). In places, there has been further re-covering of the London Clay with river terrace deposits (sands and gravels) as well as alluvial deposits. The weathering of London Clay is apparent to differing depths across the London Basin. Where the clay has been covered with terrace deposits these have afforded some protection and the weathered zone tends to extend to only about 1m below the top of the clay; elsewhere the weathering may be apparent to depths of 10m.

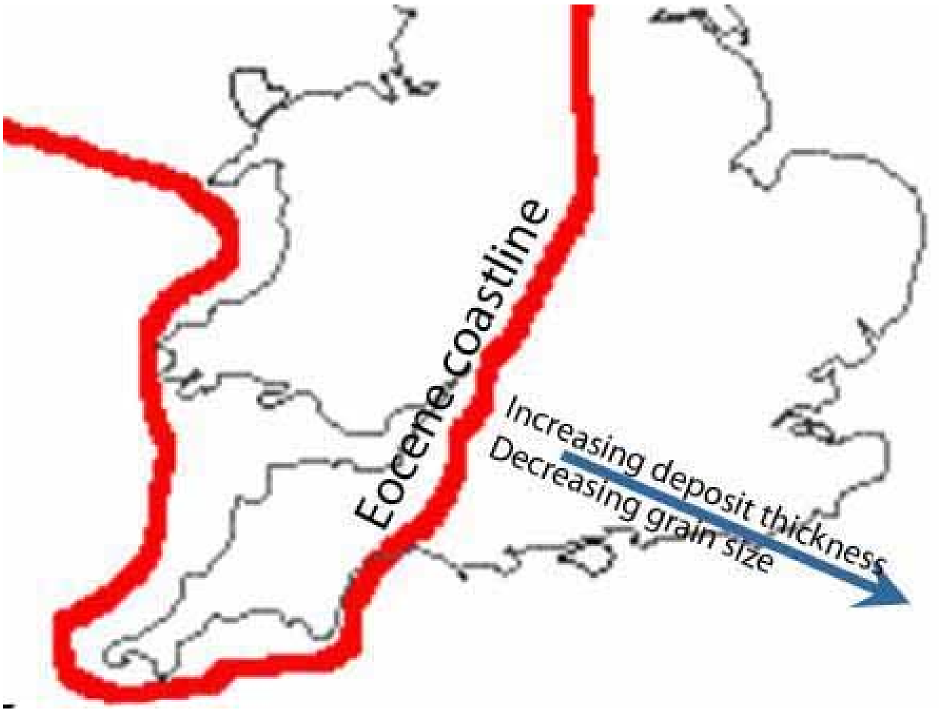

Figure 2.5. Outline map of the UK with the approximate position of the Eocene coastline overlaid. (Map outline adapted from about.com (2006)).

De Freitas (2007) suggests that faulting may also influence the continuity of properties across the basin. The location of faults (and the suggested network of minor faults that would result from the brittle behaviour of the clay) has not been mapped.

The remaining variation arises from the inherent variability of a natural material. Uzielli *et al.* (2005) showed how variation can be decomposed into an underlying trend and a fluctuating component. The fluctuating component represents the natural variation. Figure 2.6 shows an example of how values fluctuate around a trend, in this case for cone resistance in a cone penetrometer test. Measurement variation is additional to this.

The degree of this variation has been observed by several authors Schneider (1997) found the coefficient of variation, V_x, of the undrained shear strength of a soil to be in the range 0.3 to 0.5. Hooper and Butler (1966) found a coefficient of variation of 0.25 to 0.33 for triaxial compression tests carried out on 100mm samples of London Clay from a single site. Phoon and Kulhawy (1999) report a coefficient of variation of undrained shear strength in fine grained soils of 0.06 to 0.56, with a mean value of 0.33 based on 38 separate data sets.

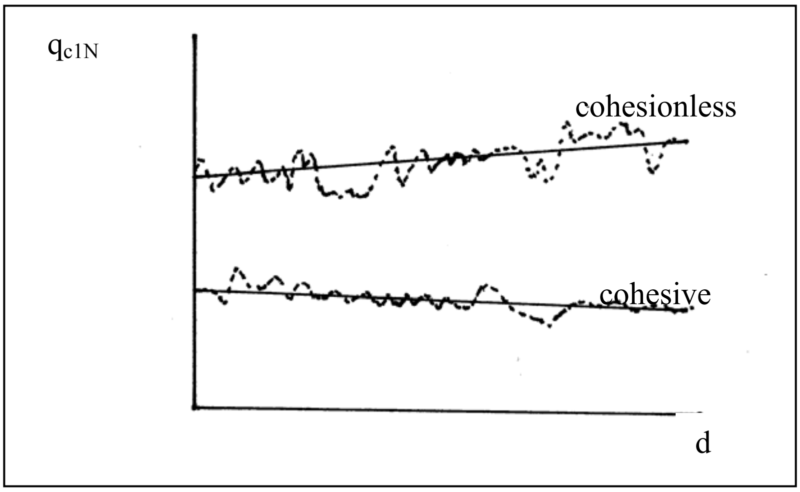

Figure 2.6. Plot showing the variation (dashed line) from the underlying trend (solid line) for the cone resistance (q_{c1N}) with depth (d) for cohesive and cohesionless soils. (After Uzielli, *et al.* 2005).

2.3.2 ADHESION BETWEEN PILES AND LONDON CLAY

A pile in cohesive soils transfers the majority of its load to the ground through the pile shaft/soil interface. The ability of this interface to transfer load is the adhesion, this has been described in section 1.1.2. The adhesion between pile and soil is a proportion of the cohesion of the soil, represented by an empirical factor, α .

Skempton (1959) used case studies of rotary bored piles constructed in London Clay to determine values for α in this material for bored piles. The key steps in Skempton's method are as follows;

- Find average cohesion over length of pile from site investigation data.
- Observe total pile capacity from testing.
- Calculate theoretical end bearing capacity.
- Calculate shaft capacity by subtracting end bearing from observed capacity.
- Find adhesion from shaft capacity and pile dimensions.
- Deduce α from calculated adhesion and measured cohesion.

Skempton's investigations (Skempton, 1959) found α for bored piles to vary from 0.25 to 0.61 and recommended 0.45 for calculations. Skempton suggests that α will be less than one because of the increase in moisture content of the soil during the pile construction and hence the lowering of the clay strength. Skempton proposes four explanations for the softening of the soils; these are a) water flowing out of the clay during boring, b) migration of water towards the area of stress relief, c) water added during boring, and d) water derived from concrete. In any pile construction these four factors may vary.

No account is taken of the variability of the data or the effect of extreme values. Skempton (1959) did however analyse the design of piles using the suggested value of α = 0.45. Comparison of predicted with observed capacity showed that the predicted load (for all but three cases) is within the range ± 22%. The extreme value shows an error of 35%.

Patel (1992) reanalysed the results of Skempton (1959) and those of Whitaker and Cooke (1966) and added additional data. Patel (1992) worked with clay strengths from 100mm diameter samples, the strengths were consistently 30% lower those from than 38mm diameter samples and hence the observed value of α was higher. Patel (1992) suggested revised values for α for bored piles which vary according to pile load test method used to find pile capacity; pile load tests are reviewed in section 2.5. Patel (1992) found α to be in the range 0.4 to 0.5 for maintained load tests (average 0.5) and 0.5 to 0.8 for constant rate of penetration (average 0.6). Patel (1992) suggested that the difference was due to the rate of loading and the time allowed for dissipation of pore pressures between load increments. Both Skempton (1959) and Patel (1992) concluded with average values of α over the pile length. Skempton (1959) showed that the average value of α may vary with length of pile but does not consider local variation with depth within a pile.

2.3.2.1 Adhesion Values for Design

London District Surveyors Association (LDSA, 2000) provided α values which are considered acceptable for design; it is based loosely on the work on Skempton (1959), Stroud (1974) and Patel (1992). Table 2.2 is taken from the LDSA (2000) guidance note and shows how a designer should select an α according to the site investigation data available and the extent of the pile load testing regime (and hence the likelihood of detecting defective piles). Adequate site investigation is defined (LDSA, 2000) as “adequate to understand stratigraphy and undrained shear strength profile of London Clay and samples taken using 100mm diameter metal open tube samplers to BS5930 (BSI, 1999)”. The guidance is designed to encourage better site investigation to be carried out by imposing a penalty on the designer when the site investigation is not ‘adequate’; this does however lead to the allocation of risk being split between the factor of safety and the adhesion factor. The adhesion factor takes on an element of risk and variability and no longer represents the adhesion alone.

The values for α listed in Table 2.2 have been successfully used in design practice for many constructions and are considered to be reasonable for design purposes by practising engineers and checking authorities. Due to the designed degree of conservatism and the allowance for risk included in the suggested values of α , they are not suitable for more detailed analysis.

Table 2.2. Guidance on the selection of α in London Clay (after LDSA 2000).

Adequate Site Investigation Carried Out?		Preliminary Pile Load Test?	Factor of Safety, F	**Adhesion factor, α**	Diameter of specimen for triaxial testing (mm)	Requirements for load testing of working piles (1.5 x working load)
Part A	No	No	3.0	**0.5**	100	No
Part B	Yes	No	3.0	**0.6**	100	No
	Yes	No	2.5	**0.6**	100	1% of working piles
	Yes	Yes – Constant rate of penetration	2.25	**0.6**	100	1% of working piles
	Yes	Yes - Maintained Load	2.0	**0.5**	100	1% of working piles

The values for α given by LDSA (2000) are intended for bored pile design. These values have also been used in practice for CFA pile design. Such application of these α values are somewhat conservative (Baxter, 2006) this is thought to be because there is less scope for soil softening during CFA construction compared to an open bored pile. Investigation into the adhesion factor for BD piles has not been conducted and there is a need for research in this field.

An alternative approach is to construct preliminary test piles to investigate the α value achieved with a particular technique on a particular site, under specific conditions which are repeated during construction. The initial selection of α can made using an estimate based on the work of Skempton (1959) or Patel (1992).

2.4 CONSTRUCTION OF PILES

Piles may be classified by their method of construction as reviewed in section 1.1.1. The focus of this research is CFA and BD piles. The construction methods of each of these pile types are described below.

2.4.1 CFA PILES

CFA piles are distinct by their construction method which has been described by many authors (e.g. Derbyshire, 1984; Brown, 2005) and appears to be fairly uniform between different contractors and even internationally. In summary, the piles are formed by drilling an auger with continuous helical flights into the ground to the required depth. As the auger is withdrawn, concrete (or grout) is pumped through the hollow stem of the auger. Following the concreting phase, reinforcement can be placed into the concreted bore. The process is shown diagrammatically in Figure 2.7 while Figure 2.8 shows a CFA pile under construction.

CFA piles are typically of diameters between 300mm and 1000mm and generally less than 30m in length. These restrictions are set by the practical limits of the plant available (Brown, 2005).

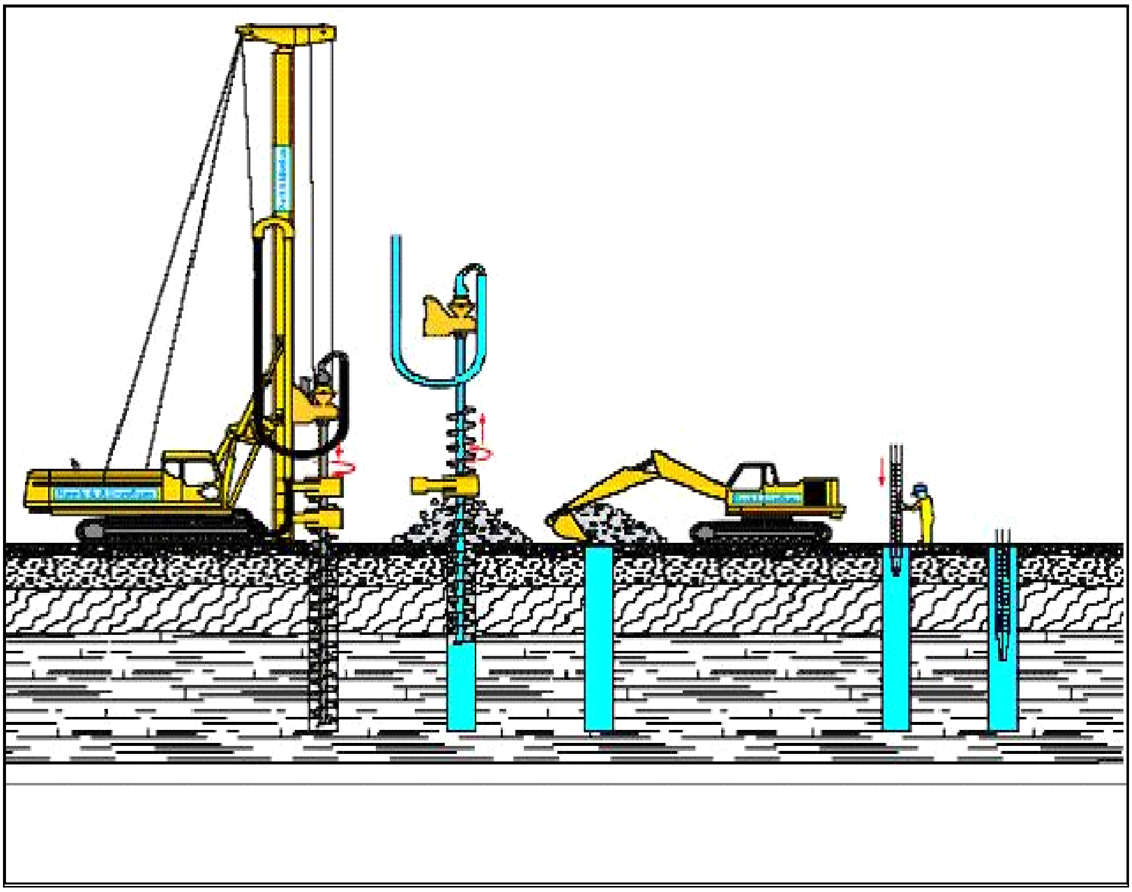

Figure 2.7. Schematic showing CFA construction (courtesy of Rock and Alluvium Ltd).

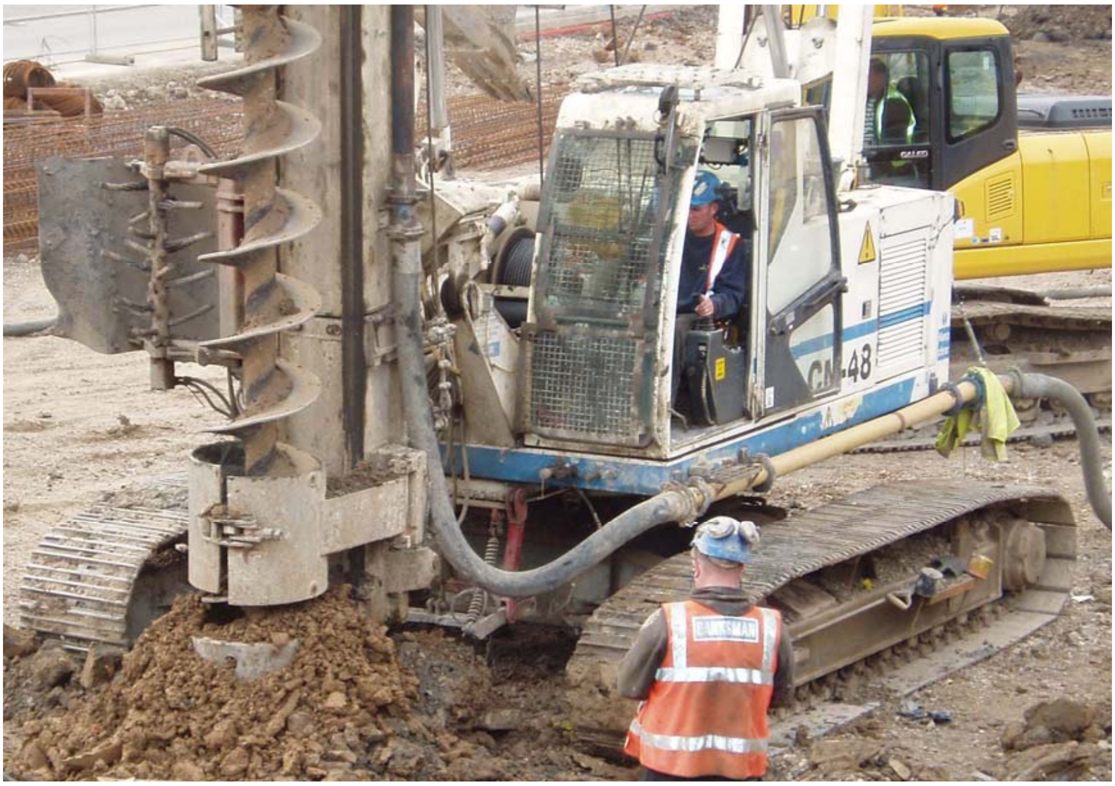

Figure 2.8. Construction of a CFA pile.

2.4.2 BD PILES

BD piling is a relatively new technique which has emerged as an economical pile construction method. BD piles are particularly favourable for contaminated land and brownfield redevelopments; this is because the risk (as well as the cost) of handling and disposing of contaminated soil is greatly reduced compared to replacement piling and noise and vibration are less than other displacement piling techniques (English Heritage, 2007). BD piling is also known as displacement auger piling (ICE, 2007) and drilled displacement piling (Brown, 2005).

The BD pile construction process is similar to that of CFA piles. The main difference is that the rotating auger displaces soil laterally (Hollingsworth and Imbo-Burg, 1992) rather than removing it.

The classification of ‘BD pile’ is a broad category partly because of the number of proprietary systems that exist. Early development of BD piles was brought about by technological advances in hydraulic drive systems for piling equipment which allowed sufficient torque and vertical force to be applied simultaneously to enable the lateral displacement of the soil and the insertion of the auger into the ground (Legrand, 2001).

Gradual refinement of BD piles followed and Van Impe (2001) identified three stages in the development of modern BD piling techniques. The first generation of BD piles used an auger that resembles a continuous flight auger with an oversize stem.

The second generation of BD piles used only a short flight to aid penetration as shown in Figure 2.9. This development reduces drag forces by eliminating the continuous sections of flight previously described. The flight displaces soil as the auger is screwed into the ground, in addition to the central stem section.

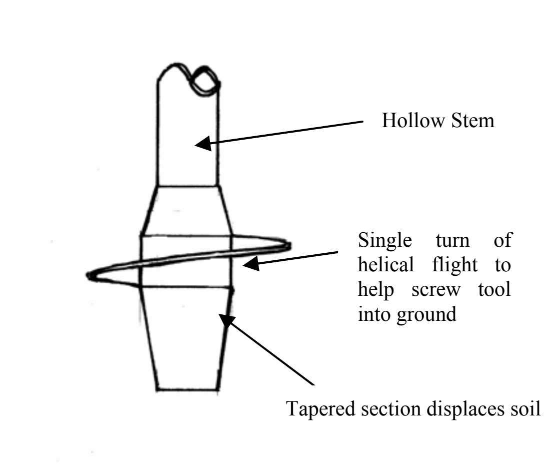

Figure 2.9. Second Generation BD pile auger.

Both the first and second generation of BD auger can be used in two different ways to provide differently shaped piles with diverse properties and advantages. The simplest method is to screw the pile into the ground and then extract it without rotation (or with continuing forward rotation). Such an approach produces a larger diameter cylindrical pile shaft. Considerable spoil is produced, although the lateral displacement will reduce this slightly.

The alternative is to reverse the rotation as the auger is withdrawn in order that it retraces its path. This produces an irregular shaped pile, with a cylindrical central stem and a continuous helical flange. Cross sections of the two piles are shown in Figure 2.10.

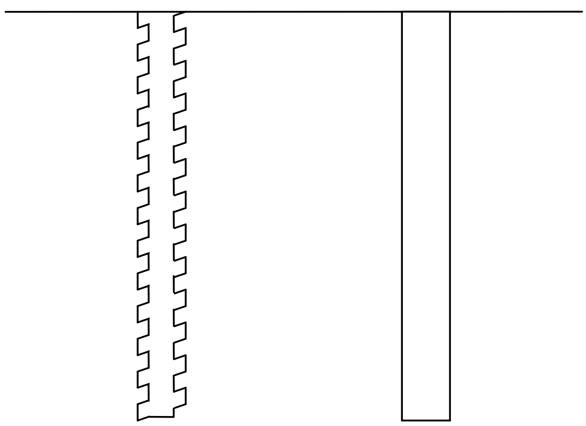

Figure 2.10. Pile cross sections. a) with helical flange, b) cylindrical.

The third generation of BD pile provided full displacement to the outside diameter of the flights, producing a cylindrical pile. A tapering stem displaces the soil up to the diameter of the central section; the tapered section has a flight attached which has been designed to pull the auger into the ground. The final section has a left handed flight that allows the bore to be maintained in loose ground as the auger is extracted. The pile is concreted on withdrawal through the hollow stem. Figure 2.11 shows a typical design of such a BD auger and examples in use are shown in Figure 2.12.

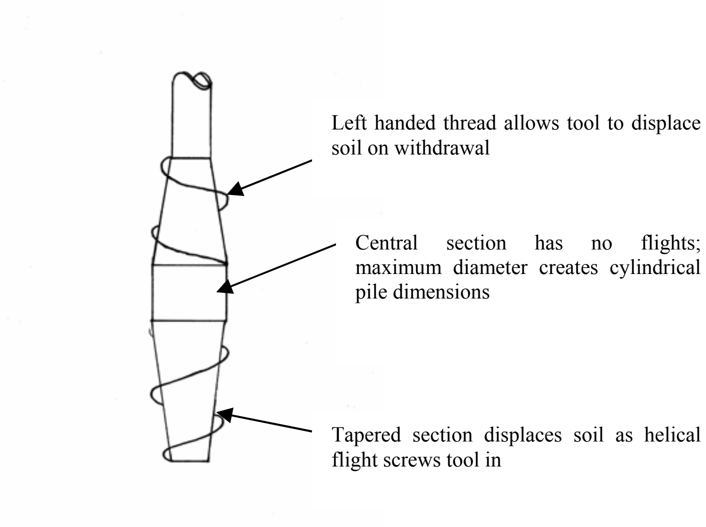

Figure 2.11. Third generation BD pile auger.

Figure 2.12. Third generation BD pile augers in use.

There is a wide variety of BD pile systems available in the market. These systems have subtle yet important differences in the methods of construction which may lead to different behaviour and capacity. Further research is required leading to the classification of BD piles, and a descriptive framework, to enable comparison and discussion and to enable particular types to be specified.

2.5 PILE LOAD TESTING

Pile load tests can be carried out to ascertain the capacity of a pile. This capacity can then be used to determine the design parameters such as the adhesion coefficient, α. It is necessary, however, to first define the capacity of a pile.

There are several values which can be said to represent the capacity of a pile. Fellenius (2001) conducted a review of the most popular and provides comment on their application and limitations. Fellenius dismisses failure criteria such as prescribed movements (set distance or percentage of pile diameter). These are often limits imposed from the perspective of the superstructure and are serviceability limits. Fellenius considers ultimate limits based on geotechnical behaviour.

The Davisson offset limit (Davisson, 1972) is widely used in North America but is predominantly intended for driven piles. It provides a capacity based on an estimate of the toe movement required to mobilise it and not the true capacity. The De Beer yield load (Originally put forward by De Beer (1967) and described in English by Fellenius (2001)) returns a value of the point where pile behaviour changes and where failure can be assumed to have occurred. Hansen (1963) proposed that the capacity of the pile occurs when the movement is four times that obtained for 80% of that load. Fellenius (2001) suggests that this often compares well with the perceived ‘plunging failure’.

Other methods described rely on extrapolation of the test data. They are the Chin extrapolation (Chin, 1978) and a variation on that theme, the Decourt extrapolation (Decourt, 1999). Both methods are useful to predict capacity where test loads have not approached the ultimate load. Such methods which rely upon extrapolation of data can be useful when the pile has not reached failure during the test, but caution is added that they may over predict particularly if settlements are low. Borel *et al*. (2004) have conducted more detailed analysis of the Chin method. The method does not allow for elastic shortening of the pile under load. It does however work for maintained load or constant rate of penetration tests and is not reliant upon soil properties. Borel *et al.* conclude that errors can be >30% and that this is a limitation of the test.

In order to ascertain the actual capacity of a pile it is necessary to carry out a load test. Two types of test are described by the Institution of Civil Engineers (ICE, 1996, 2007), maintained load (ML) and constant rate of penetration (CRP). For an ML test the load is increased in predetermined steps to a maximum value and then unloaded in a similar manner. A number of cycles may be conducted. Each load increment is maintained for a specified time, after which the settlement is measured. In a CRP test the load is increased at a rate that is controlled to produce a constant rate of settlement of the pile. Continual logging of load and settlement monitors the pile's performance. CRP tests tend to return capacities higher than ML test (Weltman, 1980). This is a result of the rate of loading during the test. CRP tests replicate conditions closer to the undrained case, while ML allows time between load increments for dissipation of pore water pressures (Weltman, 1980).

2.6 VARIABILITY OF THE PILING PROCESS

The performance and capacity of a pile can vary not only because of variations in the properties of the soil but also because of variations in the construction of the pile. The construction of a pile may affect the capacity in one of two ways. From consideration of Equation 1.3 it can be seen that the capacity is related to the dimensions (length and diameter) of the pile and the adhesion. The adhesion is affected by the construction process, as described in section 2.3.2. Cameron and Chapman (2004) identified the following factors that can influence the quality of bored piling on a particular site for a given pile type:

- The variability of ground conditions across the site and with depth.
- Knowledge of the specific ground conditions gained from a detailed site investigation.
- Contractor skill and experience with a given pile type under similar operating conditions.
- Supply of materials of correct quality.

- Appropriateness of construction procedures for the particular soil and groundwater conditions.
- Level of workmanship and site supervision during the construction.
- Intensity of the programme pressure on the piling work.
- Effects of ground movements and site traffic during hardening of concrete.
- Appropriate method and care in trimming pile head to final cut-off level.

Fleming (1995) described monitoring techniques which can assist in the control of CFA production and in avoiding poor quality construction. Piling rigs can be fitted with on-board computers to record, and adjust, speeds (rotation, insertion, and extraction), torque, vertical force and concrete pumping rates and pressures. These measurements can be used to calculate as built pile diameters, lengths and volumes and to measure the energy used to construct a pile. Instrumented rig monitoring and an associated automation of construction operations can lead to improved pile quality and reduce variation (Van Impe, 1998; Mure *et al.,* 2003).

The degree of variation in the adhesion has been reported in section 2.3.2. The remaining variation which is of interest is that of the dimensions (length and diameter) of the pile. The diameter and length can vary due to over excavation and inconsistency in the supply of concrete. The extent of this variation is unknown and requires further investigation.

2.7 STATISTICAL TECHNIQUES

A distribution of a variable, such as the measured strength of a soil obtained in a number of tests, can be represented by descriptive statistics (HELM, 2006). Usually a measure of location and a measure of spread are required. The distribution's location can be described by the arithmetical mean. The mean of a population is noted as μ, the mean of a sample is noted as $\bar{x}$.

The mean, $\bar{x}$, of a sample of n results of value x can be calculated using Equation 2.1.

$$\bar{x} = \frac{1}{n}\sum x \qquad \textbf{Equation 2.1}$$

The spread can be represented by the standard deviation. The standard deviation of a population is noted as σ, the standard deviation of a sample is noted as s.

The standard deviation, s, is calculated using Equation 2.2.

$$s = \sqrt{\frac{\sum (x - \bar{x})^2}{n}}$$ **Equation 2.2**

The standard deviation can be normalised with respect to the mean to give the coefficient of variation, V_x using Equation 2.3.

$$V_x = \frac{s}{\bar{x}}$$ **Equation 2.3**

Using the approximation proposed by Schneider (2001), shown in Equation 2.4, a characteristic value , x_k, can be obtained from the summary statistics.

$$x_k = \bar{x}(1 - \frac{V_x}{2})$$ **Equation 2.4**

The type of distribution of data is an important consideration; many statistical techniques assume a normal distribution. While most geotechnical parameters can be shown to have normal distributions (Sia and Dixon, 2007), Van Alboom and Menge (1999) warned that for some parameters log-normal distribution may be more appropriate. Hooper and Butler (1966), however, demonstrated that the variation of shear strength of London Clay (from laboratory test results) is well represented by a normal distribution. Figure 2.13 shows a typical normal distribution curve and that of a log-normal distribution.

2.7.1 GOODNESS-OF-FIT TESTING

The χ^2 distribution can be used to test the goodness-of-fit of a table of observed frequencies to a theoretical model, such as whether the observed values follow a normal distribution. To apply the test it is usual to have a total frequency of about 50 or more and a minimum class frequency of 5. Where class frequencies fall below this level, adjacent frequencies are combined. The steps followed to test the goodness-of fit are as follows:

1. The expected frequencies (E) are calculated under the null hypothesis that the distribution is normal
2. The observed frequencies (O) are put into similar classes.
3. The χ^2_{test} statistic is then calculated using Equation 2.5

$$\chi^2 = \sum \frac{(O - E)^2}{E}$$ **Equation 2.5**

4. Determine the degrees of freedom, v, for the normal distribution where the mean and standard deviation have been estimated from the observed frequencies $v = n - 3$ where n is the number of classes.

5. The χ^2 value for the chosen significance level (usually 5%) and calculated degrees of freedom is then found from tables.
6. If $\chi^2_{test} > \chi^2_{5\%}(\nu)$ then the null hypothesis is rejected, otherwise it is accepted.

2.7.2 BAYESIAN UPDATING

Interpretations of ground conditions based on sample distributions can be improved by inclusion of existing knowledge by application of Bayesian updating. Tang (1971) showed that by application of Bayes' theorem, the posterior distribution of a variable is proportional to its prior distribution (existing knowledge or experience of the general ground conditions) and the likelihood function (represented by the ground investigation results specific to the site). The resulting posterior distribution is a closer representation of the population, the statistical error is reduced and hence greater confidence can be assigned to the model.

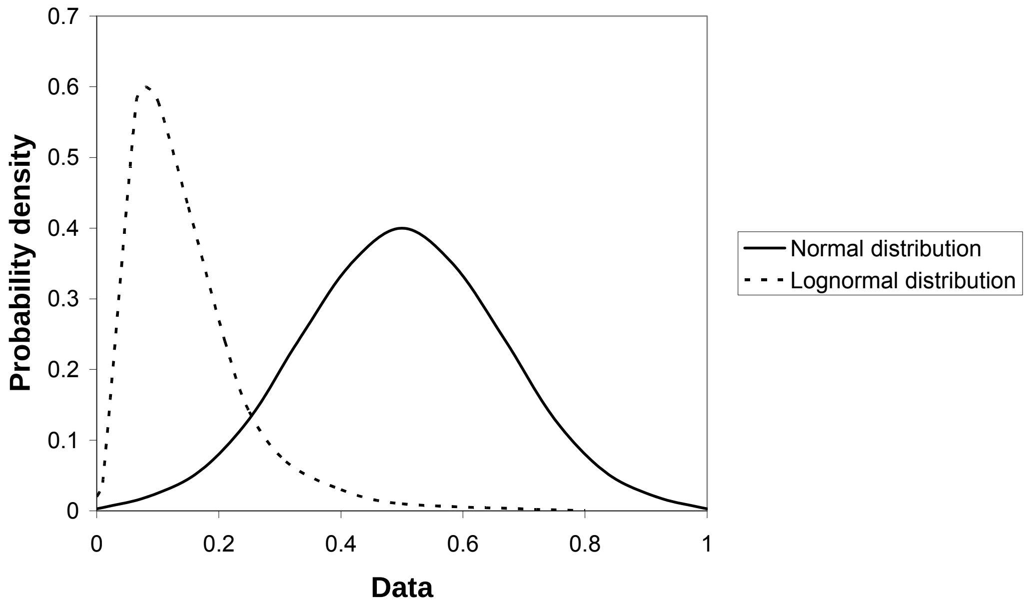

Figure 2.13. Example normal and log-normal distribution curves.

If both the prior distribution and the likelihood function are normally distributed, Tang (1971) showed that the posterior distribution would also be normally distributed with a mean value which is a weighted average of the mean values of the prior distribution and the likelihood functions. The weighting is proportional to the variances of the distributions.

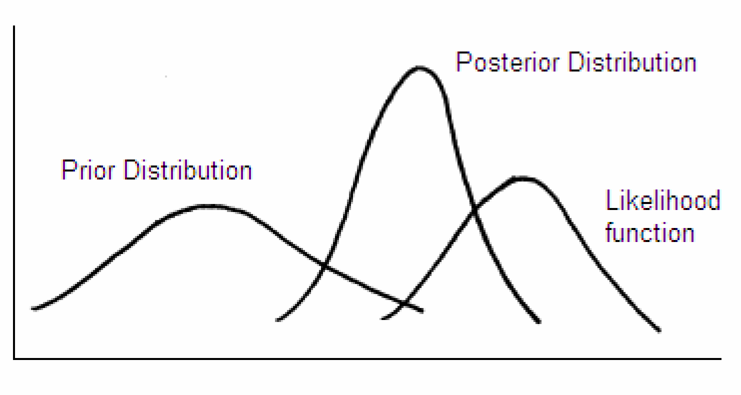

Figure 2.14. Relationship between the prior distribution, likelihood function and the posterior distribution (after Tang, 1971)

The mean μ'' and standard deviation σ'' of the posterior distribution can be calculated using Equation 2.6 and Equation 2.7 respectively. It is assumed that the variance of the sample σ^2 is known.

Posterior mean, $$\mu'' = \frac{\mu' \dfrac{\sigma^2}{n} + \bar{x}(\sigma')^2}{\dfrac{\sigma^2}{n} + (\sigma')^2}$$ **Equation 2.6**

Posterior standard deviation, $$\sigma'' = \sqrt{\frac{\dfrac{\sigma^2}{n}(\sigma')^2}{\dfrac{\sigma^2}{n} + (\sigma')^2}}$$ **Equation 2.7**

where the prior data has mean μ' and standard deviation σ' and the derived values for the site are represented by the mean $\bar{x}$ and standard deviation σ; n represents the number of data points.

2.7.3 PROBABILISTIC ANALYSIS

Probabilistic analysis provides a useful technique for comparison of alternative solutions. It can be conducted by calculating the respective probabilities of failure of a foundation. The advantage of this is that the overall cost of a solution with costs for contingency operations (weighted by the likelihood of occurrence) can be subsequently calculated, thereby allowing an assessment of the commercial risk associated with a design solution. Probability of failure can be calculated using simple reliability analyses such as the Taylor Series approach described by Duncan (2000). This consists of an essentially four step process;

1. Determination of the most likely values (MLV) of each parameter, and calculation of the associated factor of safety (F_{MLV})
2. Estimation of the standard deviations of each parameter
3. Computation of the Factor of Safety with each parameter increased by one standard deviation (F^{+}) and decreased by one standard deviation (F^{-}), from these values the coefficient of variation of the factor of safety, V_F , can be calculated using Equation 2.8

$$V_F = \frac{\sqrt{\sum_{i=1}^{n}\left(\frac{F_n^{+} - F_n^{-}}{2}\right)^2}}{F_{MLV}}$$ **Equation 2.8**

Where n represents the number of parameters.

4. The probability of failure can be found from tables using F_{MLV} found in step 2 and V_F found in step 3.

2.8 DISCUSSION OF LITERATURE REVIEW

Current codes and standards (BSI, 1999, 2004) allow for inclusion of previous experience during the modelling of site conditions and development of values of soil properties for design. Such an approach may be necessary due to the insufficiency of site specific data; this is more likely to be the case where the geology is well understood and ground investigations have only been carried out to confirm previous assumptions rather than gather detailed, new information. This process is, however, typically conducted in a subjective manner. There is a need to construct an objective method in which previous knowledge and experience can be used to complement site specific information. Bayesian updating offers a method for achieving this (Tang 1971, Schneider 2001). In the case of shear strength of London Clay (which is used for pile design) a method of summarising the data and allowing for the trend of increasing strength with depth is required. There is then a need to apply this theoretical technique to actual case studies to investigate its effects and value to engineering practice.

In order to apply objective methods of incorporating the data representing previous experience, it is necessary to possess a quantitative measure of that experience. The properties of London Clay and, of particular interest to this research, its strength have been described by authors such as Skempton (1959) and Patel (1992). Further information is required, however, relating to the variation of the population if the statistical techniques described in section 2.7 are to be applied. It is also necessary to demonstrate that all of the strength results gathered are from the same population and that the variations in the deposit described in section 2.3.1 do not lead to a number of smaller separate populations. Furthermore it is necessary to demonstrate that this data is distributed normally. In the literature the relationship between strength of the London Clay and depth has been described by straight lines (Patel, 1992) and curves (Skempton, 1959). Considering the degree of variation present, either approximation of the trend can be observed to be reasonable.

A variety of BD pile types exist, many of which can be described by consideration of the three generations of BD piles as described by Van Impe (2001). This classification system provides a description based on the historical development of the tools involved, additional descriptors are required to communicate the method of construction, finished pile shape and the associated pile behaviour under load. There is also inconsistency across the industry in the adoption of descriptive terms and classifications. There is a need to develop a framework which transcends the proliferation of proprietary terms currently used and provides an accurate, yet simple, explanation of the method of construction, shape of pile and which indicates the likely behaviour of the pile. Development of such a framework requires an initial proposition and then, critically, dissemination and discussion with other practitioners to facilitate adoption as a customary approach.

It has been shown in section 2.3.2 that the adhesion between a pile and the surrounding soil is related to the shear strength of the soil by a factor, α. Skempton (1959) attributed this to the softening of the soil during construction. The degree of softening, the nature of the pile-soil interface and hence the value of α is dependent upon a number of factors including the pile type, pile construction method, soil type and site conditions. The variation of α within the LDSA (2000) guidance results in a misleading approach that lacks rigour. The effect of the testing method and the degree of certainty in the ground conditions are included in the selection of α as shown in Table 2.2; these should be dealt with explicitly elsewhere in the calculation of pile capacity such that α represents only the adhesion.

The values suggested by LDSA (2000) for α in London Clay are based on bored piles and are not directly applicable to either CFA or BD piles. It is reasonable to assume that α

would be higher for CFA due to the support provided to the bore during construction, the quicker construction and the avoidance of the need to add water (or drilling fluids) during construction. It is reasonable to adopt the values of α suggested by LDSA (2000) for routine design as they would provide a conservative estimate; they are not, however, suitable for detailed analysis or for more accurate modelling and design. Investigations into the magnitude of α for CFA piles in London Clay have been conducted (e.g. Baxter, 2007). Such investigations have not been reported for BD piles and a need for further research exists.

Probabilistic design of piles is possible if the degree of variation of each of the parameters is understood. The variability of the soils can be studied as indicated above. The variability of the adhesion can be affected by differences during the construction process. The construction process and its repeatability can be studied through analysis of the monitoring data which is recorded automatically during construction. The adhesion, α, for bored piles in London Clay has been shown by Patel (1992) to vary within the range 0.4 to 0.5 (average 0.5) when maintained load (ML) tests are conducted and 0.5 to 0.8 (average 0.6) when constant rate of penetration (CRP) tests are conducted. The remaining variable is the dimensions of the pile. An understanding of how this varies is required to complete the knowledge of how pile capacity could vary. Pile dimensions can be estimated from the volumes of concrete supplied when constructing the pile.

2.8.1 Identification of Research Needs

From the literature review a number of gaps in knowledge were identified. These provided justification for the research conducted and described in this thesis. A summary of the gaps in knowledge that have been identified is presented below.

- There is a need to develop and implement objective methods for the inclusion of previous knowledge and experience in the modelling of ground conditions for design and analysis of piled foundations.
    - In order to utilise the above methods, a quantification of existing knowledge relating to the strength of London Clay, in terms of average values with depth and a measure of the dispersion about that average.
- A framework for the description of BD piles needs to be developed.
- Performance of BD piles in London Clay needs to be investigated such that the adhesion factor can be determined.

- An understanding of the variation in actual pile dimensions is required to permit probabilistic analysis of pile capacity.

3 METHODOLOGY

The purpose of this chapter is to examine the research methodologies available for this study. The main elements of this chapter are firstly a review of research methods, followed by a statement of the adopted methodologies, their suitability and benefits. The research strategy is presented and how the chosen methods are suitable to achieve the overall aim is addressed. The style and methods adopted for the research are then described and justified with reference to how they meet the research objectives identified in section 1.4. This chapter also demonstrates how the publications relate to the aim, the methods adopted and the findings.

3.1 REVIEW OF RESEARCH METHODS

Research can be classified using a scale at one end of which is 'pure' research and at the other end is 'applied' research (Fellows and Liu, 2003). Pure research is concerned with the discovery of theories or laws of nature and the development of knowledge whereas applied research concentrates on the end uses of knowledge and practical application of knowledge (Fellows and Liu, 2003). The close association with the construction industry that is fostered through this Engineering Doctorate programme lends itself to research of a predominantly applied nature.

To achieve the aim of the research project it was necessary to employ a research strategy, a way in which the objectives are questioned (Naoum, 2007). Research strategies can be either quantitative or qualitative (Naoum, 2007). Quantitative methods are objective in their nature and are designed to test a theory rather than to develop one, they are associated with the collection of factual evidence (Naoum, 2007). Qualitative methods are subjective in their nature and are used developmentally or to help define a problem (Naoum, 2007). They may be exploratory or attitudinal (Naoum, 2007). Qualitative methods are used without prior formulations and are often a precursor to quantitative research (Fellows and Liu, 1999).

3.2 ADOPTED METHODOLOGIES

A combination of quantitative and qualitative strategies was considered to be required for this research to achieve the different objectives. This dual approach can provide outcomes that are more mutually informative in relation to the aim of the research while maintaining the distinct contributions of each element of research (Fellows and Liu, 1999).

Within the frameworks of the research strategies a number of research styles are available, these can be used with either quantitative or qualitative strategies. In addition to a literature review, those which are relevant to this research are descriptive research (case studies), philosophical research and experimental research. These styles of research are described in the context of their application to this research in sections 3.2.1 to 3.2.4.

The interaction of the research methods and their relation to the objectives is summarised below and shown in the research map, Figure 1.4, (this figure is repeated as Figure 3.1 for ease of reference) along with the findings and outputs. The individual tasks are described in more detail in section 4 and the findings and the implications of the research are discussed further in section 5.

The literature review met the requirements of **objective 1** by establishing the state of the art for CFA and BD piles but also informs the investigations carried out to meet the remaining objectives.

Objective 2 required a descriptive and analytical approach that consisted of two tasks – the quantification of knowledge and its subsequent application. This work led to a quantification of existing knowledge relating to the shear strength of London Clay and development of methods for subsequent application of this knowledge.

Objective 3 required a philosophical methodology. The task was to construct a logical framework leading to a BD pile classification system.

Objective 4 followed an experimental approach. Pile load tests were conducted and analysed to obtain empirical design values and to develop knowledge relating to the performance of BD piles.
Objective 5 and **Objective 6** also employed experimental methodologies. The tasks involved monitoring and observing construction of CFA and BD piles to find the degree of variation in pile diameter and to investigate the installation energies of CFA and BD piles.

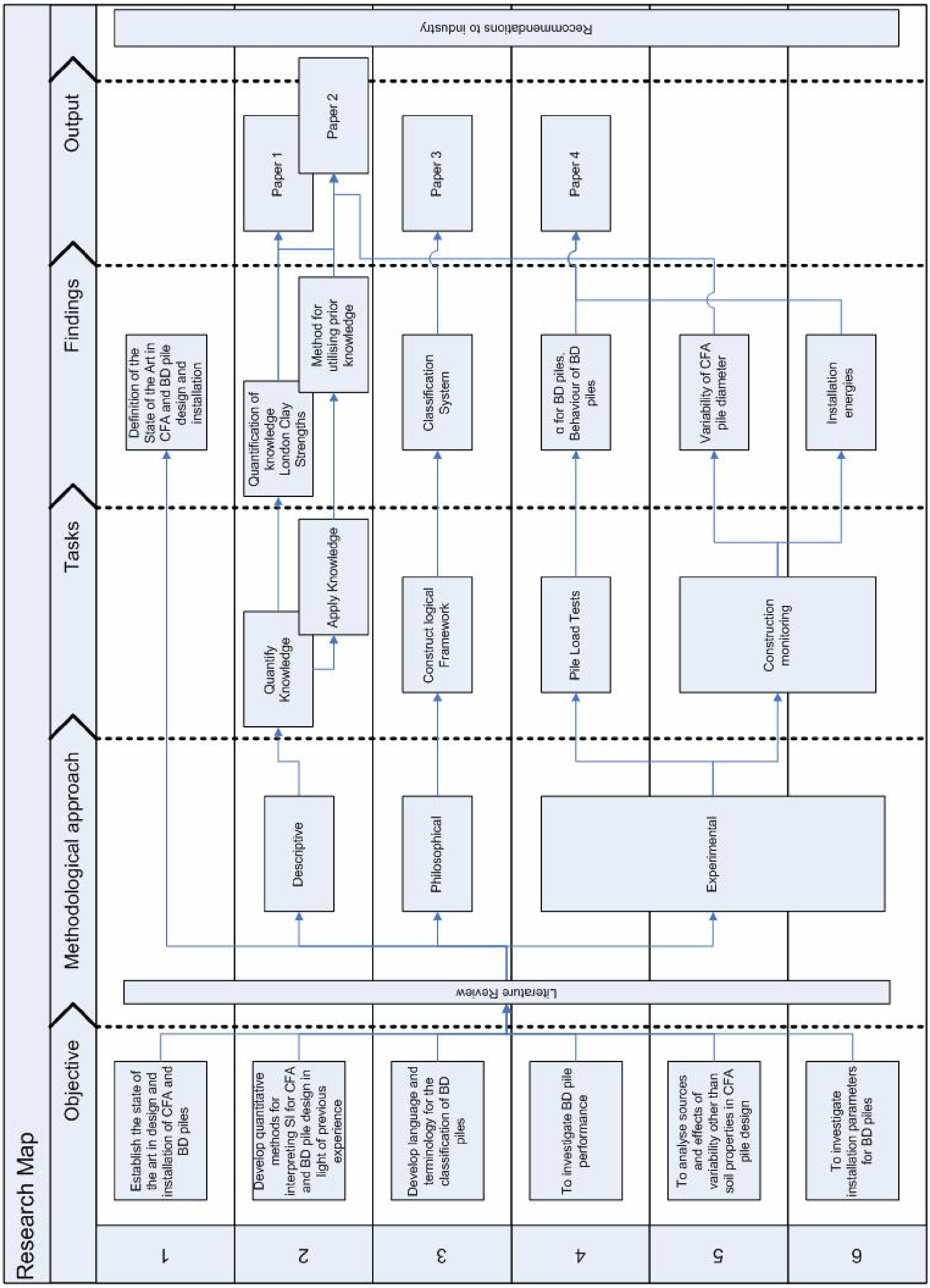

Figure 3.1. Research map.

3.2.1 LITERATURE REVIEW

Initial research took the form of a review of literature, this is an essential stage in conducting a research project (Naoum, 2007) and was used to identify and justify the aim and objectives of the project. Extension of this initial research also addressed objective 1 to define the current state of the art in CFA and BD pile design. The rationale for, and scope of, the literature review were discussed in section 2.

3.2.2 DESCRIPTIVE RESEARCH (CASE STUDY)

A case study or descriptive approach has been used to address objective 2. Descriptive research is a method that involves in-depth investigation of particular instances; the subject of the case study is often selected as being representative of the general situation (Fellows and Liu, 1999). The purpose of descriptive research is to systematically identify and record the aspects of a specific issue. (Fellows and Liu, 1999). The stages of descriptive research have been identified by Allison *et al.* (1996) and the process adopted in this research is set out below.

1. Examine and define the problem
2. List assumptions
3. Select appropriate subjects and source materials
4. Select, construct and validate techniques for gathering and classifying data
5. Make observations, analyse and interpret findings

There were two aspects to the descriptive research carried out to address objective 2; these were a quantification of experience and knowledge relating to the strength of London Clay and application of this data to modify the modelling of ground conditions. An extension of the ground modelling work to include pile design and probabilistic comparisons was also carried out. Each aspect is described in turn.

3.2.2.1 Quantifying Experience of Clay Strength

The problem has been identified as a need to quantitatively define knowledge and experience of the strength of London Clay and the associated variability. Experience is defined for the purpose of this research as information gained from previous site and ground investigations. A number of assumptions have been made in the collation of the data, these are listed below.

- Test results are from a single, normally distributed population. This has been demonstrated to be reasonable by inspection of the distribution in section 4.1.5.
- The ground investigations are representative of the population, a sufficient sample size has been drawn and that there are no time effects which would lead to future results being markedly different.
- Furthermore, it has been assumed that the ground investigations contain details of all tests conducted and that results have not been removed or omitted based on subjective judgements or expectation of likely results or otherwise.

In this research, London Clay from the London Basin has been selected as the subject. The Industrial Sponsor has a large resource of archive information containing site investigation reports and ground investigation data for sites that are underlain by London Clay. The current section describes the process of constructing, analysing and manipulating a database which adds to the current knowledge in addition to providing new information regarding variability. The process is summarised in Figure 3.2. Data has been sourced from Rock and Alluvium's archive of past and current projects. Information spans the period 2004 to 2007. The database contains records from 68 sites. A total of 234 boreholes are recorded from these sites. The boreholes and site investigations have been reported by 31 different contractors. A total of 947 quick, undrained triaxial compression tests have been conducted on undisturbed samples from the boreholes. Further, a total of 1080 SPT results have been recorded. Ground level has been used as a reference datum, this provides a simple, readily available datum but means that the depth to the clay may differ. The data contained in this resource is viewed as being typical of standard 'commercial' ground investigation rather than the extended, higher quality data which is often sought for research purposes. This is acceptable given the applied nature of the research and has the advantage of being relevant and appropriate to practice. The structure of the database assembled is described below.

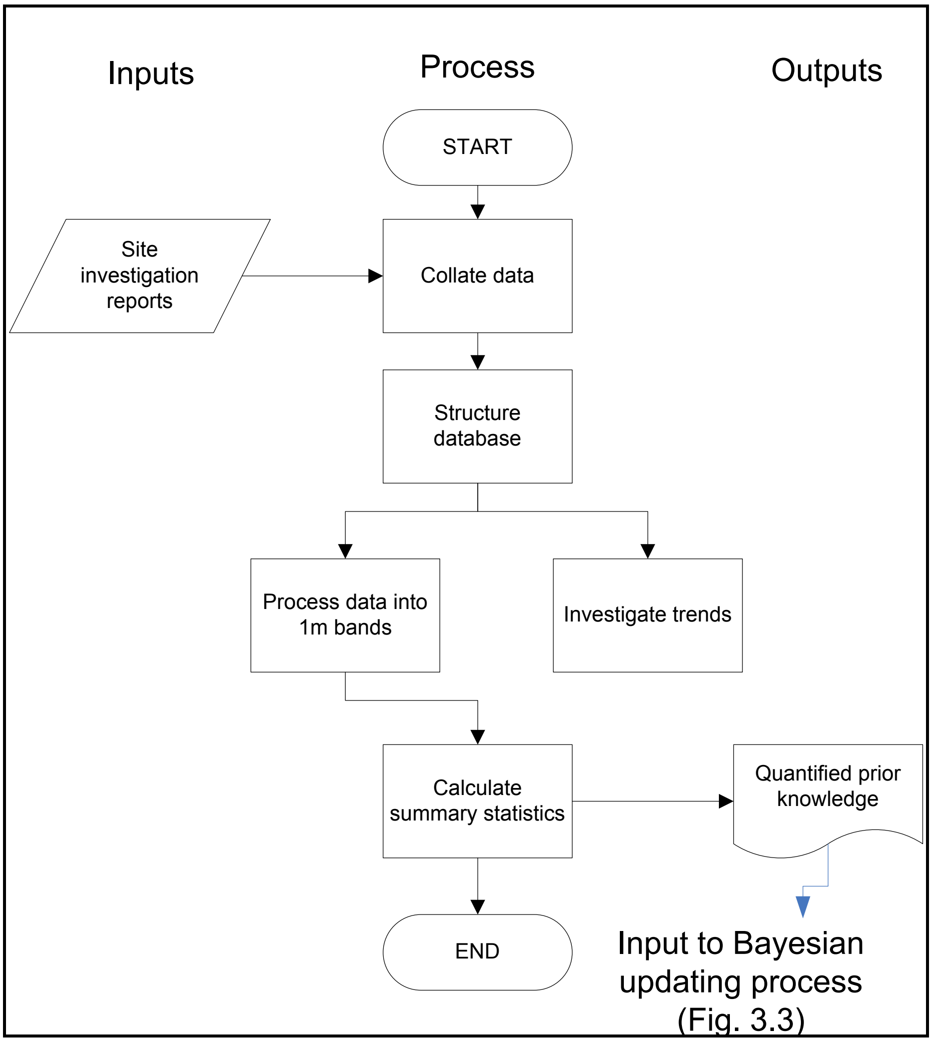

Figure 3.2. Process diagram for the quantifying of prior knowledge and experience.

The size of the database created was only limited by the data available. All suitable, London Clay sites encountered in the period 2004 to 2007 by Rock and Alluvium were used as these represent sites where the designs are based on current practice. Earlier results were not used as these were not as readily available and there was disparity in the method of recording information. Only summary results, not detailed information, were retained for many earlier sites. Furthermore, from 2004 onwards the data was stored in an electronic format that enabled faster, precise recall and processing of the information. The number of investigation points at each site and the number of samples tested from, and *in situ* tests conducted in, each borehole were determined by the site investigation contractor and/or client at the time as part of normal commercial practice.

The ground investigation results are recorded by site with subdivision where more than one investigation has been carried out on a site (by different contractors and/or at a different time).

Further division is made by borehole. The sites have been catalogued by Ordnance Survey National Grid coordinates to enable geographical location and evaluation. The reported depth to the top of the London Clay as reported was recorded. Distinction between the weathered and unweathered clay was also noted.

General trends were investigated from the data and investigation was carried out as to whether there were any sub-divisions of the data such as by degree of weathering, loading history or geographical location. Further processing of the data was, however, necessary for the subsequent use of this experience to modify models of ground conditions at a site.

The data in its raw format, or simply summarised by a regression line, cannot be easily employed as prior knowledge. A method of processing the data is required such that a mean and a measure of the distribution can be readily obtained for use in Bayesian updating. The method used in this research is to group the strength data by depth to top of sample and to calculate summary statistics for each group; bands of 1m thickness have been used for the grouping; depths have been measured from ground level. This was done to break down the increase in strength with depth into a set of discrete steps. Error will be introduced by the associated approximation of depth though this will have minimal effect on the trend of strength with depth. A 1m band thickness was selected as this was considered to represent a manageable interval that was consistent with the degree of confidence which could be associated with the actual location of a 200mm long triaxial test sample which has been taken from within an original sample of 450mm length.

3.2.2.2 Using Experience in Modelling the Ground

The second aspect of the descriptive/analytical research conducted as part of this project involved the utilisation of the gathered data. The problem has been identified as a need to improve the modelling of ground conditions by using previously gathered knowledge and experience (represented quantitatively by the data discussed in section 3.2.2.1).

As described above it is assumed that the data is from a single normally distributed population. Furthermore it is assumed that statistical methods are applicable and that the approximations that have been used are sufficiently accurate given the degree of variation observed. The data is assumed to contain all sources of variation and has not been subjected to previous selective refinement such as the removal of perceived outliers or inconsistencies.

This research is an application of the theories described by Tang (1971) and Schneider (2001). These methods have been selected as being relatively simple and practical in their application. This is considered to be important in encouraging future use and application. Such methods do not require additional skills to be learnt by engineers who choose to adopt them (such as detailed statistical analysis and manipulation).

Bayes theorem is applied to ground investigation results for a site, using the method described by Tang (1971), using the database described in section 3.2.2.1 as *a priori* data. The method of application is illustrated in Figure 3.3 and briefly consists of the following steps:

1. The derived values for the site data are found and used to generate summary statistics for the site; mean strength with depth is found via a linear regression of the derived values and the coefficient of variation of the site is calculated.
2. At the depth of each test/sample location in turn, the mean value is processed using Bayesian updating to produce the geotechnical parameter values. The sources of the *a priori* data are the mean value and the coefficient of variation for the corresponding 1m thick band from the database.
3. The Characteristic values are found using the approximation by Schneider (2001) described by Equation 2.4

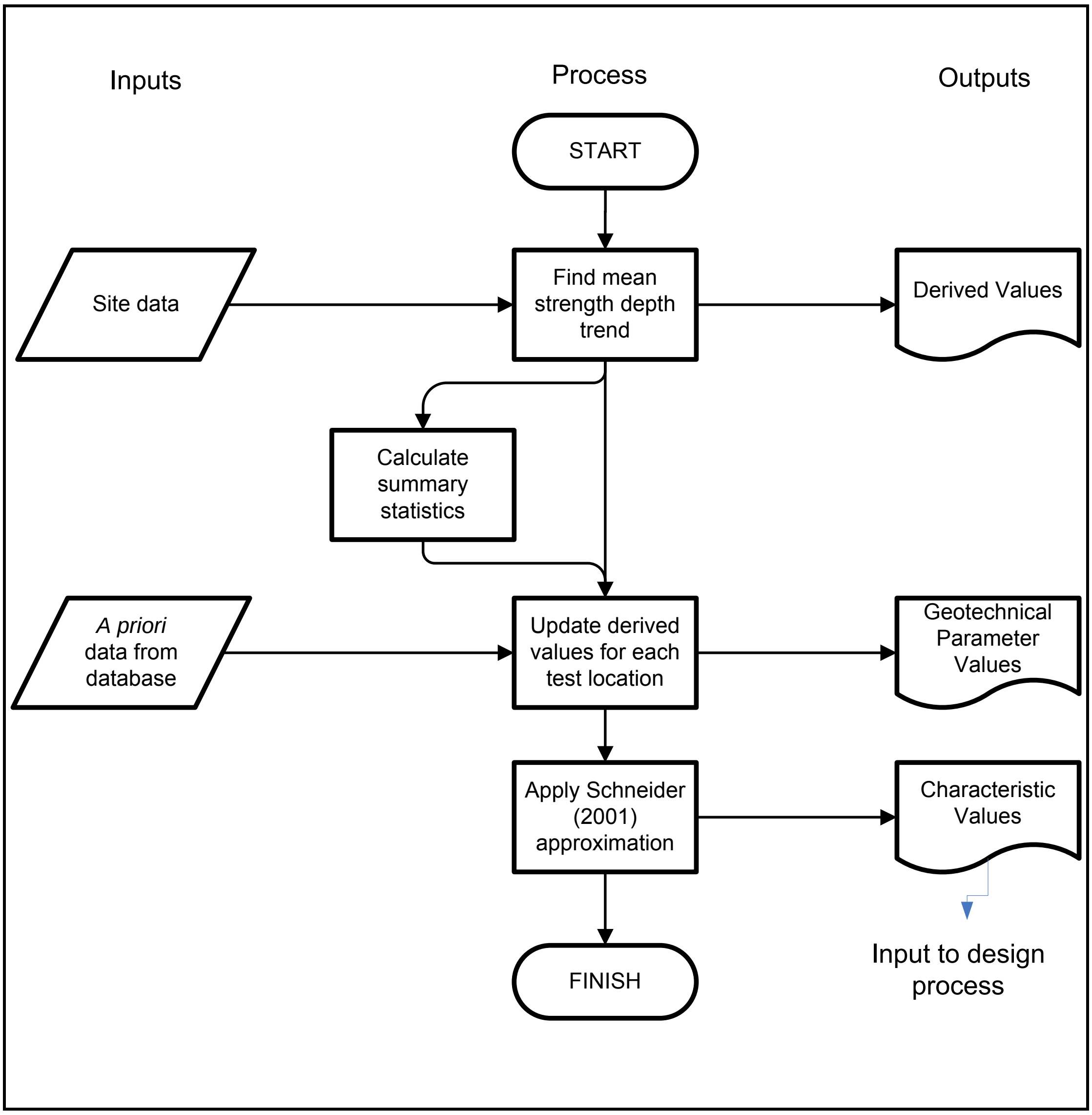

Figure 3.3. The process of including prior knowledge to ground model by application of Bayes Theorem.

This method has been applied to, and is illustrated by, case study examples as detailed in Paper 1 (Appendix A).

Paper 1 provides a framework for, and example of, the quantification of existing knowledge for use as *a priori* data in Bayesian updating. The shear strength of London Clay is used in the example. Bayesian updating is then presented as an objective, quantitative method for refining models of ground conditions to include previous knowledge as well as site specific data.

To investigate the merit of applying Bayesian updating, the case studies have been extended by way of probabilistic comparison of the risk of failure of a pile designed using characteristic values generated with and without the application of Bayesian updating. Probabilistic comparison is conducted by application of a Taylor Series comparison using the method proposed by Duncan (2000). A suitable pile design is proposed using the total stress approach. The probability of failure is then found following the Taylor Series approach by Duncan (2000). A second analysis is then carried out using the characteristic values and coefficients of variation from the updated data. In each case the pile dimensions are kept constant, only the interpretation of the ground conditions and the variation therein is changed. This is illustrated through case study in Paper 2 (Appendix B) and this is further extended to include dimensional variability of the pile in section 4.2.1.4.

Paper 2 builds on the quantification of existing knowledge and subsequent Bayesian updating through application of probabilistic analysis to the design process. The probability of failure is seen to reduce when the previous information is added in this way due to a greater degree of certainty in the ground conditions.

Sites have been selected which are considered to be reasonably typical of those where London Clay is the main underlying stratum. Data at the chosen sites are from multiple boreholes and test locations. They are real life examples considered to provide ample information for use as examples that can be followed by an engineer who wishes to understand and to apply such techniques to pile design.

3.2.3 PHILOSOPHICAL RESEARCH

Philosophical research is essentially concerned with the meaning of terms and language (Allison *et al.*, 1996). Although more often used in pure research and with a focus on linguistics, the process can be applied to the development of a new classification and description system and has been used in this manner to address objective 3. The steps in this process are:

1. Identification of the problem.
2. Application and examination of the concepts. This involves a search for definitions and applications.
3. Summation of meaning and applications to create a logical construct.
4. Consideration of the implications and applications to practice.

In this case the problem has been identified as ‘how can BD piles best be categorised?’. Current practice relies on proprietary and trademark names for systems and has no structure. It is hypothesised that categorisation can be by construction method and/or by the nature of the pile soil interface. The research was conducted by review of current literature and practice, the questions asked were primarily what methods of construction exist, how are they different, what shape is the resultant pile and how does it achieve its resistance to loading? This review of literature and practice is described in section 2.4.2.

Following the review process, it was possible to create a logical framework for the description of BD piles. The purpose of the classification systems was to provide information relating to the construction process and the likely behaviour and performance of the pile in a way that can be easily understood by Client organisations as well as Geotechnical Specialists.

Once the framework had been constructed, application was considered and validation was sought. The reasoning behind the development of the framework was to unify the previously disparate descriptive terms employed by different contractors and consultants for different, often proprietary, BD piling systems. It is necessary therefore to seek a consensus. The next important step therefore is dissemination of the logical construct and discussion of its applicability. This was achieved by publishing the proposed logical construct (Paper 3-Appendix C) and seeking feedback from interested parties.

3.2.4 Experimental Research

The purpose of experimental research is to observe causal connections (Allison *et al.*, 1996). By controlling some variables and manipulating others, experimental research seeks to predict outcomes from known inputs (Allison *et al.*, 1996). The steps involved in experimental research are:

1. Identification of the problem and formulation of hypothesis
2. Identification and selection of experimental and control variables
3. Experimentation
4. Analysis of results

Experimental research has been used in this project to address objectives 4 5 and 6.

3.2.4.1 Analysis of BD Pile Performance

Experimental research has been used to address objective 4, to investigate BD pile performance. The variables involved are those parameters of Equation 1.1, Equation 1.2 and Equation 1.3. In brief these are the ground conditions, the pile dimensions and the interaction between the pile and the soil. The adhesion achieved by BD piles in London Clay is the dependent variable under investigation. The soil conditions are the subject of the research described in section 3.2.2 and the pile dimensions are explored in section 3.2.4.2; the undrained shear strength, pile length and diameter are considered as fixed variables in this experiment. This research is concerned with piles in fine grained soils where the majority of pile capacity will be provided by shaft friction. The end bearing is taken to be a fixed variable.

Two sites for pile construction and testing were deemed to be suitable for the research. The sites were active construction sites on which Rock and Alluvium had been contracted to provide pile design and construction. The sites were selected on the basis that they were representative of typical conditions encountered where the main stratum was London Clay. The sites available for selection were restricted by practical considerations including the availability of space for test construction, associated cost, time and programme implications for the contracted works and the availability of suitable plant. Two piles were tested at one of the chosen sites and one pile at the other.

The pile load testing was carried out according to the procedures described by the ICE (1996) and followed the maintained load (ML) approach. The displacements of the pile were measured and logged by four electronic gauges at the head of the pile. The applied load was measured and logged using a load cell. Reaction for the applied load was provided by anchor piles constructed beyond the assumed zone of influence of the test pile and transferred to the pile using a rigid frame. It was not practically feasible to install strain gauges or other in-pile monitoring to measure the distribution of resistance along the shaft or between the base and shaft due to cost constraints, time limitations and difficulties with installation.

Where a clearly defined geotechnical failure can be observed from the load settlement curve this value has been taken as the pile capacity. Where this value has not been reached, an extrapolation is required; the method proposed by Chin (1978) is used. Use of an extrapolation, however, introduces some uncertainty.

The test setup and procedure follows accepted practice and this facilitates comparison with previous work for other pile types. A typical test arrangement is shown in Figure 3.4.

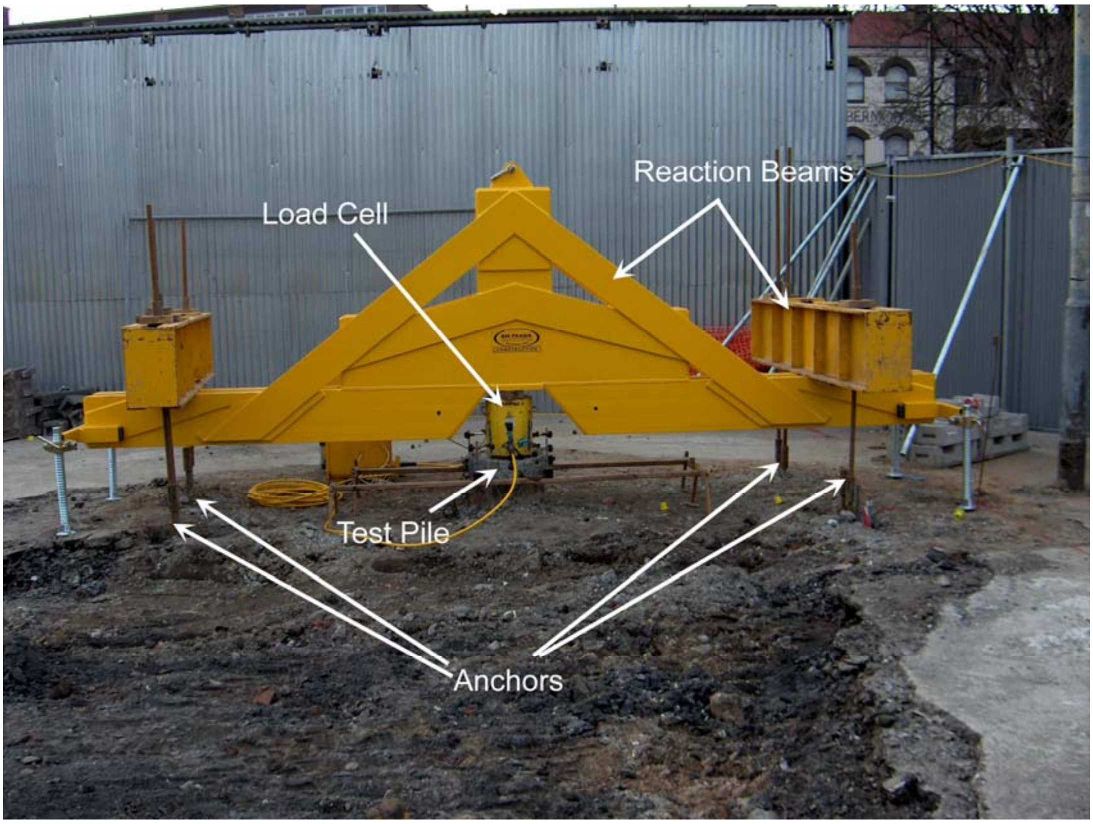

Figure 3.4. Pile load test arrangement (Photo courtesy of NDT Services Ltd).

The shaft capacity is found by rearrangement of Equation 1.1, as the actual end bearing capacity is not known it must be estimated from Equation 1.2, this has been demonstrated by Skempton (1959) to be a reasonable approach for bored piles in London Clay when N_c is taken as 9. The shear strength at the base of the pile is used for this calculation. Employing this value for the observed shaft capacity and by using the mean shear strength of the soil over the length of the pile, the empirical factor, α, representing the adhesion can then be calculated by rearrangement of Equation 1.3.

3.2.4.2 Variation of Pile Diameter

To facilitate probabilistic consideration of pile performance and to satisfy objective 5 it is necessary to develop an understanding of the magnitude of deviations in the pile dimensions.

The critical dimension to the performance of a pile in fine grained material is the diameter as the length can be controlled to well within the bounds of significance. The as-constructed length is used in calculations although in practice there was no difference between the nominal and as-constructed length. The pile diameter has an effect on the end bearing of a pile as the area is proportional to the square of the diameter, as the end capacity is a small proportion of the total capacity this is of less consequence. The bearing capacity factor remains unchanged as it is a function of the shape of the pile base which remains nominally circular.

Without exhuming a pile for measurement, it is very difficult to ascertain accurate, as-constructed dimensions. The actual as-constructed diameter of the pile at any depth can be estimated from the concrete volumes injected, this is measured by either flow meters in the concrete supply lines or by counting the concrete pump strokes and applying a correlation based on the volume supplied per stroke. Computerised rig logs provide profiles of concrete volumes injected against depth. These logs can be interrogated to yield average diameters achieved within a particular strata.

In this investigation the average diameter of the pile within the bearing strata will be calculated from such estimates and compared to the nominal value. The diameter at any depth can be calculated from the concrete volumes supplied. The piles used in the investigation were all monitored during construction using a proprietary computer system (manufactured by Jean Lutz SA). The system records concrete volumes for each 0.08m of pile length. These volumes are considered to contribute to 0.8m of pile length to either side. Calculation then yields an assumed profile for the pile.

Seven piles at seven different sites have been selected for analysis. These sites have been chosen for several reasons. Primarily they are of interest because the pile construction has been monitored using the same technology. Each site consists of predominantly London Clay (overlain by some fill material). A range of pile diameters is represented. The piles selected are expendable test piles and therefore further information is available regarding the pile behaviour. The test piles were representative of the typical pile dimensions and ground conditions encountered at each site. It is considered that, for the piles used in the investigation, the capacity is provided entirely within the London Clay, the capacity provided by overlying materials is much less than from the clay due to the shallow depth and low adhesion of these soils. Therefore the average diameter is calculated only over the length of pile embedded in the London Clay. Capacity is then calculated using the nominal (design) and the estimated actual diameters. For both calculations at each site, the same soil strength profile is adopted, although variation will have been present across the site, the interpretation of soil conditions uses multiple sample points and allowed for such variation.

The tests at four of the sites are further described in Paper 4 (Appendix D).

3.2.4.3 Installation Energy of BD Piles

Experimental research has been used to address objective 6, to investigate BD pile installation. The two sites used for the research described in 3.2.4.1 have also been selected for investigation of the installation parameters. Detailed data have been recorded processed and reported for one BD pile at each site. Summary statistics from a larger sample of piles has also been collated and processed to investigate general trends. Furthermore, there was an opportunity at one site to monitor a comparable CFA pile in the same ground conditions. The CFA pile was constructed with the same plant and equipment as the BD piles, with the exception of the auger.

Observations can be made using the recorded data and comparison between the overall installation processes is permitted by calculating the total energy required to advance the auger into the ground. The energy is calculated using the formula for screwing in energy proposed by Van Impe (1998) shown in Equation 3.1.

The screwing in energy, $E_s = \dfrac{N_d v_i + n_i M_i}{\Omega v_i}$ **Equation 3.1**

where N_d is the vertical thrust of the auger, v_i is the vertical speed of the auger, n_i is the rotational speed of the auger, M_i is the torque applied and Ω is the area of the borehole. In each case the subscript "i" refers to the screwing in phase.

Example piles have been selected for comparison, these have been selected as they are piles that have been closely monitored and tested. Furthermore, a sample of piles from each site has been randomly selected to obtain statistical estimates of the overall trends and variability in energy.

Observation and analysis of the pile installation logs is used to increase the understanding of the rate at which BD piles can be constructed, the difficulties encountered and limitations of the technique. Comparison with a CFA pile in similar ground conditions provides a useful benchmark for comparison.

3.3 SUMMARY OF METHODOLOGY

A variety of research methodologies has been employed, both quantitative and qualitative. The state of the art has been defined through literature review. A descriptive and analytical approach has been used to quantify existing knowledge of ground conditions and to apply this knowledge to interpretation of ground conditions for use in pile design. Philosophical research methodology has been used to develop a BD pile classification system, while the performance of these piles has been investigated through experimental research. The degree of variability and its effect on pile capacity has also been investigated using an experimental methodology.

4 RESEARCH SUMMARY

This section describes the tasks undertaken during the research in order to address each of the objectives and presents the results and outcomes of those tasks. The work relating to each objective, as shown in Figure 3.1, is described in turn. Objective 1 has been addressed through the literature review described in section 2. Sections 4.1 and 4.2 describe the quantification of existing experience relating to shear strength of London Clay and the application of this information to design through quantitative, objective methods (objective 2). The construction of a pile classification system for BD piles (objective 3) is described in section 4.3. The performance of BD piles (objective 4) is described in section 4.4. Investigations into the variation of pile dimensions (objective 5) are described in section 4.5. Investigations into the installation of BD piles (objective 6) are described in section 4.6.

Papers 1 and 2 (Appendices A and B) directly address objective 2, and detail the development of quantitative methods for interpreting site investigation for CFA and BD pile design. The quantification of existing knowledge is described and the methods for combining this knowledge with site specific data are presented and discussed. Paper 2 goes further to explore the effects of variability on the pile design through probabilistic analysis and hence addresses objective 5. Paper 3 (Appendix C) presents the philosophical research and the development of a logical framework and classification system for BD piles and therefore describes the work that addresses objective 3. Paper 4 draws together objectives 4 and 6 to investigate pile performance and to investigate installation parameters for BD piles; experimental test pile results are reported, analysed and discussed with respect to the monitored installation.

While the fundamental approach to pile design (as described in section 1.1.2) remains unchanged, this research has addressed each component in turn seeking to better understand and define the factors contributing to the capacity of a pile. The largest uncertainty in design, and most difficult to define, is the input parameter soil strength. A rigorous, objective method for the inclusion of previous knowledge and experience was proposed and explored which led to greater certainty in the values selected to model site conditions (sections 4.1 and 4.2). Adhesion between the pile and soil was investigated for BD piles through analysis of load tests, (section 4.4). The as-constructed diameter of a pile and the effect of any difference between actual and assumed sizes was investigated (section 4.5). This is of value not only for pile design but is also of importance for analysis of pile performance and back calculation of adhesion (α).

4.1 QUANTIFYING EXPERIENCE OF CLAY STRENGTH

There are two aspects to the quantification of experience and existing knowledge in relation to Objective 2 as described in section 3.2.2. The first aspect is the gathering of information; this includes the selection of data sources and the type of information required. The second aspect is the analysis and interrogation of the data for the identification of general trends or groupings and observed variations from these. In this research, London Clay from the London Basin was selected as the subject. Current knowledge of the London Clay was identified in the Literature review described in section 2.3. Potential subdivisions which may occur in the deposit were identified and included the degree of weathering present, the load history of the soil, stratigraphical differences and the presence of faulting.

The methodology used to construct of the database was discussed in section 3.2.2.1. Data was sourced from Rock and Alluvium's archive of projects during the period 2004 to 2007. The database contains records from 234 boreholes at 68 sites. Results from 947 quick undrained triaxial compression tests and 1080 SPTs have been recorded. The data is from across the London Basin, the plan distribution of sites is shown in Figure 4.1. The structure of the database is further described in Paper 1. The data collected is summarised in sections 4.1.1 to 4.1.6.

4.1.1 RELATIONSHIP BETWEEN STRENGTH AND DEPTH

The strength of the soil (from quick undrained triaxial shear tests) is plotted against depth of sample in Figure 4.2. Plots of strength against depth can be described by a trend line. These are often 'eyed-in' in engineering practice. This allows a subjective element to be incorporated into design. Bias can be introduced by way of conservatism or perceived confidence in the strength values on the part of the engineer. Such subjective decisions are difficult to quantify and are a product of the application of engineering judgement by the experienced engineer.

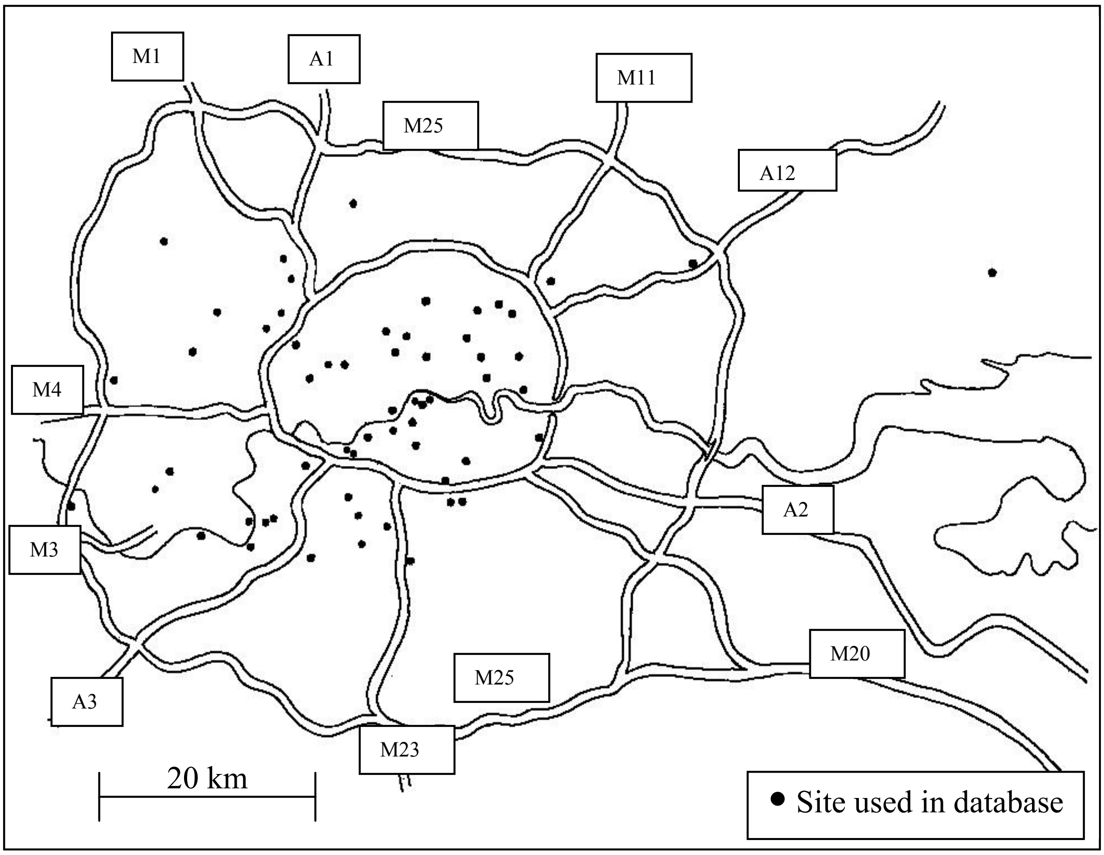

Figure 4.1. Location of sites in the Greater London area used in database.

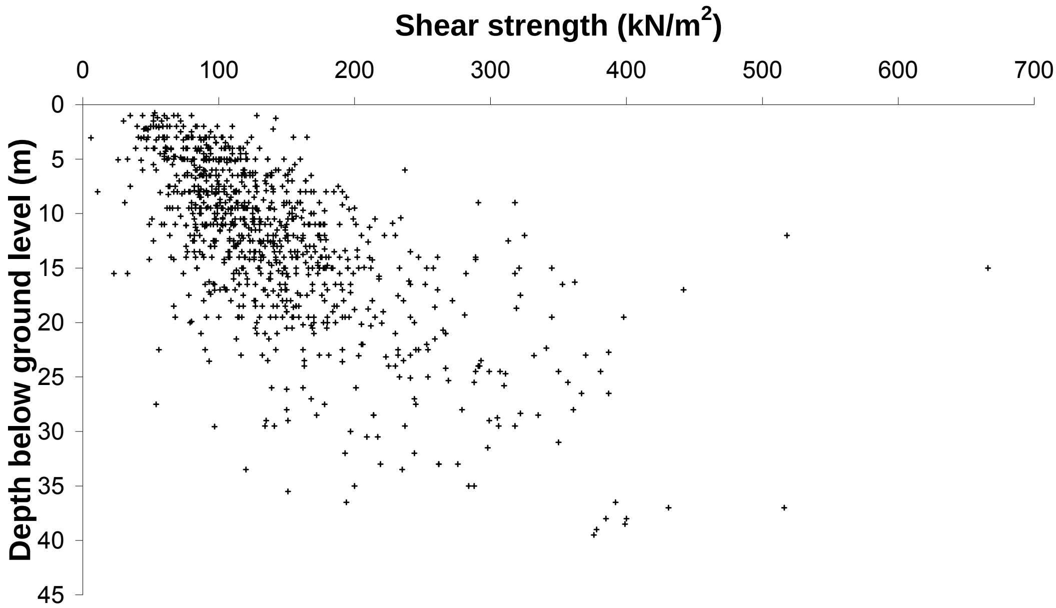

Figure 4.2. Shear strength from triaxial compression tests against depth of sample for all sites in the dataset.

In a situation where such experience is not available, or where a more rigorous approach is called for, a statistical approach is necessary. A regression of the data plot will produce an average trend through the data. In this research, the method of least squares is adopted. By this method (using the regression functions in Microsoft Excel) various line types can be fitted to this data. Figure 4.3 shows the various regression lines. The equation of these lines and the correlation coefficient (R^2) are shown in Table 4.1. There is a large degree of variation apparent in the results plotted. This variability is dealt with in design through application of factors of safety. For a pile in cohesive material, the majority of the capacity is provided by shaft friction and hence this variation is less critical as an average cohesion over the length of the pile can be considered. The value at a point is more critical for end bearing piles and further consideration of the variation is required; often in practice the potentially detrimental effect of a lower value occurring is avoided by selection of a lower bound value.

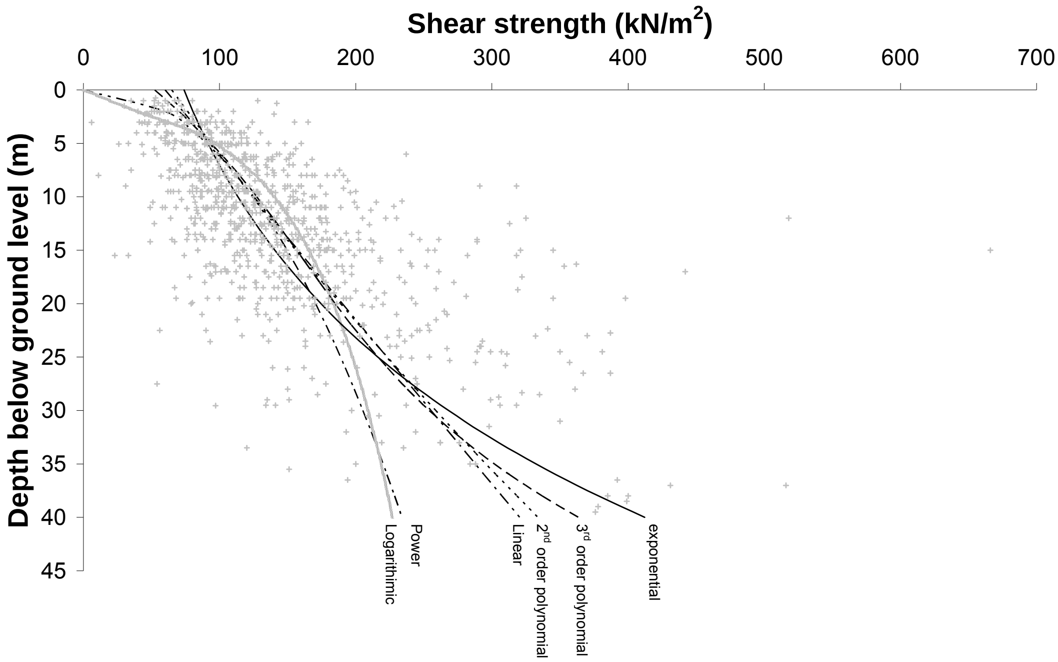

Figure 4.3. Comparison of different regression lines for shear strength from triaxial compression tests against depth of sample using all data.

Table 4.1. Equations of regression lines fitted to strength versus depth plot with R^2 values.

Line Type	Equation	R^2
Linear	$c_u = 6.5d + 60.0$	0.42
Power	$c_u = 43.485d^{0.4561}$	0.41
logarithmic	$c_u = 62.634\ln(d) - 4.0971$	0.35
exponential	$c_u = 73.28e^{0.043d}$	0.41
polynomial (2nd order)	$c_u = 0.026d^2 + 5.6808d + 64.868$	0.43
polynomial (3rd Order)	$c_u = 0.0048d^3 + 0.2307d^2 + 9.3226d + 52.323$	0.43

It can be seen from comparison of the R^2 values that there is little difference in the degree of fit. Inspection of the plotted lines (shown in Figure 4.3) shows that there is little difference in the lines, particularly between 5 and 25m (where in practice a large proportion of CFA and BD pile (shaft) capacity is mobilised). It is therefore considered appropriate to adopt a straight line as this is the simplest to work with – there is little or no advantage to increasing the complexity. The regression line is not forced through the origin as it is reasonable to assume that the over-consolidated London Clay has some strength at surface level. The medium degree of correlation displayed by the R^2 values for all of the lines reflects the large scatter in the data. Interpretation of an R^2 value is inexact and subjective and is dependent upon context and purpose. In this case the combination of material variability, sampling error, measurement error and the use of different, uncontrolled, site investigation contractors and methods leads to a large variation. Viewed in the context of the previous literature however, (e.g. Skempton, 1959; Patel, 1992) the trend observed is reasonable, as is the degree of variation. The purpose of calculation of the R^2 value is also trivial in this case. It has been calculated solely to demonstrate that there is no benefit in adopting a more complex fit than a straight line. The general relationship and presence of a trend has been demonstrated in the literature (Patel 1992) as has the nature of the distribution (Sia and Dixon, 2007; Hooper and Butler, 1966). This degree of variation is suitable for the subsequent analyses described as the distribution is normal.

In the following sub-sections, the subdivision of the dataset is considered according to the degree of weathering and geographical location. Sub-division of the dataset by load history is not considered possible as the precise history can not be determined and the site investigation and associated testing does not provide information relating to pre-consolidation values. It is therefore essential to consider the various loading histories as a source of the variation which is observed in strength depth relationship of the data.

4.1.2 Consideration of Effect of Weathering of London Clay

Weathering of the soil may lead to a change in its engineering properties. The degree of weathering is not recorded in the site investigation data beyond a bi-modal descriptor (weathered or unweathered); it is believed that this is based primarily on the colour of the clay observed during the investigation.

The weathered and unweathered clay has been examined separately to investigate whether there is any significant difference in the observed trends and variation. This produces regression lines shown in Figure 4.4. The equations of the lines and the R^2 values are shown in Table 4.2. While it is possible that the shallower gradient for the brown, weathered clay is a result of degradation associated with weathering, the difference between the two lines is small with respect to the degree of variation present. The R^2 is lower for each of these two lines than for the dataset as a whole, although this may be a symptom of the smaller sample size in each case. Furthermore, the limited information available and lack of precision in the description of the degree of weathering leads to a low degree of confidence in the classification of weathered and unweathered clay and therefore a subdivision was not used.

Table 4.2. Regression of triaxial strengths with depth for weathered and unweathered clay.

Soil sub-division	Equation of regression line	R^2
Weathered Clay	$c_u = 6.3d + 66.7$	0.32
Unweathered Clay	$c_u = 6.6d + 58.0$	0.34

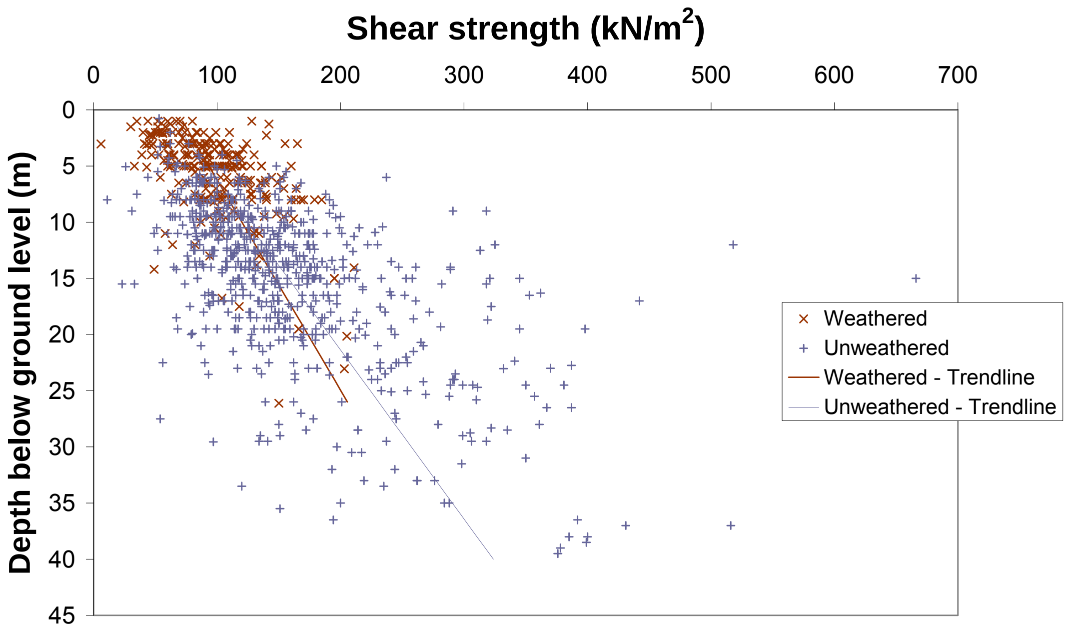

Figure 4.4. Relationship between strength from triaxial compression test and depth of sample below ground level, for sub-sets of weathered and unweathered clay.

4.1.3 Consideration of Effect of Geographical Location

To investigate geographical trends, the London Basin was divided up into three divisions, 'east', 'central', and 'west'. The allocation of sites to geographical regions was done by inspection of the plan locations of sites. Piles are typically located in the upper 30m of the London Clay; the boundary between west and central regions was placed in a way so as to be consistent with the transition from piles being predominantly in lithological unit B to unit A (after King, 1981) as shown in Figure 4.6, although there is nothing in the literature to suggest that triaxial strength will differ between the lithological units. This falls somewhere between Bishopsbridge and Camden Town and this boundary was extended in a north-south direction. Sites close to this boundary were omitted from this analysis. A cluster of sites, which could be described as central London, were then apparent and these were assigned to the central group. The eastern group consisted of the sites to the east of the central cluster. The allocation of sites to these divisions is shown in Figure 4.5. The plots of strength versus depth are shown in Figure 4.7. There are between 130 and 360 data points in each division.

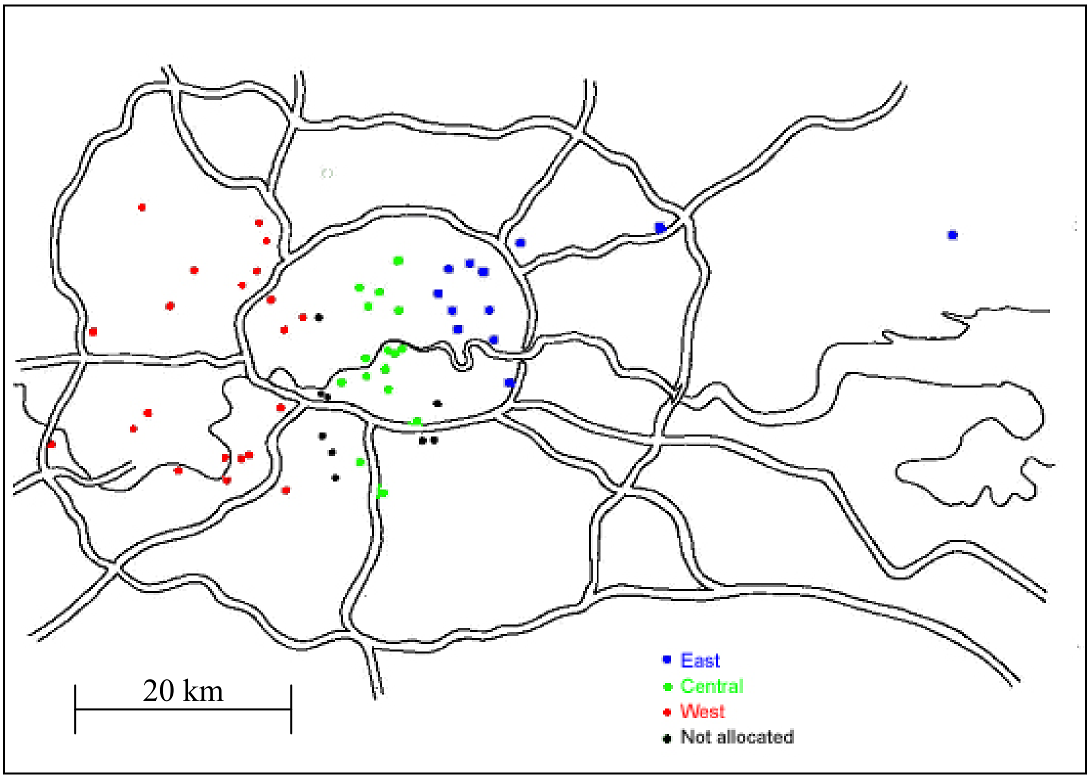

Figure 4.5. Allocation of sites to geographical divisions.

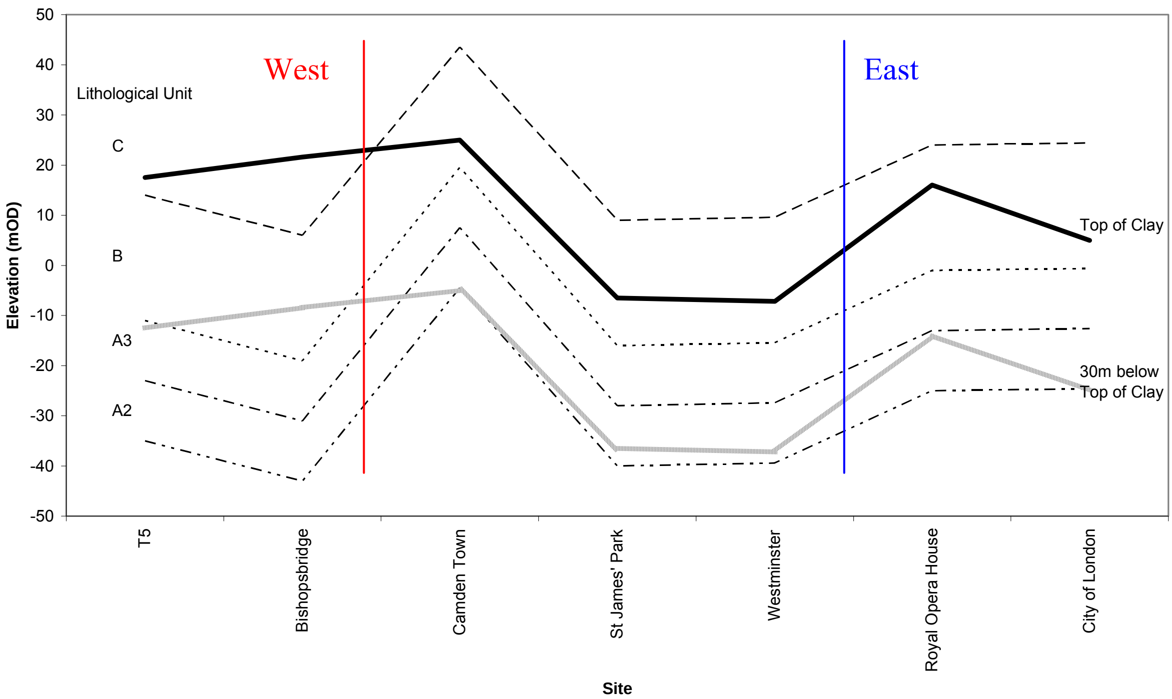

Figure 4.6. Levels of the lithological units in relationship to the current surface of the London Clay.

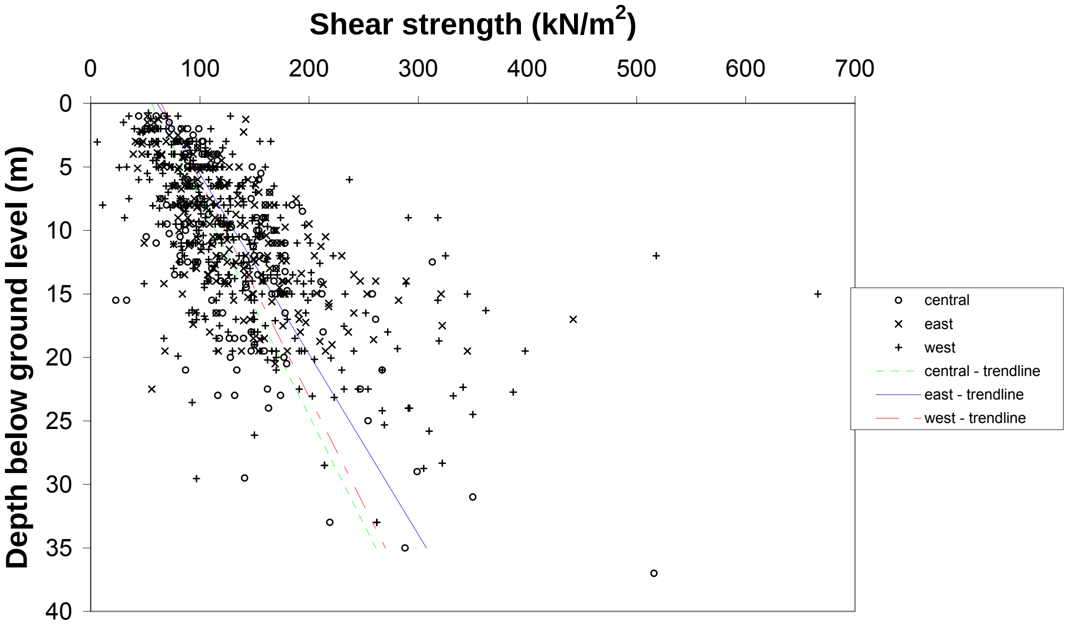

Figure 4.7. Relationship between strength from triaxial compression test and depth of sample below ground level for sub-sets of geographical regions.

Inspection of the trends gives rise to an ambiguous situation. There is a difference in the gradient for the central London section; the east and west divisions however appear to be very similar. This suggests that the different lithological units do not give rise to different strength characteristics. Due to the limited information available regarding the different sites, it is not possible to further differentiate the sites geographically in any meaningful way. Any geographical variation is therefore treated as a component of the overall observed variation.

4.1.4 COMPARISON WITH PREVIOUS WORK

The same strength data used above has been re-plotted against depth below top of London Clay to obtain the scatter plot shown in Figure 4.8. The equation of the regression line is $c_u = 7.4d + 79.0$. The R^2 is 0.4, this is similar to that found using ground level as a datum. There is no apparent advantage gained by introducing the additional complication of plotting strength versus depth below top of clay. It does, however, allow comparison to be drawn with previous work. Patel (1992) used the top of the London Clay as a reference point, not ground level. Figure 4.9 shows mean regression lines of the strength with depth below top of clay for four example sites, selected randomly from the sites used to build the database presented in section 4.1, along with the mean for the whole dataset; overlain on this is the envelope of the mean lines presented by Patel (1992) for 23 different sites across the London basin.

The mean regression line of the data collected in this study is observed to be close to the middle of the range reported by Patel (1992). This is consistent with an assumption of normal distribution of shear strength. The individual mean regression lines for the example sites from this study are also shown to fall within the range found by Patel (1992).

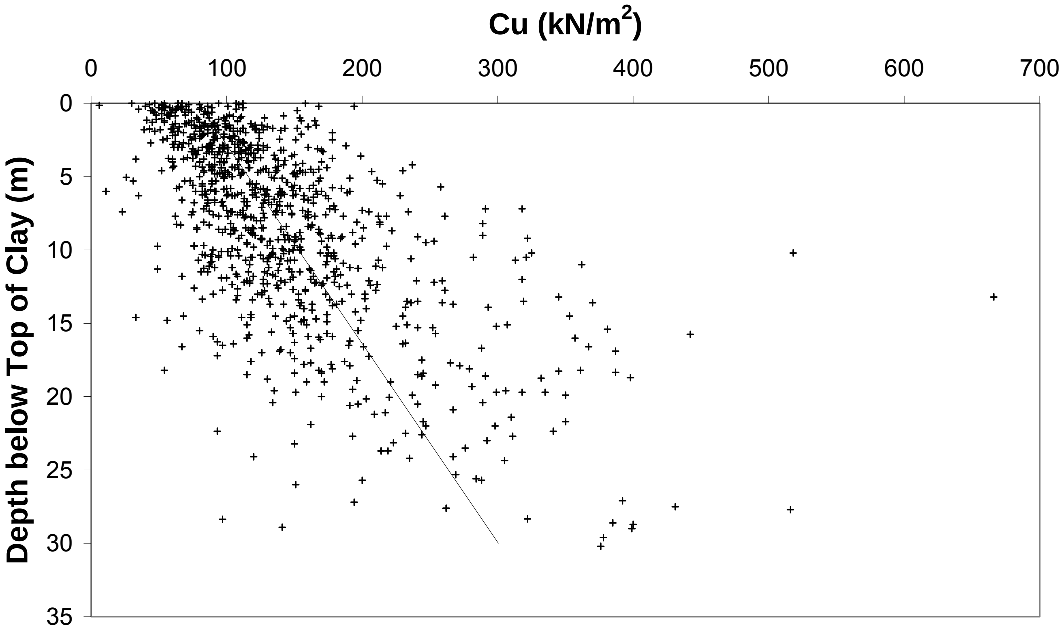

Figure 4.8. Relationship between strength from triaxial compression test and depth of sample below top of London Clay.

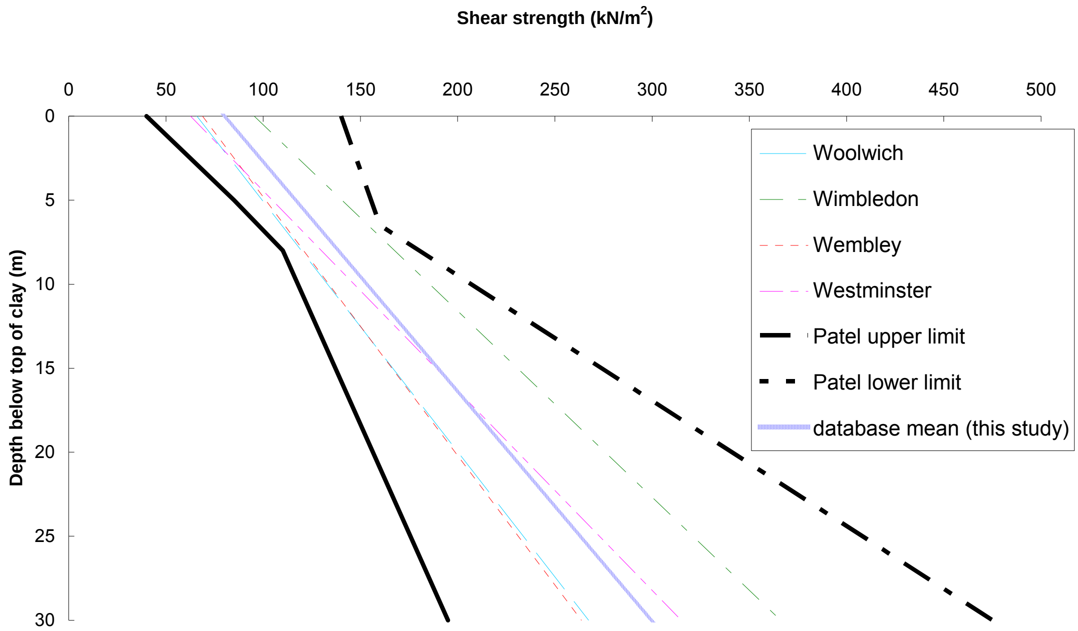

Figure 4.9. Comparison of site data and dataset mean for strength-depth relationship with data published by Patel (1992).

4.1.5 Distribution of Data

The statistical manipulation described in section 4.2 relies upon the data having a normal distribution. It is not the intention of this research to investigate the nature of the distribution of shear strength as It has been seen from the literature (Hooper and Butler, 1961; Sia and Dixon, 2007), that the shear strength of London Clay is normally distributed about the mean value. It is possible, however, to demonstrate consistency with previous work through application of a goodness-of-fit test (χ^2 test). This is demonstrated by example in section 4.1.6.1.

4.1.6 Grouping of Data into Bands

Triaxial data in its raw format, or simply summarised by a regression line, cannot be easily employed as prior knowledge. A method of processing the data is required such that a mean and a measure of the distribution can be readily obtained for use in Bayesian updating. The method used in this research is to group the strength data by depth to top of sample and to calculate summary statistics for each group; bands of 1m thickness have been used for the grouping; depths have been measured from ground level. A 1m band thickness was selected as this was considered to represent a manageable interval that was consistent with the degree of confidence which could be associated with the actual location of a 200mm long triaxial test sample which has been taken from within an original sample of 450mm length. The summary statistics and the number of sites and data points used to generate them are shown in Table 4.3. The mean values for each band are plotted in Figure 4.11, overlaid by the mean regression line found previously for all test results (first shown in Figure 4.3) and a reasonable fit can be observed, particularly for depths between 0 and 20 metres where a greater number of data points are available. The deviations which occur below 20m are likely to be a result of the low number of test results available at these depths as many boreholes used to build the database did not extend below 20 m, as indicated in Table 4.3. Furthermore the regression line for the summary data is also shown and there is good agreement with the regression line from the untreated data.

The coefficients of variation shown in Table 4.3 and Figure 4.11 are in general agreement with those reported in previous literature (Schneider, 1997; Hooper and Butler, 1966; Phoon and Kulhawy, 1999).

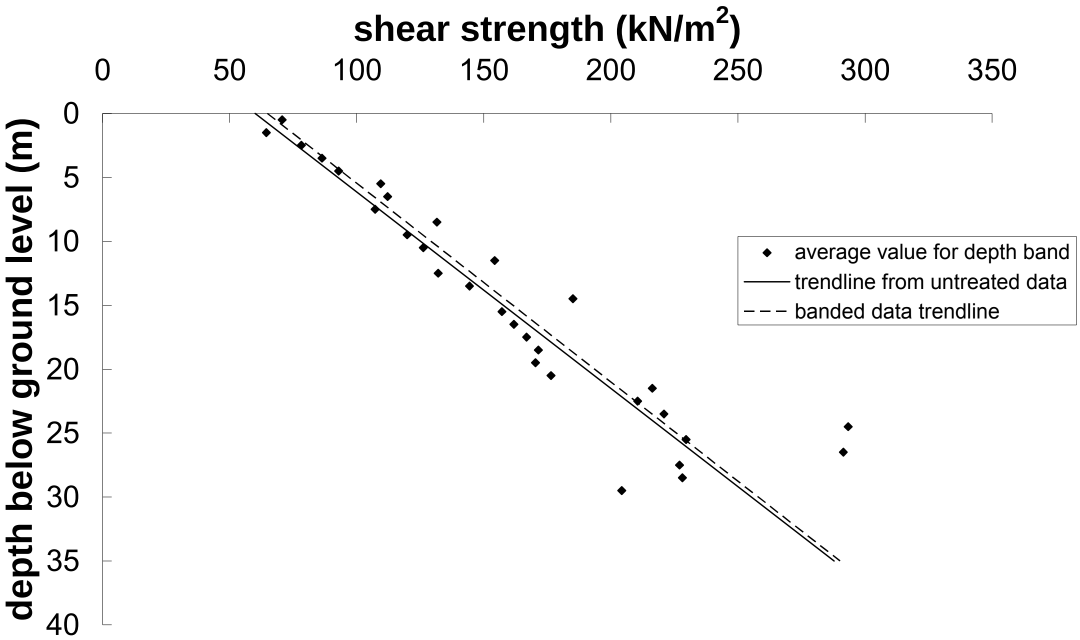

Figure 4.10. Mean values for shear strength grouped into 1m thick depth bands, shown with the original regression line for all data points and regression of summary points (banded data).

Table 4.3. Statistical analysis of shear strength data grouped by depth for London Clay.

Nominal Depth (m)	Depth range (m)	Number of sites	Number of test results	Mean shear strength, x_m (kN/m^2)	Coefficient of variation, V_x
1	0.5-1.5	9	12	70.7	0.46
2	1.5-2.5	14	28	64.4	0.36
3	2.5-3.5	23	38	78.2	0.38
4	3.5-4.5	20	42	86.3	0.26
5	4.5-5.5	34	57	92.9	0.29
6	5.5-6.5	20	31	109.4	0.36
7	6.5-7.5	27	42	112.1	0.25
8	7.5-8.5	43	67	107.2	0.34
9	8.5-9.5	25	38	131.6	0.41
10	9.5-10.5	34	58	119.8	0.31
11	10.5-11.5	42	71	126.2	0.32
12	11.5-12.5	23	39	154.3	0.50
13	12.5-13.5.	31	49	132.0	0.32
14	13.5-14.5	39	64	144.4	0.36
15	14.5-15.5	25	40	185.1	0.51
16	15.5-16.5	19	29	157.2	0.48
17	16.5-17.5	21	37	161.8	0.45
18	17.5-18.5	13	22	166.8	0.35
19	18.5-19.5	18	24	171.5	0.33
20	19.5-20.5	20	37	170.3	0.39
21	20.5-21.5	10	14	176.5	0.33
22	21.5-22.5	2	7	216.3	0.36
23	22.5-23.5	10	20	210.5	0.41
24	23.5-24.5	9	13	220.9	0.30
25	24.5-25.5	5	10	293.4	0.16
26	25.5-26.5	2	7	229.6	0.38
27	26.5-27.5	1	4	291.5	0.36
28	27.5-28.5	2	7	227.0	0.47
29	28.5-29.5	5	8	228.1	0.33
30	29.5-30.5	3	7	204.3	0.42

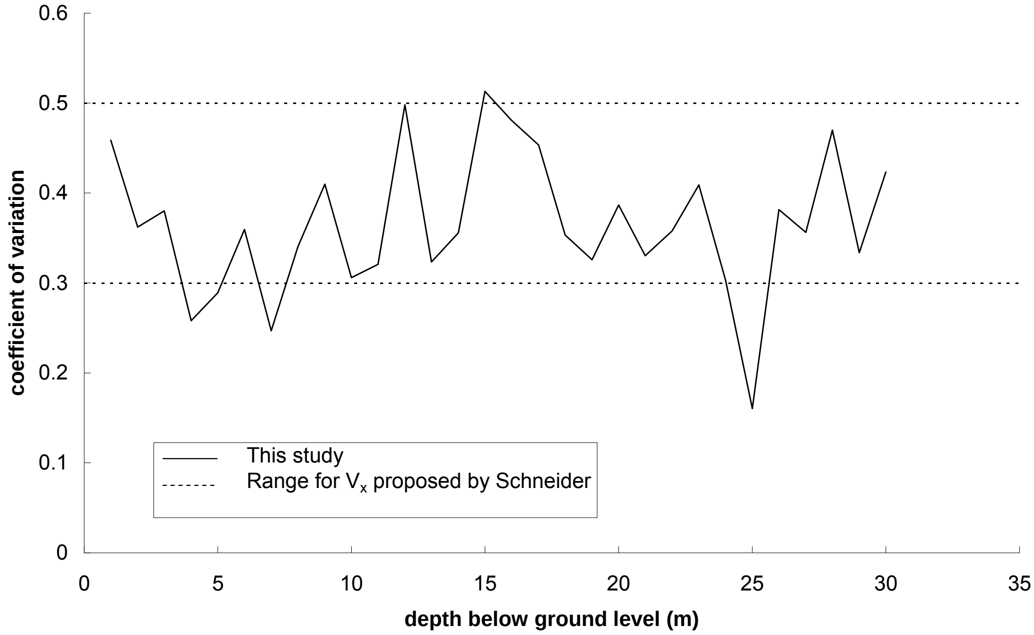

Figure 4.11. Coefficient of variation (standard deviation/mean) for the gathered data as grouped into 1m depth bands compared to published data (Schneider, 1997).

4.1.6.1 Goodness-of-Fit Test

The χ^2 test has been described in section 2.7.1 ands can be applied here to verify that the distribution within a depth band is normal. For this example test the 10 to 11m depth band has been selected as the number of results is well suited to the test.

The null hypothesis is that the distribution is normal with mean value, 126, and standard deviation 40.5. The calculation of the χ^2_{test} statistic is set out in Table 4.4. χ^2 test calculations for 10 to 11m depth band of London Clay triaxial test results. There are 8 degrees of freedom and hence, from tables, $\chi^2_{5\%} = 15.51$. Since $\chi^2_{test} < \chi^2_{5\%}$ this is not significant and there is no evidence that the distribution is not normal. The test can be applied in a similar manner to other observations; Table 4.5 shows a summary of the application of the χ^2 test to each of the depth bands where there are sufficient observations for the test to be valid. In each case there is no evidence that the distribution is not normal.

Table 4.4. χ^2 test calculations for 10 to 11m depth band of London Clay triaxial test results

	Observed Value, O	Expected Value, E	$O-E$	$\frac{(O-E)^2}{E}$
<75	5	5.856901	-0.8569	0.12537
75<x<95	11	7.318835	3.681165	1.85152
95<x<105	8	5.194772	2.805228	1.51485
105<x<115	6	6.090722	-0.09072	0.001351
115<x<125	6	6.720327	-0.72033	0.077209
125<x<135	9	6.978008	2.021992	0.585905
135<x<145	3	6.818549	-3.81855	2.138478
145<x<155	6	6.270064	-0.27006	0.011632
155<x<165	3	5.425895	-2.42589	1.084607
165<x<185	8	7.804973	0.195027	0.004873
X>185	6	6.154607	-0.15461	0.003884
			$\chi^2_{test} = \frac{(O-E)^2}{E} =$	7.3997

Table 4.5. Summary of chi test applied to other depth bands

Depth Band	**Number of Samples**	χ^2_{test}	**Degrees of freedom**	$\chi^2_{5\%}$
4 to 5m	57	3.31	4	9.49
7 to 8m	67	10.31	6	12.59
9 to 10m	58	4.84	6	12.59
10 to 11m	71	7.4	8	15.51
12 to 13m	49	8.11	4	9.49
13 to 14m	64	3.89	5	11.07

4.1.7 Additional Soil Property Data From *In Situ* Tests

A larger data set was constructed by converting the standard penetration resistance (N) to equivalent shear strength by using the mean observed ratio (The calculation of this ratio is discussed further in section 4.1.7.3). The enlarged database can then be interrogated and potentially applied in the same fashion as previously discussed for the shear strength (from triaxial compression test) database. This will have the advantage of containing more data points and from two diverse sources.

4.1.7.1 General Trend of Standard Penetration Resistance with Depth

A linear trend for SPT blow count (N) against depth below ground surface for the SPT dataset is shown in Figure 4.12 a straight line regression has been performed on the data using the method of least squares. The equation of the line is $N = 1.4d + 11.7$, where N is the standard penetration resistance and d is the depth below ground level; the fit of the line to the data has a correlation coefficient (R^2) of 0.55. A straight line has been chosen as this provides the simplest representation of the data, as discussed in section 4.1.1. Other types of line (power, logarithmic, exponential and polynomial) do not provide significantly greater correlation because of the high degree of scatter in the data. Some test results are limited to a maximum value of 50 blows, regardless of penetration depth, in some site investigations extrapolations have been conducted to achieve equivalent values above 50, in others they are simply reported as 50. It is impossible to know, in all cases, which results have been extrapolated, which have been limited to 50 and which were naturally 50 because of the lack of information in the SI reports. The results have therefore been analysed using ‘as reported’ values and the lack of information is treated as an additional source of variation in the results.

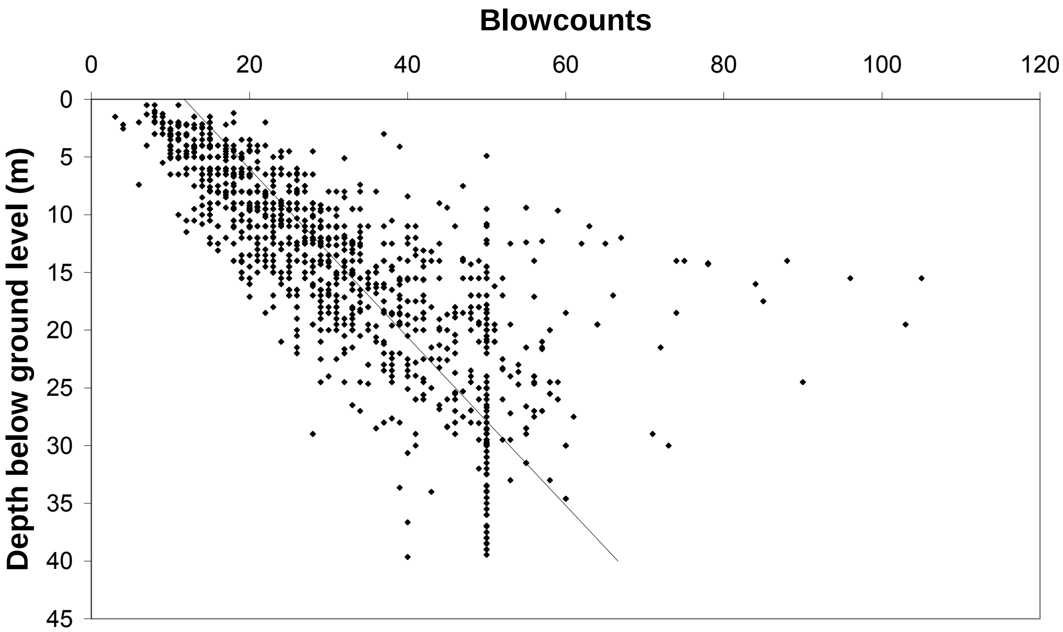

Figure 4.12. Relationship between standard penetration resistance and depth of test below ground level. Tests terminated before full penetration was achieved are plotted as N=50.

An alternative approach is to consider only those results where the actual value is known. Results where the test was terminated at 50 blows or where an extrapolated value has been reported are removed from the dataset. Figure 4.13 shows this refined selection of data and a straight line regression. The equation of the line is $N = 1.3d + 11.1$. This is very similar to that found from all of the data and suggests that, in this instance, the inclusion of the additional data has had little effect on the observed trend.

A correlation factor to obtain undrained shear strength is required if the results of SPTs are to be combined with the results of the triaxial compression tests. This explored below in section 4.1.7.3.

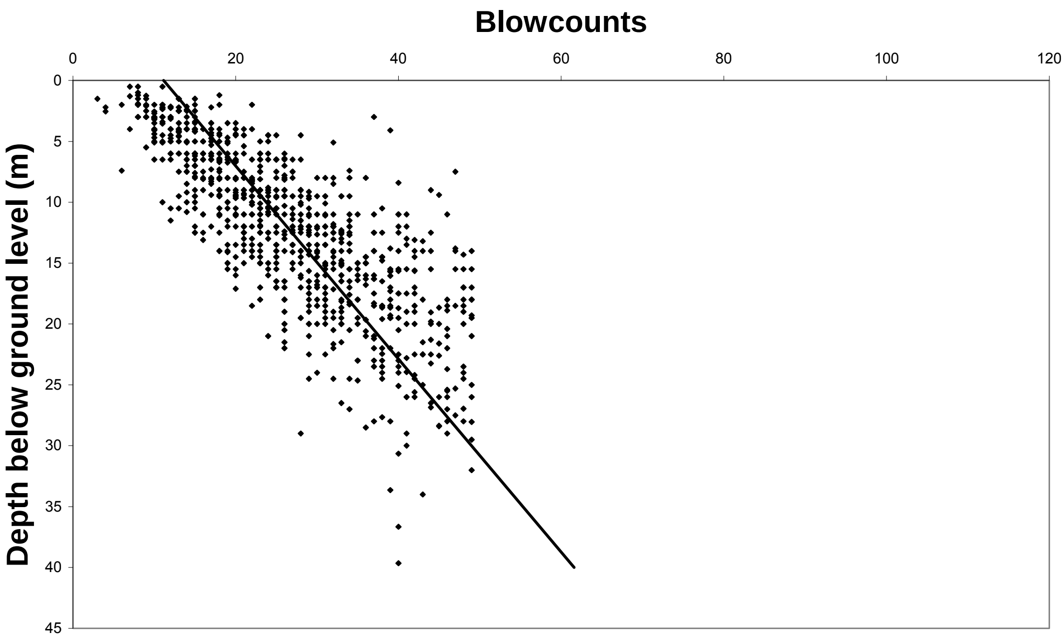

Figure 4.13. Relationship between standard penetration resistance and depth of test below ground level. Tests terminated before full penetration was achieved ere removed.

4.1.7.2 Grouping SPT Data into Bands

The SPT data can be grouped into 1m bands and summarised in the same manner as described for the triaxial test result data described in 4.1.6. The summary statistics and the number of sites and data points used to generate them are shown Table 4.6. The mean values for each band are plotted in Figure 4.14, shown with a trend line calculated by the method of least squares, the equation of which is $N = 1.4d + 10.5$. The coefficients of variation are shown in Figure 4.15. The difference in gradient between the regression lines calculated using all of the untreated data and the banded mean values is due in part to the number of results limited to 50 at greater depth. The decrease in observed coefficient of variation with depth is also due to the increase in results limited to 50; many of the results are artificially the same (50) and so there is little observable variation.

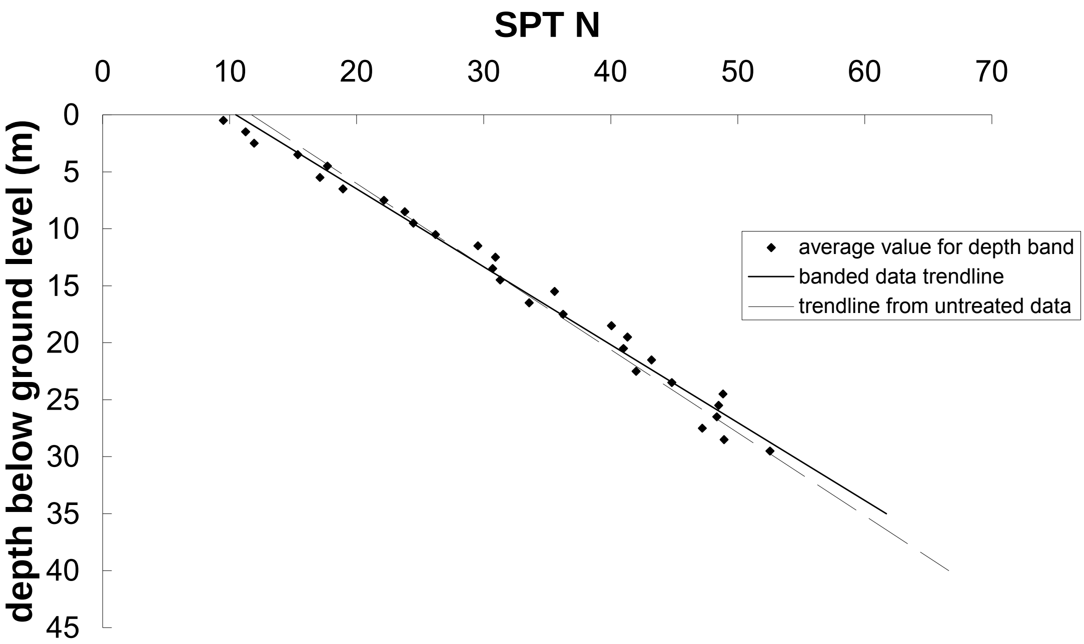

Figure 4.14. Mean values for standard penetration resistance grouped into 1m thick depth bands, shown with the original regression line for all data points and regression of summary points (banded data).

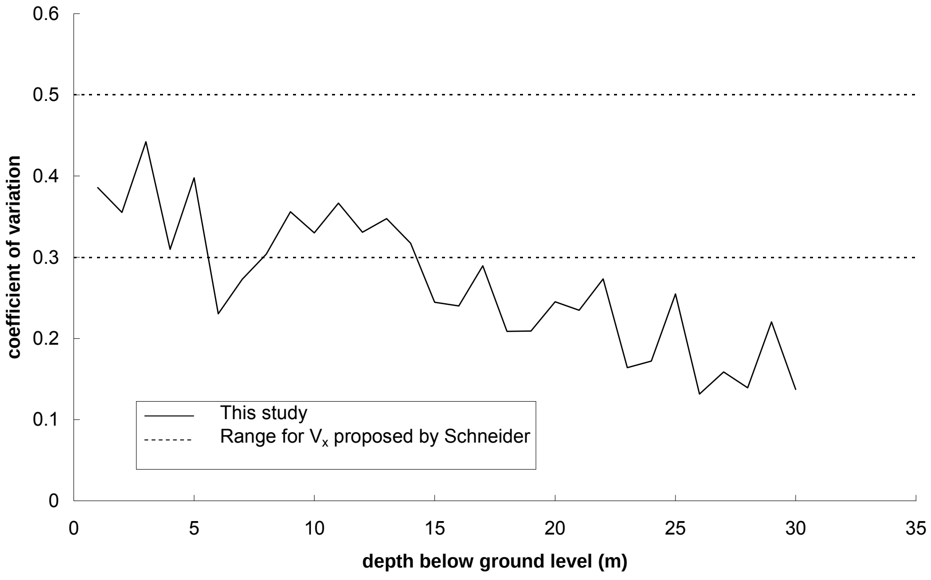

Figure 4.15. Coefficient of variation of standard penetration resistance with depth.

Table 4.6. Statistical analysis of SPT data grouped by depth for London Clay.

Depth (m)	Number of data points	Average N	standard deviation	Coefficient of Variation
0.5	8	9.5	3.66	0.39
1.5	31	11.3	4.00	0.36
2.5	31	11.9	5.28	0.44
3.5	48	15.4	4.77	0.31
4.5	46	17.7	7.04	0.40
5.5	34	17.1	3.94	0.23
6.5	49	18.9	5.16	0.27
7.5	53	22.2	6.73	0.30
8.5	38	23.8	8.47	0.36
9.5	54	24.5	8.07	0.33
10.5	63	26.2	9.61	0.37
11.5	55	29.5	9.77	0.33
12.5	52	30.9	10.74	0.35
13.5	60	30.7	9.74	0.32
14.5	48	31.3	7.65	0.24
15.5	42	35.6	8.53	0.24
16.5	38	33.6	9.72	0.29
17.5	33	36.2	7.57	0.21
18.5	38	40.1	8.38	0.21
19.5	41	41.3	10.13	0.25
20.5	19	41.0	9.63	0.23
21.5	19	43.2	11.82	0.27
22.5	20	42.0	6.89	0.16
23.5	16	44.8	7.71	0.17
24.5	24	48.8	12.44	0.25
25.5	14	48.5	6.38	0.13
26.5	14	48.4	7.68	0.16
27.5	14	47.2	6.58	0.14
28.5	12	48.9	10.78	0.22
29.5	15	52.5	7.21	0.14

4.1.7.3 Correlation of Standard Penetration Resistance and Shear Strength

The standard penetration resistance alone is not of direct use for geotechnical design in fine grained (cohesive) material and a correlation with undrained shear strength is needed. It was assumed that the variation of this correlation factor is of little significance across the deposit in comparison to the variation already observed. From the mean values of N and c_u for each depth band, an average ratio can be found. It has been assumed that both N and c_u increase linearly with depth. The results are shown in Table 4.7. The mean value of c_u/N is 5.2. This can be used to convert the N values to equivalent shear strengths to facilitate combination of the two data sets, this is discussed in section 4.1.7.5. Furthermore, the mean value of c_u/N enables the comparison with previous work discussed in 4.1.7.4.

4.1.7.4 Plasticity Index

Stroud (1974) developed a correlation between c_u/N and plasticity index. By inspection of the correlation, clay with a plasticity index of between 30 and 50 would be expected to display c_u/N of about 5. Plasticity indices were not recorded in many of the site investigations used in this research. There were 344 locations at which PI was reported. The mean plasticity index reported was 45. Therefore a correlation of 5.2 is consistent with the correlation suggested by Stroud (1974).

4.1.7.5 Combined data

Using the mean value for $c_u/N = 5.2$, the standard penetration resistances have been converted to equivalent shear strengths. These equivalent values can then be added to the triaxial test results to create a much larger dataset. The mean values and coefficients of variation in each band are shown in Table 4.8. Figure 4.16 shows the trend of shear strength with depth for the combined data. The equation of the line is $c_u = 7.0d + 60.3$. The coefficient of variation is shown in Figure 4.17.

Table 4.7. Correlation between standard penetration resistance and undrained shear strength by triaxial test by depth band.

depth (m)	SPT N	c_u from triaxial (kN/m^2)	c_u/N (kN/m^2)
	average	average	
0.5	9.5	70.7	7.4
1.5	11.3	64.4	5.7
2.5	11.9	78.2	6.6
3.5	15.4	86.3	5.6
4.5	17.7	92.9	5.2
5.5	17.1	109.4	6.4
6.5	18.9	112.1	5.9
7.5	22.2	107.2	4.8
8.5	23.8	131.6	5.5
9.5	24.5	119.8	4.9
10.5	26.2	126.2	4.8
11.5	29.6	154.3	5.2
12.5	30.9	132.0	4.3
13.5	30.7	144.4	4.7
14.5	31.3	185.1	5.9
15.5	35.6	157.2	4.4
16.5	33.6	161.8	4.8
17.5	36.2	166.8	4.6
18.5	40.1	171.5	4.3
19.5	41.3	170.3	4.1
20.5	41.0	176.5	4.3
21.5	43.2	216.3	5.0
22.5	42.0	210.5	5.0
23.5	44.8	220.9	4.9
24.5	48.8	293.4	6.0
25.5	48.5	229.6	4.7
26.5	48.4	291.5	6.0
27.5	47.2	227.0	4.8
28.5	48.9	228.1	4.7
29.5	52.5	204.3	3.9

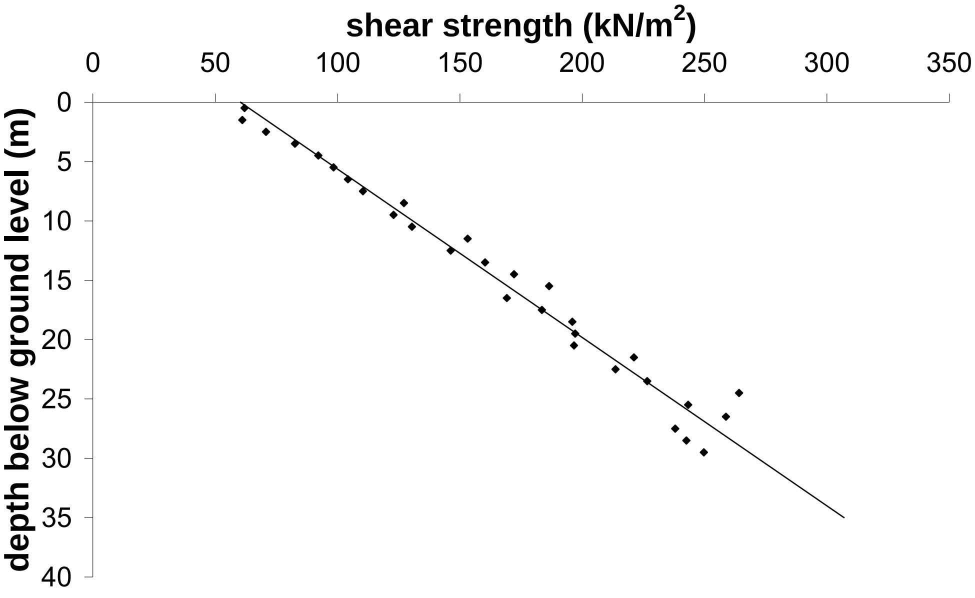

Figure 4.16. Relationship between shear strength and depth below ground level for combined triaxial and standard penetration resistance (equivalent c_u) data.

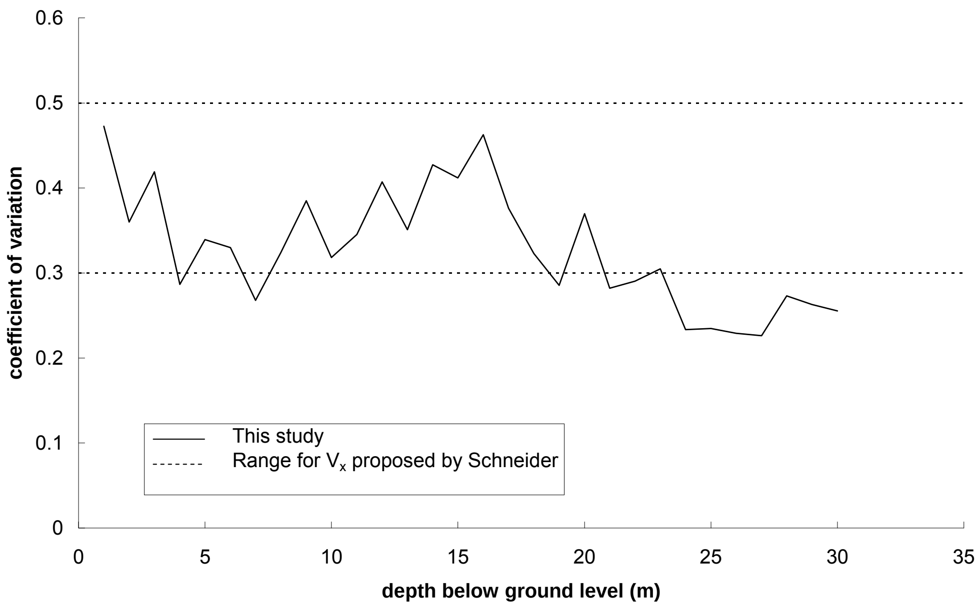

Figure 4.17. Coefficient of variation with depth of combined triaxial and standard penetration resistance (equivalent c_u) data.

Table 4.8. Combined standard penetration resistance equivalent and triaxial data grouped by depth band.

Depth (m)	Average shear strength (kN/m^2)	standard deviation (kN/m^2)	Coefficient of Variation
0.5	62.0	29.30	0.47
1.5	61.1	21.99	0.36
2.5	70.7	29.63	0.42
3.5	82.6	23.66	0.29
4.5	92.2	31.26	0.34
5.5	98.4	32.43	0.33
6.5	104.3	27.93	0.27
7.5	110.3	35.75	0.32
8.5	127.1	48.94	0.38
9.5	122.9	39.08	0.32
10.5	130.4	45.02	0.35
11.5	153.2	62.35	0.41
12.5	146.2	51.29	0.35
13.5	160.3	68.49	0.43
14.5	172.1	70.88	0.41
15.5	186.4	86.24	0.46
16.5	169.2	63.63	0.38
17.5	183.5	59.25	0.32
18.5	196.0	55.94	0.29
19.5	197.1	72.84	0.37
20.5	196.6	55.44	0.28
21.5	221.1	64.18	0.29
22.5	213.6	65.09	0.30
23.5	226.5	52.84	0.23
24.5	264.1	62.01	0.23
25.5	243.3	55.72	0.23
26.5	258.7	58.54	0.23
27.5	238.0	65.00	0.27
28.5	242.6	63.80	0.26
29.5	249.7	63.77	0.26

4.2 USING EXPERIENCE IN INTERPRETATION OF GROUND CONDITIONS

The information collated in the database can be used as prior knowledge to refine, or update, the 'derived values' for soil properties from a site specific investigation, using Bayesian techniques as proposed by Tang (1971). First, however, it is essential for the engineer to make a judgement regarding the applicability of the prior data. An assessment must be made of whether the site specific results are from the same population as the database, for example are the soils of the same strata and have they been subject to the same environmental and loading history. Where there are significant departures from the database mean values, it should be investigated whether there is a reasonable explanation, such as sampling difficulties. If there is a reasonable explanation, and the data is still deemed to be from the same population then there is a strong case for the application of Bayesian updating.

The process of using Bayesian updating to augment site specific data using quantified experience has been illustrated through application to case study examples of pile design that have been reported in Papers 1 and 2. An overview of the case studies is given in section 4.2.1.

The database used as prior information contains all available results from London Clay (of the London Basin). Section 2.3 explored the literature relating to the London Clay and, with the investigation described in section 4.1, showed sufficient uniformity across the deposit for the results to be considered as members of the same population.

4.2.1 CASE STUDIES

The site for the first case study is located near to Woolwich, South East London. The site investigation consisted of eight boreholes, from which 37 samples (100mm diameter) were tested in quick undrained triaxial tests. There were no appreciable superficial deposits reported, London Clay is present from just below the surface to an unproven depth beyond the investigation limits and anticipated pile toe depths. The test results are shown, with mean regression line, in Figure 4.18. The equation of the mean regression line for the site data is $c_u = 6.8d + 51.8\,\mathrm{kN/m^2}$.

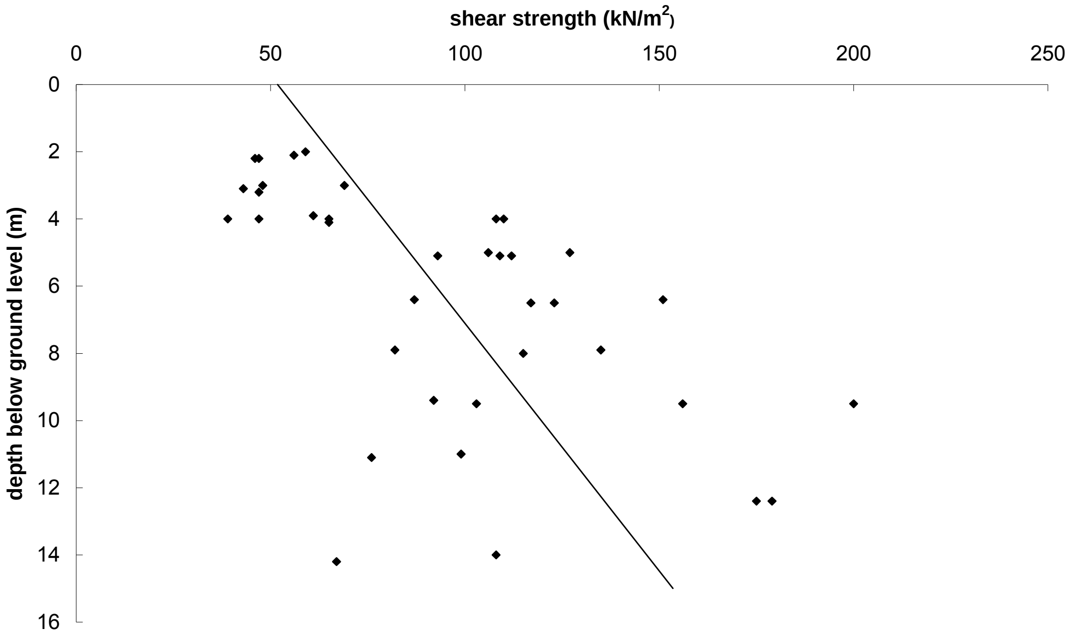

Figure 4.18. Example of shear strength versus depth below ground level, with linear regression trend line, from site investigation at a site near Woolwich.

The site for the second example is in Wimbledon, South West London. The site investigation consisted of 7 boreholes, from which 15 samples were tested. The soil typically consists of approximately 1.5m of loose granular fill material overlying 1.5m of alluvial deposits, largely reported as silty sandy clay. Below 3m the site is London Clay to unproven depth. The clay is weathered to approximately 5 m. The test results are shown, with mean regression line, in Figure 4.19. The equation of the mean regression line for the site data is $c_u = 9.3d + 61.7$ kN/m^2.

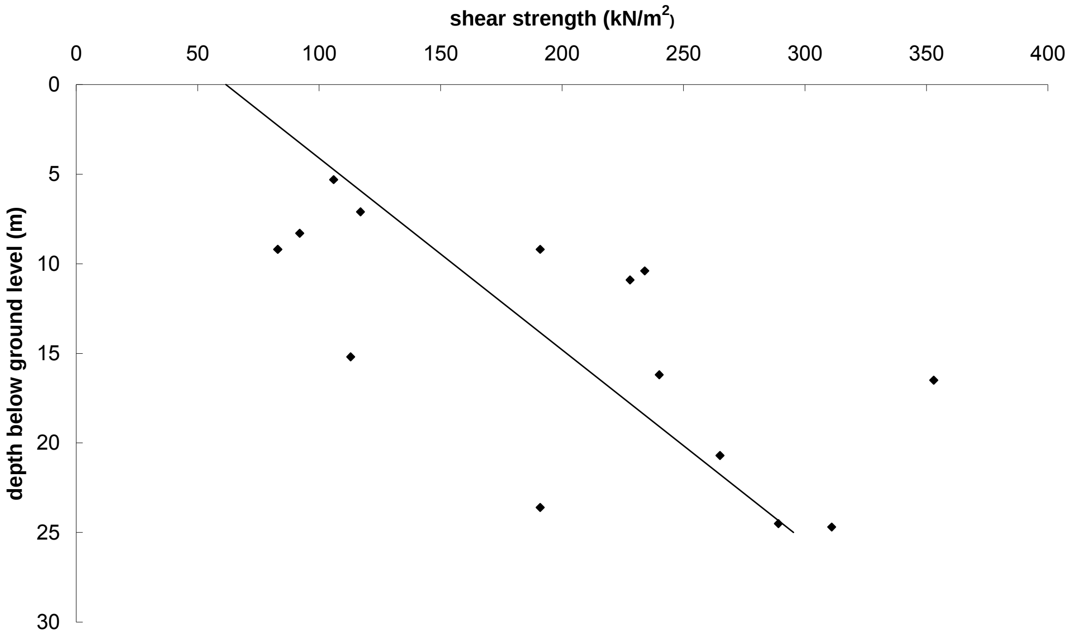

Figure 4.19. Example of shear strength versus depth below ground level, with linear regression trend line, from site investigation at a site near Wimbledon.

4.2.1.1 Application of Bayesian Updating

The data from a site specific investigation is refined using the global database described in section 4.1. Each test result is updated in turn following the method discussed in section 3.2.2.2 and described in detail in Papers 1 and 2. Table 4.9 and Table 4.10 show the results of the updating for each of the test results from the two case study sites. Example calculations are included in Paper 1 and Paper 2. The effect of Bayesian updating on the mean strength-depth profile is illustrated in Figure 4.20 and Figure 4.21. It can be seen that in the case of the Woolwich case study the effect of the updating is to increase the mean strength of the soils, whereas in the case of the Wimbledon case study, the effect is to reduce the mean strength. While the former is clearly beneficial for the economy of pile design, the latter remains of interest because of the associated reduction in risk. Without the inclusion of the previous knowledge, there is often a lack of data this can lead to difficulty in forming an interpretation of the ground conditions which in the worst case can result in an under-designed pile.

Table 4.9. Bayesian updating of data for the Woolwich case study site.

Depth of Sample (m)	Test result (kN/m^2) (Derived Value)	Site mean strength from regression line at depth of sample (kN/m^2)	Site Coefficient of variation	Relevant depth band (m)	Database mean for depth band (kN/m^2) (*a priori* data)	Database Coefficient of variation (*a priori* data)	Updated mean strength at depth of sample (kN/m^2)	Updated Coefficient of variation
2.0	59	65.39	0.3	1.5-2.5	64.43	0.36	65.0	0.20
2.1	56	66.07	0.3	1.5-2.5	64.43	0.36	65.4	0.20
2.2	46	66.75	0.3	1.5-2.5	64.43	0.36	65.8	0.20
2.2	47	66.75	0.3	1.5-2.5	64.43	0.36	65.8	0.20
3.0	48	72.17	0.3	2.5-3.5	78.24	0.38	74.3	0.20
3.0	69	72.17	0.3	2.5-3.5	78.24	0.38	74.3	0.21
3.1	43	72.84	0.3	2.5-3.5	78.24	0.38	74.7	0.21
3.2	47	73.52	0.3	2.5-3.5	78.24	0.38	75.2	0.21
3.9	61	78.27	0.3	3.5-4.5	86.31	0.26	82.5	0.21
4.0	108	78.94	0.3	3.5-4.5	86.31	0.26	82.9	0.21
4.0	47	78.94	0.3	3.5-4.5	86.31	0.26	82.9	0.23
4.0	110	78.94	0.3	3.5-4.5	86.31	0.26	82.9	0.23
4.0	39	78.94	0.3	3.5-4.5	86.31	0.26	82.9	0.19
4.0	65	78.94	0.3	3.5-4.5	86.31	0.26	82.9	0.19
4.1	65	79.62	0.3	3.5-4.5	86.31	0.26	83.2	0.23
5.0	127	85.72	0.3	4.5-5.5	92.89	0.29	89.2	0.23
5.0	106	85.72	0.3	4.5-5.5	92.89	0.29	89.2	0.23
5.1	93	86.40	0.3	4.5-5.5	92.89	0.29	89.5	0.24
5.1	112	86.40	0.3	4.5-5.5	92.89	0.29	89.5	0.21
5.1	109	86.40	0.3	4.5-5.5	92.89	0.29	89.5	0.21
6.4	87	95.21	0.3	5.5-6.5	109.42	0.36	100.1	0.21
6.4	151	95.21	0.3	5.5-6.5	109.42	0.36	100.1	0.22
6.5	117	95.89	0.3	6.5-7.5	112.12	0.25	104.3	0.22
6.5	123	95.89	0.3	6.5-7.5	112.12	0.25	104.3	0.26
7.9	135	105.37	0.3	7.5-8.5	107.22	0.34	106.2	0.26
7.9	82	105.37	0.3	7.5-8.5	107.22	0.34	106.2	0.23
8.0	115	106.05	0.3	7.5-8.5	107.22	0.34	106.6	0.23
9.4	92	115.54	0.3	8.5-9.5	131.58	0.41	120.2	0.20
9.5	103	116.22	0.3	9.5-	119.84	0.31	117.9	0.20
9.5	156	116.22	0.3	9.5-	119.84	0.31	117.9	0.20
9.5	200	116.22	0.3	9.5-	119.84	0.31	117.9	0.20
11.0	99	126.38	0.3	10.5-	126.19	0.32	126.3	0.20
11.1	76	127.06	0.3	10.5-	126.19	0.32	126.7	0.21
12.4	175	135.87	0.3	11.5-	154.26	0.50	139.9	0.21
12.4	179	135.87	0.3	11.5-	154.26	0.50	139.9	0.21
14.0	108	146.71	0.3	13.5-	144.41	0.36	145.7	0.21
14.2	67	148.07	0.3	13.5-	144.41	0.36	146.5	0.21

Table 4.10. Bayesian updating of data for the Wimbledon case study site.

Depth of Sample (m)	Test result (kN/m^2) (Derived Value)	Site mean strength from regression line at depth of sample (kN/m^2)	Site Coefficient of variation	Relevant depth band (m)	Database mean for depth band (kN/m^2) (*a priori* data)	Database Coefficient of variation (*a priori* data)	Updated mean strength at depth of sample (kN/m^2)	Updated Coefficient of variation
5.3	106	111.18	0.3	4.5-5.5	100.08	0.29	100.1	0.21
7.1	117	127.99	0.3	6.5-7.5	117.55	0.25	117.6	0.09
8.3	92	139.20	0.3	7.5-8.5	121.07	0.34	121.1	0.24
9.2	83	147.61	0.3	8.5-9.5	141.15	0.41	141.2	0.29
9.2	83	147.61	0.3	8.5-9.5	141.15	0.41	141.2	0.29
9.2	191	147.61	0.3	8.5-9.5	141.15	0.41	141.2	0.24
10.4	234	158.82	0.3	9.5-10.5	134.34	0.31	134.3	0.25
10.9	228	163.49	0.3	10.5-	141.30	0.32	141.3	0.25
15.2	113	203.65	0.3	14.5-	198.21	0.51	198.2	0.33
16.2	240	212.99	0.3	15.5-	189.71	0.48	189.7	0.13
16.5	353	215.80	0.3	16.5-	192.17	0.45	192.2	0.34
20.7	265	255.03	0.3	20.5-	205.30	0.33	205.3	0.05
23.6	191	282.12	0.3	23.5-	244.41	0.30	244.4	0.22
24.5	289	290.52	0.3	24.5-	292.75	0.16	292.8	0.01
24.7	311	292.39	0.3	24.5-	293.17	0.16	293.2	0.06

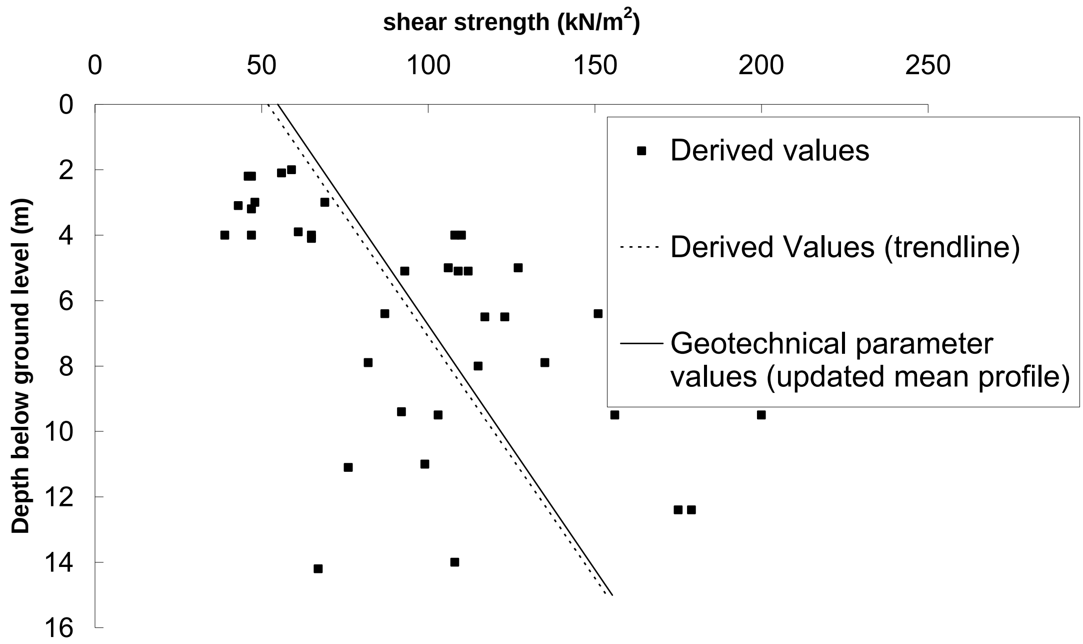

Figure 4.20. Bayesian updating of derived values of shear strength versus depth below ground level from the Woolwich site.

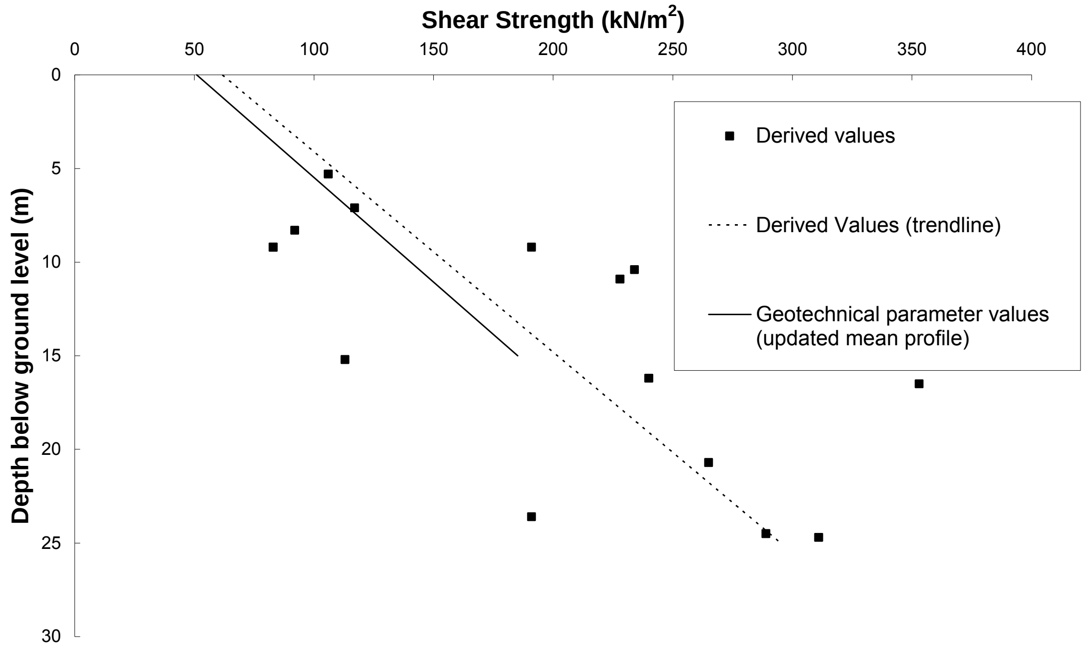

Figure 4.21. Bayesian updating of derived values of shear strength versus depth below ground level from the Wimbledon site.

4.2.1.2 Determination of Characteristic Values

From the geotechnical parameter values calculated in section 4.2.1.1 (the updated or posterior values which result from the Bayesian updating), the characteristic values can be calculated by applying the approximation proposed by Schneider (2001). This has been done for the untreated and the posterior data for comparison. The results are shown in

Table 4.11 and displayed graphically in Figure 4.22 for the Woolwich site and in Table 4.12 and Figure 4.23 for the Wimbledon site.

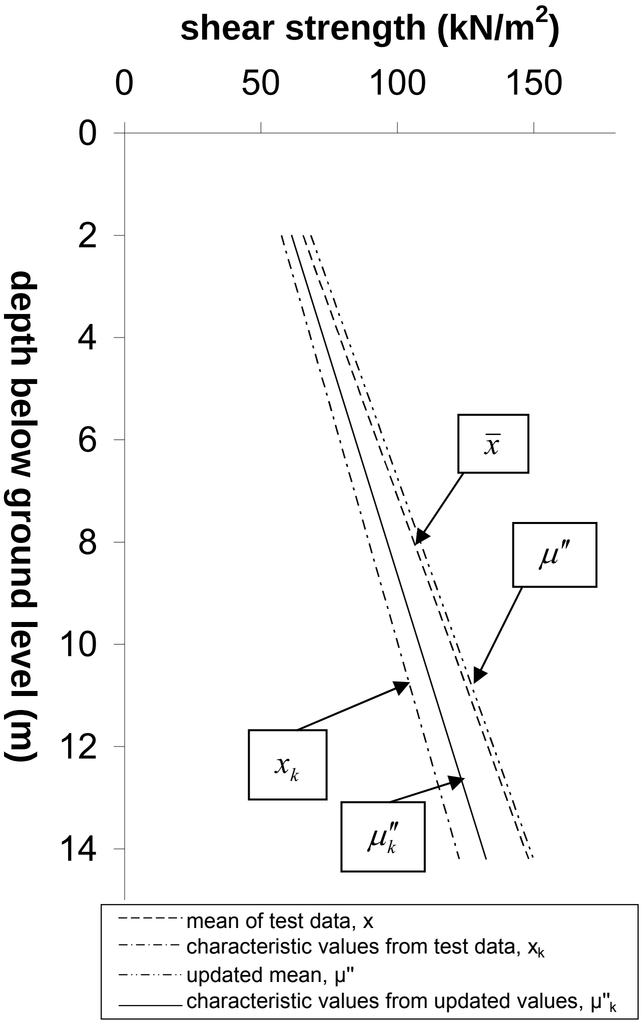

Figure 4.22. Determination of characteristic values of shear strength with depth, by Schneider's approximation, Woolwich case study.

Table 4.11. Calculation of characteristic values pre- and post-Bayesian updating for the Woolwich site.

Depth (m)	Site mean strength from regression line at depth of sample (kN/m^2)	Characteristic value (without updating)	Updated (geotechnical parameter) value	Characteristic value of updated data
2.0	65.39	62.2	65.0	57.5
2.1	66.07	61.0	65.4	57.8
2.2	66.75	56.4	65.8	58.2
2.2	66.75	56.9	65.8	58.2
3.0	72.17	60.1	74.3	65.5
3.0	72.17	70.6	74.3	65.5
3.1	72.84	57.9	74.7	65.9
3.2	73.52	60.3	75.2	66.3
3.9	78.27	69.6	82.5	74.4
4.0	78.94	64.4	82.9	74.7
4.0	78.94	63.0	82.9	74.7
4.0	78.94	63.4	82.9	74.7
4.0	78.94	59.0	82.9	74.7
4.0	78.94	72.0	82.9	74.7
4.1	79.62	72.3	83.2	75.1
5.0	85.72	65.1	89.2	79.9
5.0	85.72	75.6	89.2	79.9
5.1	86.40	83.1	89.5	80.2
5.1	86.40	73.6	89.5	80.2
5.1	86.40	75.1	89.5	80.2
6.4	95.21	91.1	100.1	88.6
6.4	95.21	67.3	100.1	88.6
6.5	95.89	85.3	104.3	94.3
6.5	95.89	82.3	104.3	94.3
7.9	105.37	90.6	106.2	94.2
7.9	105.37	93.7	106.2	94.2
8.0	106.05	101.6	106.6	94.6
9.4	115.54	103.8	120.2	105.7
9.5	116.22	109.6	117.9	105.3
9.5	116.22	96.3	117.9	105.3
9.5	116.22	74.3	117.9	105.3
11.0	126.38	112.7	126.3	112.5
11.1	127.06	101.5	126.7	112.8
12.4	135.87	116.3	139.9	121.9
12.4	135.87	114.3	139.9	121.9
14.0	146.71	127.4	145.7	129.0
14.2	148.07	107.5	146.5	129.7

Table 4.12. Calculation of characteristic values pre- and post-Bayesian updating for the Wimbledon site.

Depth (m)	Site mean strength from regression line at depth of sample (kN/m^2)	Characteristic value (without updating)	Updated (geotechnical parameter) value	Characteristic value of updated data
5.3	111.18	108.6	100.1	89.6
7.1	127.99	122.5	117.6	112.4
8.3	139.20	115.6	121.1	106.6
9.2	147.61	115.3	141.2	120.5
9.2	147.61	115.3	141.2	120.5
9.2	147.61	125.9	141.2	124.3
10.4	158.82	121.2	134.3	117.9
10.9	163.49	131.2	141.3	124.2
15.2	203.65	158.3	198.2	165.4
16.2	212.99	199.5	189.7	177.0
16.5	215.80	147.2	192.2	159.8
20.7	255.03	250.0	205.3	200.4
23.6	282.12	236.6	244.4	217.4
24.5	290.52	289.8	292.8	292.0
24.7	292.39	283.1	293.2	284.5

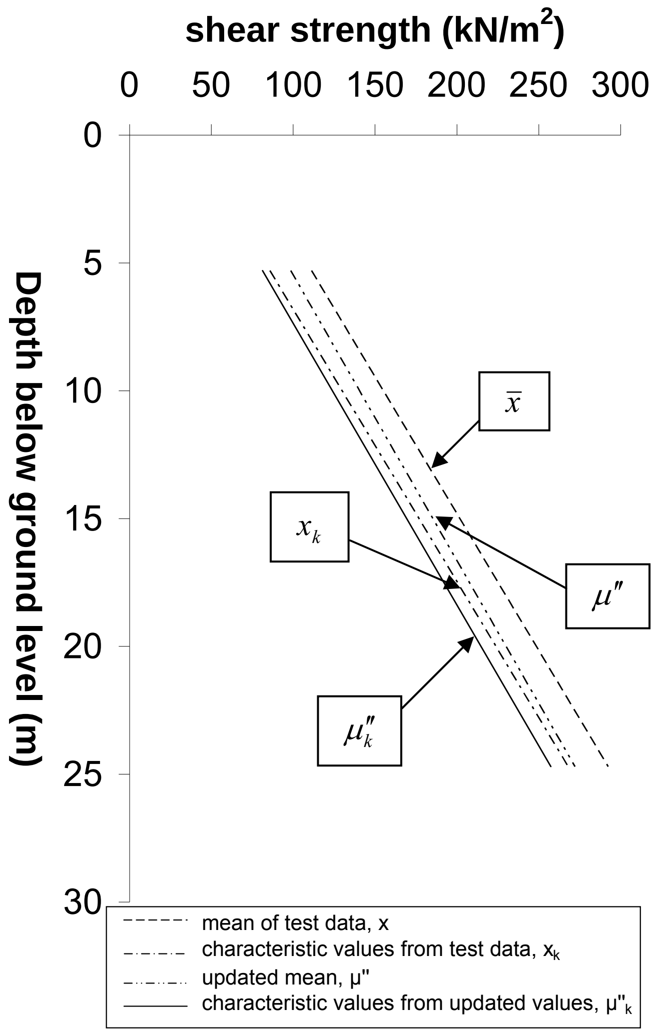

Figure 4.23. Determination of characteristic values of shear strength with depth, by Schneider's approximation, Wimbledon case study.

4.2.1.3 Significance of Bayesian Updating for Pile Design

The alteration made to the strength depth profile has an immediate effect on pile design, this can be best illustrated through example. The case study at Woolwich is used for the following example.

To carry a compressive, load of 350kN, a typical solution would utilise CFA piles of 500mm diameter. Such a pile would need to be 14m long (measured from ground level) using the unrefined values. In this design it is assumed that there is no contribution to shaft capacity from the overlying fill material (which extends to 2m below ground level) and the adhesion factor, α, is taken as 0.5 in the London Clay. If the updated values of shear strength are used, the pile would need to be 13.2 m, i.e. 800mm shorter. The cumulative volume saved over a number of piles (120 in the case of this example) would produce significant cost savings in terms of materials volume (18.9 m^3 of concrete) and time for construction. Savings have been estimated to be of the order of 3-4% of the actual pile construction costs. Although such significant savings may have been found by fine-tuning and manipulating subjective interpretations of the variables based on experience and engineering judgement (e.g. selection of design line through site shear strength data), the approach presented in this research allows such savings to be found in a justifiable, objective manner. It is clear however that such direct savings may not always be realised, such as in the case of the Wimbledon case study where the effect of the Bayesian updating was to lower the mean strength at a given depth. The coefficient of variation is also lowered, however, through the Bayesian updating leading to greater certainty in the soil strength and an associated reduction in risk. Economies in pile design can then be found through adoption of a lower factor of safety, which results in shorter piles. The advantage of conducting Bayesian updating lies largely in the reduction of uncertainty and associated risk. This can be demonstrated through probabilistic comparison as discussed in section 4.2.1.4, the factor of safety can therefore be justifiably lowered.

4.2.1.4 Additional Case Studies

Similar analyses were carried out at 8 further sites to investigate the effect of conducting Bayesian updating. At each site a 450mm diameter pile was designed to carry 750kN using three different soil strength profiles. In each case the adhesion factor, α, was taken as 0.6 and a factor of safety of 2 was applied. The base resistance was calculated using a bearing capacity factor N_c of 9. The soil strength profiles used are the characteristic values found from a least squares regression of the site test data without any Bayesian updating, a similar set of characteristic values following application of the Bayesian updating and finally a design using the engineer's initial subjective interpretation of the ground conditions with no knowledge of the Bayesian updating techniques. The results are shown in Table 4.13.

Table 4.13. Case studies of pile required to carry 750kN using soil strengths determined by three methods.

Site name	Number of piles	Depth of 450 mm diameter pile (m)		
		using regression line without Bayesian updating	using regression line with Bayesian updating	using engineer's subjective interpretation
Woolwich	660	17.3	16.4	12.8
Wimbledon	1136	15.4	15.9	15.4
Wandsworth Bridge Road	532	18.9	18.3	16.3
Wembley	72	15.1	15.1	14.9
Edmonton	172	16	16	16.3
Battersea	451	17.6	17.4	16.2
Heathstan Road	503	16.9	16.6	17.5
Leytonstone	154	15.9	16.4	14.9
Kingston	85	16.4	16.4	14.1
Nine Elms	36	19.3	18.7	18.8

It can be seen that the Bayesian updating results in shorter piles at some sites yet longer piles at others. This can be explained by consideration that some sites will have soil strengths above average and some below as the effect of the Bayesian updating is to move the mean values closer to the average. At three sites there is no change, this is a result of the soil strengths being close to the average and with little variability, hence there is little change in the average soil strengths.

In all but two of the cases studied, the pile length using the engineer's interpretation is closer to the length calculated following Bayesian updating than the length calculated from the untreated regression line. This suggests that there is an element of subjective application of experience by the engineer when interpreting the data; the selection of the design line by the engineer appears to be influenced by a pre-conceived expectation based on experience of the strength of the soil. The large variation present however suggests that this subjective interpretation can be unreliable.

It is important to note that the solution using the engineer's interpretation is shorter than the solution using statistical approach in 7 of the case studies. This shows that in these cases a higher risk is being taken in the design and there would be a higher probability of failure. The increased risk cannot be fully quantified using the traditional approach and indeed it may not be known that this risk is being taken. This additional risk may have been masked in the past by the common practice of adopting large global factors of safety (2.5 to 3), failure has subsequently not occurred but the assumed probability of failure is still higher than would have been expected.

The optimum solution needs to take account of the degree of associated risk while providing the most economical solution. This is important not only in terms of cost but increasingly with regards to sustainability. Concrete production is very energy intensive with a large degree of associated carbon emission; it is therefore desirable to seek to use as little as possible in pile construction. Such considerations could lead to a change in the current design philosophy whereby the monetary cost of testing a pile is seen as less desirable than increasing the factor of safety (and hence length) of piles across the site. This change would however require a significant change in procurement methods, which are currently simplistic and favour lowest cost solutions. If sustainability and the carbon by-product of a foundation are truly considered important then this needs to become a deciding factor in the procurement process.

4.2.1.5 Probabilistic Comparison

The Woolwich site has been used again to investigate the effect of Bayesian updating on the probabilistic design of piles. To maintain clarity only shaft capacity was considered. A 450mm diameter bored pile to carry a load of 750kN is considered. It is assumed that there is no contribution to shaft capacity from the overlying fill material (which extends to 2m below ground level).

For a factor of safety, F, of 2 a pile length of 26.7m (24.7m pile length in London Clay) is required. If this dimension is fixed, the effect of variation on the factor of safety and the associated probability of failure can be calculated from the values given in Table 4.14 using the method proposed by Duncan (2000), as shown by Table 4.15. The length of the pile is strictly controlled and as a result varies by less than 50mm or approximately 0.5% of the pile length. Variation of this factor would have negligible effect on the capacity and is therefore not included in the following analysis. The most likely value (MLV) of average shear strength acting over the length of the pile is first found from the characteristic profile shown in Figure 4.19 (without updating). The standard deviation has been estimated from the variation displayed by the site investigation data. The MLV of α is taken as the mean value of alpha found by Patel (1992) (using maintained load tests) for bored piles in London Clay, the standard deviation is also calculated from the same source. The MLV of the diameter is taken as the nominal diameter, the standard deviation of the diameter is taken as 0.06 as observed in section 4.5.

Table 4.14. Design Variables.

Dataset	**Variable**	**MLV**	σ
Site data	Shear strength (kN/m^2)	97	29
	α	0.45	0.06
	D (m)	0.45	0.03
Updated data	Shear strength (kN/m^2)	103	21
	α	0.45	0.06
	D (m)	0.45	0.03

Table 4.15. Calculation of probability of failure after Duncan (2000).

Dataset	Variable		Values	Factor of safety, F	ΔF	σ_F	Mean factor of safety, $\overline{F}$	V_F	P_f (from tables)
Site data	c_u	MLV+ σ	126	2.60	1.21	0.68	2.0	0.34	2.78
		MLV-σ	68	1.39					
	α	MLV+ σ	0.51	2.27	0.55				
		MLV-σ	0.39	1.72					
	D	MLV+ σ	0.48	2.13	0.27				
		MLV-σ	0.42	1.86					
Updated data	c_u	MLV+ σ	124	2.42	0.83	0.51	2.0	0.26	0.54
		MLV-σ	82	1.59					
	α	MLV+ σ	0.51	2.27	0.54				
		MLV-σ	0.39	1.73					
	D	MLV+ σ	0.48	2.14	0.27				
		MLV-σ	0.42	1.87					

Standard deviation of F, $\sigma_F = \sqrt{(\frac{\Delta F_1}{2})^2 + (\frac{\Delta F_2}{2})^2 + (\frac{\Delta F_3}{2})^2}$

Coefficient of variation of F, $V_F = \frac{\sigma_F}{\overline{F}}$

The standard deviation of the factor of safety is equal to 0.68 and the coefficient of variation is equal to 0.34. Using the tables provided by Duncan (2000) for the probability of failure based on the Taylor series, the probability of failure p_f is 2.78%.

The effect of including case history data upon this probability of failure can be seen by re-evaluating the design using characteristic values calculated from the updated values. When design using the updated values for shear strength is considered the required pile length is reduced to 16m in order that the same overall factor of safety is achieved ($F = 2$). Furthermore, the standard deviation has been reduced through the Bayesian updating. Table 4.14 shows the MLV and standard deviation from the updated values. The probability of failure is re-calculated and the results are shown in Table 4.15. The probability of failure is now greatly reduced and is below 1%. It is difficult to define an acceptable target for the probability of failure as this must be considered in conjunction with the consequence (financial or otherwise) of failure. Whitman (1984), however, showed the level of accepted risks for foundations to be in the range 0.1 to 1%. This is reasonable given the ability of a foundation arrangement to redistribute loads between piles.

4.3 DEVELOPMENT OF DESCRIPTIVE FRAMEWORK FOR BD PILES

The term 'bored displacement' covers a broad variety of pile types as discussed in section 2.4.2. A classification system is required to allow comparison between pile types and discussion of their relative merits. The classification system needs to avoid misleading or confusing terminology and the use of proprietary names for systems. This section describes the thought processes which led to the development and introduction of such a classification system and the workings of that system.

The development of BD piles, particularly in the UK, has been largely contractor driven. This has led to use of conflicting and confusing nomenclature and association of systems with 'trademark' names. A classification system is required which avoids all such association with proprietary systems and which gives clear description of the piles properties and expected behaviour of the pile and of the soil. The following classification system was produced by consideration of the installation processes which have been described further in section 2.4.2 and in Paper 3 (Appendix C).

It is considered that the first part of the classification should indicate the degree of soil displacement undergone during the construction. This is consistent with the classification of driven displacement piles and is important in understanding the behaviour of a pile and the state of the soil. A pile formed by inserting and back screwing an auger such as that shown in Figure 2.9 would be a low displacement pile, the displacement being limited to the diameter of the stem. A cylindrical pile formed entirely by displacement, using for example an auger similar to that shown in Figure 2.11 would be high displacement.

Further classification is provided by a description of the amount of spoil produced by a pile. This will also provide information related to the behaviour of the pile. A third generation pile (defined in section 2.4.2) that fully displaces soil to form a cylindrical pile will perform differently to a cylindrical pile formed by flighting and extracting material using a first or second generation type. Although similar to the previous categorisation, the classification proposed in this research provides information relating to the proportion of soil displaced to that removed from the bore. The classification proposed by this research is also particularly useful from a Client, or end-user, point of view as it provides clear information on the tangible effects of the process.

Behaviour and capacity of the pile is strongly influenced by the final shape of the pile formed. The second generation piles (Van Impe, 2001) can produce heliform piles by back screwing on extraction such that the auger follows the same path as on insertion. Alternatively the same augers can be used to provide a cylindrical pile; this will produce differences in load capacity, permissible internal stresses and settlement characteristics, these methods of pile construction have been described in section 2.4.2. The classification proposed by this research also allows the pile volume to be deduced form the auger dimensions. This simple classification system is summarised by Figure 4.24.

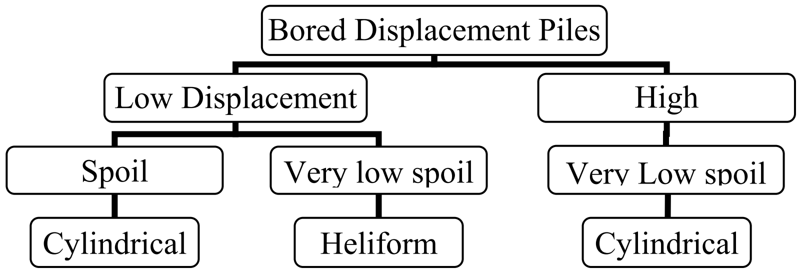

Figure 4.24. Bored Displacement Pile Classification.

This classification system was first proposed in Paper 3. Since publication, the classification systems has been used and commented upon by industry practitioners (e.g. Bourne-Webb, 2008).

4.4 BORED DISPLACEMENT PILE PERFORMANCE

During installation of BD piles, the soil undergoes a complex series of processes, as described in paper 3. While some attempts to capture the strain paths undergone by the soil have been investigated by others (Ng, 2008; Hird 2008) the effect on the pile performance of the complex processes caused by BD pile installation is not understood. The results of pile load tests can be analysed to obtain values for the empirical factors, such as the adhesion factor (α), which can be subsequently used in design calculations for other piles of the same type in the same or similar ground conditions.

This section describes pile load tests on BD piles at two sites and a comparable CFA pile at one of the sites, these tests are further described in Paper 4 (Appendix D). Both are sites where the ground conditions are predominantly London Clay and where high displacement, very low spoil, cylindrical piles have been installed and tested. The sites are typical of those which might be encountered in the London Basin; in each case BD techniques were used to construct the test piles and working piles to act as load bearing foundations for the relevant developments. The same plant was used to construct piles at both sites.

4.4.1 GROUND CONDITIONS

The first site is the Wimbledon site first described in section 4.2.1. The second site is in Chessington where the ground conditions at this site are largely similar to the Wimbledon study and generally consist of up to 0.7m of fill, over London Clay. The clay is weathered to a depth of 6.5 m. There were 5 investigative boreholes at this site with *in situ* SPT tests which have again been correlated to values for undrained shear strength using the relationship derived in section 4.1.7.3 these are shown in Figure 4.25. The depths are measured below a site datum which is equal to ground level at the highest point.

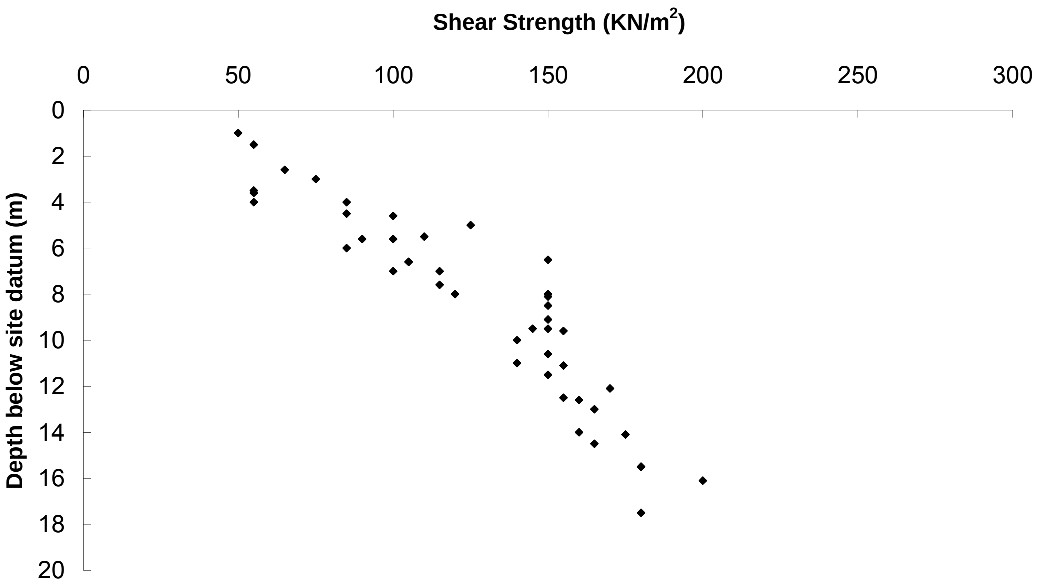

Figure 4.25. Undrained shear strength against depth below site datum for site at Chessington.

4.4.2 TEST PILES

Two BD and one CFA test piles were constructed and tested at the Wimbledon site (WBD1, WBD2 and WCFA). One BD test pile was constructed at Chessington (CBD). The piles were constructed as 400mm diameter piles with depths as shown in Table 4.16. The tests were of the maintained load (ML) type and were carried out in general accordance with the ICE Specification for Piling and Embedded Retaining Walls (ICE, 1996). The load settlement behaviour is shown in Figure 4.26 for the Wimbledon BD piles, Figure 4.27 for the Wimbledon CFA pile and Figure 4.28 for the Chessington BD pile. The behaviour of these piles is discussed in section 4.4.3.

Table 4.16. Test pile dimensions.

Pile	Diameter (m)	Depth (m)
Wimbledon BD 1 (WBD1)	0.4	15.2
Wimbledon BD 2 (WBD2)	0.4	16
Wimbledon CFA (WCFA)	0.4	16.2
Chessington BD (CBD)	0.4	18

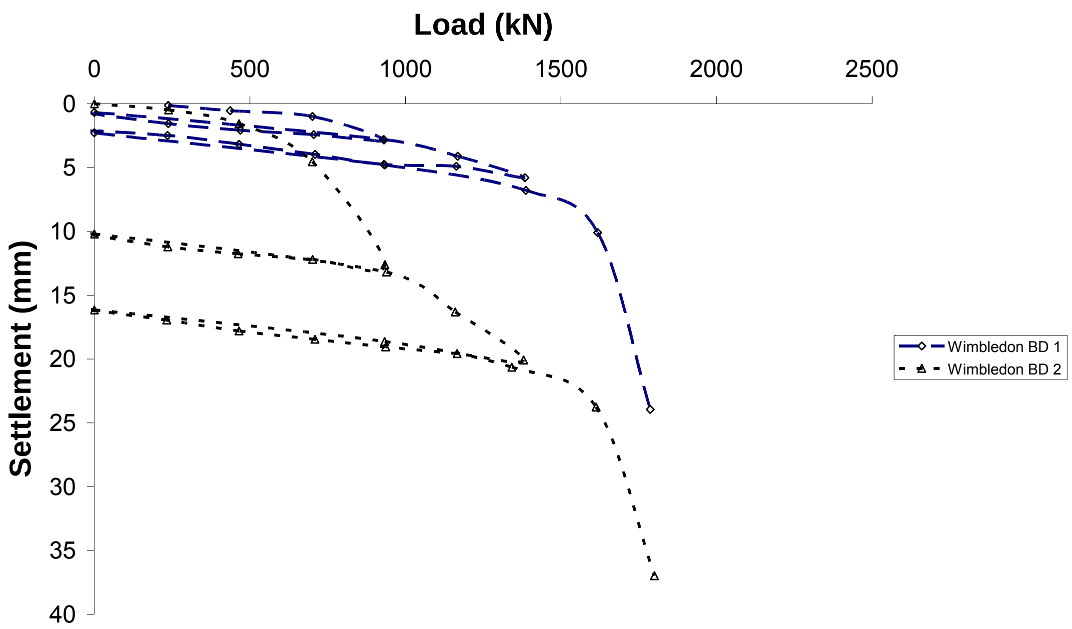

Figure 4.26. Load settlement curves for the Wimbledon BD test piles.

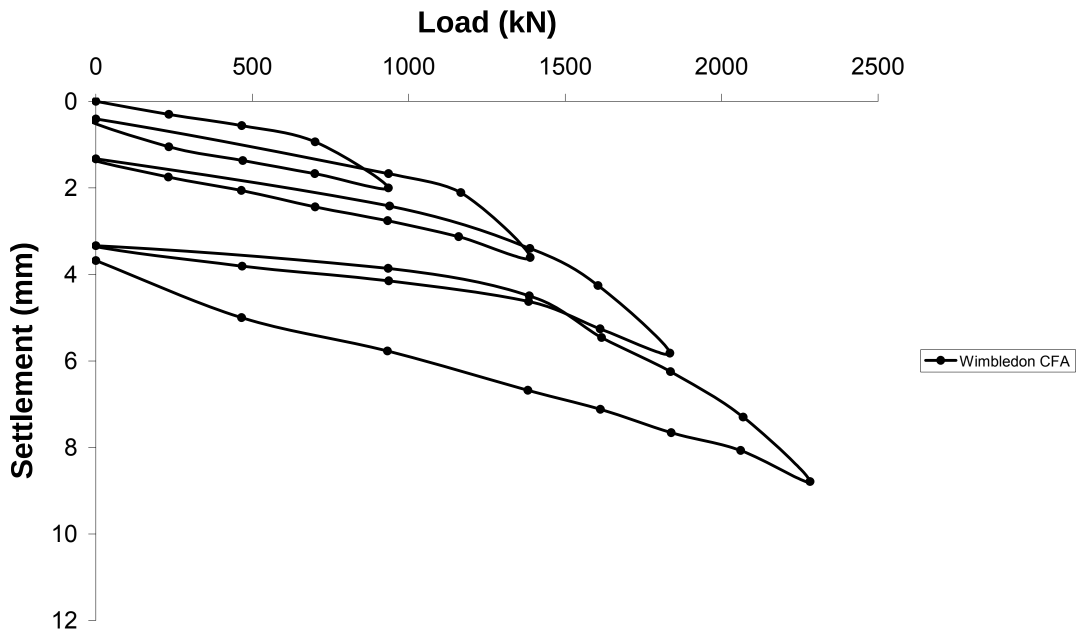

Figure 4.27. Load settlement curves for the Wimbledon CFA test pile.

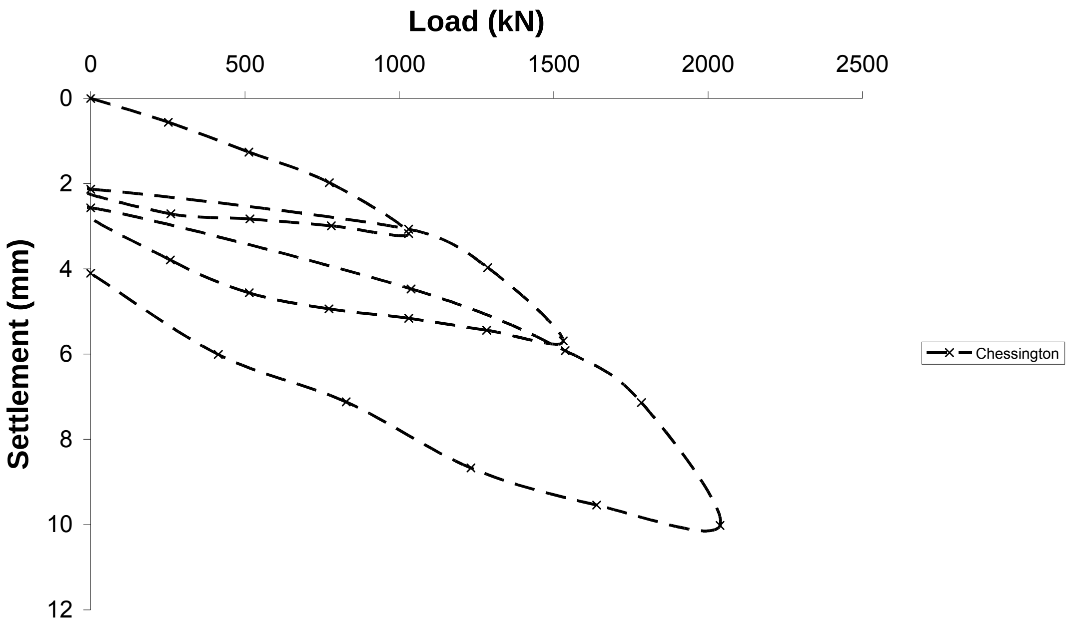

Figure 4.28. Load settlement curves for the Chessington BD test pile.

4.4.3 PILE CAPACITY AND BACK ANALYSIS

The limiting capacity for Wimbledon BD1 has been taken as 1700kN. Wimbledon BD pile 2 can be observed to enter into a plunging mode of failure at 1800kN. The capacity of the Wimbledon CFA pile is shown to be in excess of 2280kN.

In the case of the Chessington BD pile, the maximum bearing resistance of the soil has not been exceeded; the test pile has therefore not reached geotechnical failure (ultimate limit state). The highest capacity demonstrated by this test pile is 2000kN. Limitations on the allowable concrete stress and structural integrity of the pile precluded testing to higher loads,. It was not anticipated, however, that significantly higher capacity would be achieved. Using the extrapolation method proposed by Chin (1978), the capacity was found to be approximately 2200kN.

From these values of capacity, back analysis has been conducted to yield observed values for the adhesion factor, α . The piles have not been instrumented and hence it is not possible to obtain separate values for the load carried by the shaft and by the base of each pile. The end bearing capacity is estimated from Equation 1.2 taking N_c to be 9. This is then subtracted from the total capacity to leave the shaft capacity. With bored displacement piles there is not the potential for over excavation that exists with CFA and hence the diameter is assumed to be constant and equal to the nominal diameter. Experience and observations of partially exposed BD piles has demonstrated this to be a fair assumption. The adhesion factor α can be obtained from the shaft capacity and Equation 1.3 and Equation 1.4. The shear strengths for the soil have been taken from the design engineer's original interpretation of the soil conditions. While the soil strength varies about this interpretation, using this assessment of the soil strength is representative of commercial practice and leads to results which can then be applied in an industrial context. The results of this analysis are shown in Table 4.17. The adhesion factors achieved by the BD piles are comparable to a CFA pile and above those values typically permitted for design (LDSA, 2000).

Table 4.17. Calculation of apparent adhesion factor.

Test Pile	Pile depth (m)	Cohesion (kN/m^2)		Observed capacity (kN)	Calculated end bearing (kN)	Shaft capacity (kN)	Adhesion Factor
		At base	Average over shaft				
WBD1	15.2	178	111	1700	200	1500	0.70
WBD2	16	185	115	1800	210	1590	0.69
WCFA	18	204	125	2280	230	2050	0.73
CBD	16.2	206	141	2200	213	1987	0.69

The adhesion factors calculated here demonstrate that similar pile capacity can be expected for BD piles and CFA piles in London Clay. For design purposes, current practice is to adopt an adhesion factor of 0.6 for CFA piles. Fewer case studies have been conducted for BD piles to date and it is necessary to demonstrate the adhesion factor at each site through load testing. The values found from this research, however, provide a useful estimate for preliminary design and design of test piles.

4.5 VARIATION OF PILE DIAMETER

This section describes the findings of the investigation into how the diameter of a pile, as-constructed, varies in comparison to its nominal, intended diameter. Without exhuming a pile for measurement, it is impossible to ascertain accurate, as-constructed dimensions. The actual as-constructed diameter of the pile at any depth can be estimated from the concrete volumes injected. Computerised rig logs provide profiles of concrete volumes injected against depth. These logs can be interrogated to yield average diameters achieved within a particular strata, if a cylindrical pile shape is assumed.

In this investigation the average diameter of the pile within the bearing strata was calculated from such estimates and compared to the nominal value. Capacity is not directly proportional to diameter; calculations of capacity based on this average diameter are hence compared with those based on the nominal diameter.

The piles used in the investigation were all monitored during construction using a proprietary computer system (manufactured by Jean Lutz SA). The system records concrete volumes supplied to each 0.08m interval along the pile length. These volumes are considered to contribute to the pile volume of the 10 intervals either side (0.8m). Calculation then yields an assumed profile for the pile based on the volume supplied at that depth and from volumes supplied above and below. This type of analysis of pile diameter is normally used for identifying defects in pile construction such as necking, where a section of pile is below the designed diameter.

Seven sites were selected for analysis. These sites were chosen for several reasons. Primarily they were of interest because the pile construction at each site has been monitored using the same technology. Each site consists of predominantly London Clay (overlain by some fill material). A range of pile diameters is represented. The piles selected are expendable test piles and therefore further information was available regarding the pile capacity and load deflection behaviour. The piles studied are described in Table 4.18.

It is considered that, for the piles used in the investigation, the capacity is provided entirely within the London Clay. Therefore the average diameter is calculated only over the length of pile embedded in the London Clay. The results of the calculations of average diameter are shown in Table 4.19. Capacity was then calculated by the methods described in section 1.1.2 using the nominal and the estimated actual diameters. A single strength profile is adopted for each site. These calculations are shown in Table 4.20

Variation in pile diameter from the nominal diameter ranges from 4 to 20% this results in a variation in pile capacity of 5 to 24% of nominal capacity. The mean ratio of estimated as built diameter to nominal diameter is 1.08, the standard deviation of this ratio is 0.06. The mean value of $Q_{actual}/Q_{theoretical}$ is 1.10. The standard deviation is 0.67.

Variation in pile diameter should be considered when analysing pile performance or calculating an observed adhesion factor, α. Using a smaller diameter than that which has been constructed would lead to α that is artificially high. For pile design, however, a conservative estimate would be to assume that pile diameter does not increase above the intended size, unless a more detailed and efficient design is sought.

The pile capacity observed through load testing is shown in Table 4.20. The capacities correspond well with the capacities calculated using the estimated as built diameters and more closely represent the capacity than calculations made using the nominal diameter.

Table 4.18. Piles used in the investigation into variation of CFA pile diameter.

Location	**Depth to Clay (m)**	α	c_u average (kN/m^2)	base (kN/m^2)	N_c	**Pile Length (m)**
Chalk Hill, Wembley	3.4m	0.6	152.6	205.2	9	22.7
Boundary Road	2m	0.6	172.95	232	9	23.9
Paddington	3m	0.6	167.81	250	9	24.9
Kingston	1m	0.6	155	200	9	13.7
Hendon	2.5m	0.6	107	265	9	20.5
Payne Road Bow	5m	0.6	131	174	9	25.2
Camden	4.1m	0.6	123.19	187.38	9	21.2

Table 4.19. Diameter variation.

Location	**diameter**		**Estimated as built diameter** / **Nominal diameter**
	Nominal (m)	Estimate of as built diameter from concrete volume (m)	
Chalk Hill, Wembley	0.4	0.44	1.10
Boundary Road	0.6	0.63	1.05
Paddington	0.5	0.52	1.04
Kingston	0.4	0.44	1.10
Hendon	0.35	0.42	1.20
Bow	0.6	0.63	1.05
Camden	0.4	0.43	1.08

Table 4.20. Effect of diameter variation on pile capacity and capacities determined through pile load test.

Location	Q_s **(kN)**		Q_b **(kN)**		Q_u **(kN)**		**Pile load test capacity (kN)**	**Ratio of as built to nominal capacity**
	nominal	**as built**	**nominal**	**as built**	**nominal**	**as built**		
Chalk Hill, Wembley	2497	2746	232	281	2729	3027	3200	1.11
Boundary Road	4088	4292	590	651	4678	4943	5050	1.06
Paddington	3464	3602	442	478	3905	4080	4200	1.05
Kingston	1484	1633	226	274	1710	1906	1950	1.11
Hendon	1270	1524	229	330	1499	1854	2200	1.24
Bow	2991	3141	443	488	3434	3629	3850	1.06
Camden	1588	1707	212	225	1799	1931	1950	1.07

4.6 INSTALLATION OF BORED DISPLACEMENT PILES

The process of installing BD piles and its effect on soils is not fully understood. In this section an investigation into the drilling parameters is presented; this is described in further detail in Paper 4. A large amount of information relating to the forces exerted and speeds achieved during pile installation is available from the instrumentation on-board the piling rig. This information can be used to compare the installation of pile types or to compare pile installations between sites. This can yield valuable information relating to the speed of pile construction, energy required and constructability of pile types in different soils.

Figure 4.29 shows information recorded by the rig instrumentation during the auger insertion stage for two 400mm diameter, 16.1m deep BD piles (one at each of the Wimbledon and Chessington sites) and a CFA pile (at the Wimbledon site). The CFA pile was also 400mm in diameter, and was 18m in depth. The CFA pile was constructed with the same plant and equipment as the BD piles, with the exception of the auger.

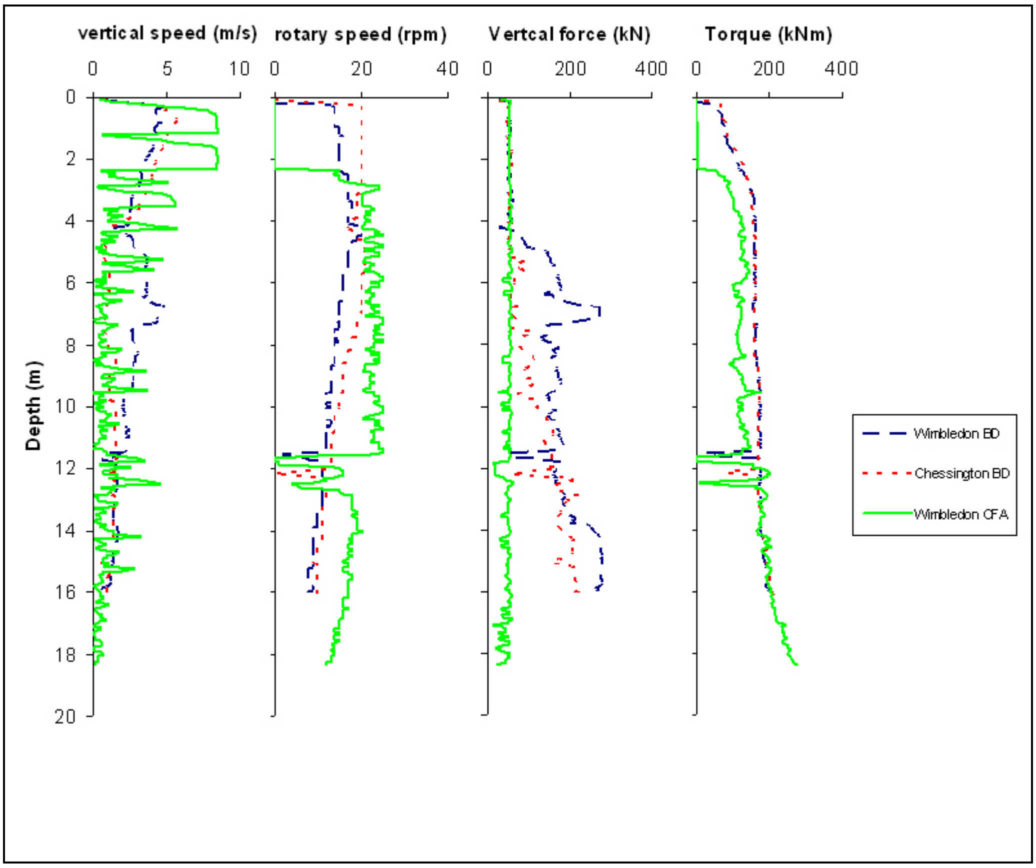

Figure 4.29. Piling Rig logs from the drilling phase of the piles.

4.6.1 INTERPRETATION OF PILING RIG LOGS

When analysing the logs of the pile construction there is a notable interruption to all of the parameters at about 11 to 11.5m in all three cases. This is simply a function of the necessary pause in the drilling to allow for the engagement of an auger extension.

In all cases there is an increase in torque required with depth until the practical maximum operating limit is approached at about 4m, coinciding with the auger reaching clays which are of higher strength. The rotary speed for the BD remains roughly constant until the practical maximum torque is reached; once this has been achieved the rotary speed falls with depth (and increasing soil strength). In order to maintain progress of the auger into the ground it is necessary for the operator to engage the pull down winch. The vertical (downward) force required is constant until the maximum torque has been reached; beyond this point the vertical force required begins to increase with depth.

The vertical (downward) speed of the CFA pile auger is more erratic than for the comparable BD piles. This is a result of the need to pause to allow soil to travel up the flight for removal in CFA pile construction. The CFA pile log shows an erratic pattern which is symptomatic of this repeated slowing of the auger to allow the soil to travel up the flight. The vertical speed of the BD piles reduces as the depth increases; this is a result of the increasing strength of clay which is encountered. The rotary speed is consistent for the BD piles up until the point where the soil becomes of sufficient strength that vertical force is required to advance the auger (and where the torque approaches its practical maximum limit). As a function of how power is supplied to drive elements of the rig, rotary speed must be sacrificed for the additional vertical force required. The vertical force is approximately constant through the softer, upper material and is comparable to that used throughout the CFA pile construction. This is essentially the self weight of the auger.

The production rates of BD piles and CFA are similar; downward speed is similar as is the time taken to concrete a pile. CFA piles can achieve faster rotation for a given rotary pressure as cutting the soil uses less energy than displacement in these ground conditions. The main observable difference is in the vertical forces which are much lower for CFA. This is thought to be a function of the continuous helical shape of the auger which transforms the torque into a downward force which effectively pulls the auger into the ground. Although the flight on the lead section of the displacement auger has a similar effect, it does not provide sufficient downward force to advance the displacement auger downwards and displace soil laterally. The additional energy required for the displacement is then provided from the application of relatively higher vertical force.

4.6.2 ENERGY USED TO CONSTRUCT THE PILES

A measure of the energy used to install the pile can be obtained from the rig monitoring data using the formula proposed by Van Impe (1998) for the screwing in energy, E_s, shown in Equation 4.1.

$$E_s = \frac{N_d v_i + n_i M_i}{\Omega v_i}$$ **Equation 4.1**

Where N_d is the vertical thrust of the auger, v_i is the vertical speed of the auger, n_i is the rotational speed of the auger, M_i is the torque applied and Ω is the area of the borehole. In each case the subscript "i" refers to the screwing in phase. Summary statistics of each parameter for each of the piles are listed in Table 4.21 An average screwing in energy is calculated from these mean values to facilitate comparison. The values shown in Table 4.21 are for the whole pile. A similar analysis has been conducted using only the section of pile in London Clay, this is detailed in Paper 4.

Table 4.21. Summary statistics of drilling phase parameters in London clay.

Parameter		Bored Displacement		CFA
		Chessington	Wimbledon	Wimbledon
Downward speed (mm/min)	Mean	1.92	2.65	1.86
	σ	1.26	0.93	1.20
Torque(kNm)	Mean	156.8	154.8	131.9
	σ	27.08	16.56	49.08
Rotary Speed (rpm)	Mean	15.86	13.29	16.85
	σ	4.17	3.33	4.69
Pull down Force (kN)	Mean	108.5	150.7	50.0
	σ	58.65	65.5	10.31
Screwing in energy (kJ/m^3)	(from mean values)	1035	1302	557

It can be seen that a CFA pile uses less energy during the construction of the void. At the Wimbledon site the mean screwing in energy of a CFA pile is approximately half of that for a BD pile. This is a result of the energy required to displace the soil in the case of the BD pile.

A measure of the typical variation across a site was found by selecting a sample of 20 piles at each site and calculating the mean screwing in energy and the standard deviation for the sample. The mean for the BD piles at Chessington was 971.5 kJ/m^3 with a standard deviation of 153. The mean for the BD piles at Wimbledon was 983.5 kJ/m^3 with a standard deviation of 102.4. The average energy at the two sites is very similar despite the energy for the piles shown in Table 4.21 being distinctly different, in particular the Wimbledon pile. Without further information relating to the construction of this pile it is impossible to ascertain why this pile used such a disproportionate amount of energy. Some of the variation may be due to natural variability in the ground conditions. The coefficient of variation observed in London Clay in section 4.1 was up to 0.5. Further variation may be due to the factors discussed in section 2.6, in particular the contractor skill and experience, and the level of workmanship. It is worth noting that the test pile at the Wimbledon site was constructed very early in Rock and Alluvium's experience of this piling technique in this soil type.

The energy used to construct the BD piles differs between the piles at the two different sites and high degrees of variation have been observed in all of the variables and indeed in the calculated energies. Pile construction is a subjective process; techniques employed by individual piling operatives may vary as they, and the plant, react to the particular ground conditions. In order to advance an auger, different combinations of vertical force, torque and rate of auger advancement can be employed and may differ between operators, rigs and auger designs and direct comparison is therefore difficult. This is compounded by the fact that some actions will expend energy without achieving any useful gain; effectively the same borehole could be constructed using different amounts of energy. Further work is required to ascertain the most effective and efficient combinations in different soil conditions. This would result in guidance for the best construction practice. Investigation into the effectiveness of different auger advancement techniques was beyond the scope of this research.

The application of a measure for the installation energy makes no distinction between energy which is usefully employed in advancing the auger or constructing the pile and that which may not have any benefit. Over-rotation of an auger (CFA or BD), for example, expends energy while not having any beneficial effect on the pile construction, possibly even having a detrimental effect such as causing excess material to removed from the bore. Further refinement of the calculation is required to allow for such non-beneficial energy expenditure and the different techniques that can be employed by the operator to advance the auger.

While BD piles consume more energy in their construction (in terms of the energy expended by the piling rig), they still offer overall savings as there is a greatly reduced amount of spoil. Handling, transportation and disposal of this spoil are all energy intensive processes. There is therefore scope for savings to be made by utilising BD piling techniques.

5 FINDINGS AND IMPLICATIONS

5.1 KEY FINDINGS

The purpose of this section is to summarise the key findings of the research. The presentation of these findings follows the order of the objectives presented in section 1.4. The findings of the literature and the investigation into the current state of the art of CFA and BD piles has been presented in section 2 and has informed the remainder of the research.

5.1.1 QUANTITATIVE METHODS FOR USING PREVIOUS EXPERIENCE

The inclusion of previous experience and existing knowledge is an important step in the process of characterising ground conditions. Prior information is often incorporated into the assessment of the soils for design by subjective means and while this remains a valid approach, objective methods provide a more rigorous solution and should be used wherever possible. Prior knowledge can be incorporated into a model of the ground conditions quantitatively using Bayesian updating. This has been demonstrated and illustrated through application to a case study in Paper 1 (Appendix A).

In the objective approach used in this research, strength depth relationships were represented using a linear regression. This differs from the subjective approach often used in engineering practice Using a subjective approach to construct a line allows extreme values to be easily omitted or different weighting may be given to some of the points. Strict application of the objective approach described here, leaves little room for such subjective weighting of data. Although errors may be introduced into the general dataset by the inclusion of all data, including the extreme values, the influence of these errors is minimised by applying equal significance to each data point. Methods for dealing with rogue results in the site specific investigation data are discussed further in section 5.2.2.2. It must be stressed that engineering judgement is not completely removed from this process and it should not be applied blindly. The critical application of engineering judgement is in deciding on a suitable database of existing knowledge and the decision as to whether or not the site in question is really part of the same statistical population.

5.1.1.1 Suggested Procedure for Objective Application of Experience

The following steps should be followed when using previous knowledge to enhance models of the ground conditions in an objective manner. This procedure has been introduced to the design process at Rock and Alluvium as an optional step to improve the interpretation of ground conditions and has been successfully used following a validation process including comparison with alternative design approaches and against pile load test results.

- **Collate data and quantify experience** – It is essential to have a statistically significant amount of quantified data to represent the previous knowledge. This step is only required once for each strata/region. In this research, a database of shear strength of London Clay (in the London Basin) has been presented (Papers 1 and 2, Appendices A and B).
- **Consider applicability of dataset to site** – An assessment must be made as to the applicability of the dataset to the site in question; are the site data and the data representing the previous knowledge from the same population? In this research a linear relationship between strength and depth has been observed for the London Clay with no discernible categorisation of results by degree of soil weathering or by geographical location albeit there is a degree of variation about this trend; this dataset can be considered suitable for sites underlain by London Clay across the London Basin.
- **Remove trends/normalise then summarise data** – the Bayesian updating requires summary statistics (mean and standard deviation) for the data. For meaningful results, trends such as an increase in strength with depth need to be removed. In this research a linear increase in shear strength with depth is dealt with by sub-division of the strata into 1m thick bands which are subsequently summarised individually.
- **Consider distribution of data** – Bayesian updating assumes a normal distribution of the data, it is essential to verify that this assumption is reasonable for the data in question. The data presented in this research shows that strength values for London Clay can be assumed to be normally distributed around a linear trend of strength with depth.
- **Apply Bayesian updating** – Once the above criteria have been satisfied, the methods presented in section 3.2.2.2 can be implemented. Examples are described in Papers 1 and 2 (Appendices A and B)

5.1.1.2 Application of Updated Values

The outputs of the updating process, namely a revised mean and standard deviation, can be used to better assess characteristic values. The effect of carrying out Bayesian updating on a set of site investigation data is to produce a revised mean which is a function of the site specific data and the prior information and which is weighted by their relative variations. Bayesian updating leads to more rigorous estimate of the soil properties which would lead to a lower likelihood of failure and a solution with less risk attached (Paper 2, Appendix B). Probabilistic comparisons can be made between alternative solutions.

5.1.2 CLASSIFICATION OF TYPES OF BD PILES

Bored displacement pile types are numerous and varied, yet there have been no published case studies of BD pile installation or testing. Products and processes are often labelled inconsistently or confusingly and subject to proprietary systems. A descriptive classification system has been proposed (Paper 3, Appendix C) which differentiates by pile shape, degree of displacement and the amount of spoil produced. This system allows direct comparison of alternative systems and enables discussion. Use of this classification system is to be encouraged to avoid the confusion associated with the proprietary systems and trade names currently used.

5.1.3 BD PILE PERFORMANCE

The capacity of BD piles in London Clay is of a similar magnitude to that achieved by CFA piles (Paper 4, Appendix D) of the same size. The design of BD piles is currently based on the results of site specific pile load testing. The adhesion factor, α, for high displacement, low spoil, cylindrical BD piles was observed as 0.6 to 0.7 in London Clay. This value may be of use for preliminary design but it may vary with the construction techniques used and the ground conditions as discussed in section 1.1.2. There are numerous variations in the types of BD pile, construction methods and techniques; hence there is insufficient evidence from load tests (in this research or in the literature) to suggest guide values for empirical design values (e.g. α) for BD piles in different soil types. It is essential therefore that design of BD piles continues to be related to site (and pile type) specific pile load testing.

5.1.4 VARIABILITY OF CFA PILE DIAMETER

The actual diameter of the piles is not predictable. The diameter of the pile has been seen to increase from that which is expected (or designed for) by up to 10% No instances of the pile being less than the nominal diameter have been observed. This is consistent with routine site observations and monitoring.

For pile design purposes, the diameter is currently assumed to be equal to the nominal, intended diameter, no allowance is made for over sizing. This is a conservative approach and should continue as values for empirical parameters (such as α) currently used are based on analysis which also makes this assumption. There is scope, however, for improving knowledge of pile soil interaction by considering the actual pile dimensions when back analysing piles. There is difficulty in ascertaining actual dimensions of the pile as exhumation remains the only method for gaining accurate measurements; estimates can be made, however, from the volumes of concrete supplied during the piling operation. The use of nominal diameter in back analysis of piles leads to an overestimation of the adhesion factor.

5.1.5 BD INSTALLATION PARAMETERS

Bored displacement piles can be constructed in stiff clays at rates which are comparable to CFA piles (Paper 4, Appendix D) but there are practical considerations and limitations which must be accounted for. These include the potential for heave, the proximity with which piles can be installed, potential damage to neighbouring piles caused by the displacement and the presence of obstructions or inclusions which cause difficulties in advancing the auger. Heave of the ground is an important issue and must be accounted for not only in the context of pile reinforcement but also in terms of induced changes in ground level. Heave and ground movements have been observed and careful planning of piling operations is standard practice for Rock and Alluvium’s BD operations as is reinforcement design for tension caused by heave.

The energy used to construct BD piles was found from the case study to be greater than that for CFA piles (for comparable ground conditions and pile dimensions) and the force required to displace the soil increases with the strength of the host soils (Paper 4, Appendix D). Bored displacement piles require application of a greater vertical force to force the auger into the ground. The screwing action alone is not sufficient to draw the auger down and cause displacement. Despite the additional energy required, a BD pile is an attractive environmental solution as there is minimal spoil to transport and dispose of and contamination is left *in situ*.

BD piles are a very specialised pile type which offers a number of benefits, including low spoil volumes, low noise and vibration installation and fast construction. Their use, however, is often limited by ground conditions (dense sands, cemented deposits, cobbles and boulders are examples of problematic soils for BD piles) or other constraints such as neighbouring structures which could be affected by the soil displacements. BD piles are of most use where contamination is present that it is desirable to keep *in situ* or where spoil removal is otherwise undesirable. Uncertainties remain regarding the constructability of BD piles in different soil types. Applications in London Clay have been investigated in this research and found to be successful; trials and testing are however important in risk mitigation when considering BD piling at a new site, previous applications of BD piles have been limited to alluvial deposits and loose cohesionless soils).

5.2 DISCUSSION OF PILE DESIGN PROCESS

This section presents a discussion of the procedure of pile design and how it can be augmented on the basis of this research. The process of bored pile design is essentially iterative; geotechnical analysis, interpretation and calculation are used to determine whether a pile can resist the imposed loading and the dimensions (diameter and length) are altered until the optimum solution is found. The design relies upon the Engineer's selection of the design input parameters listed in Table 1.1. This discussion relates to the design of CFA and BD piles in cohesive soils and hence there is a concentration on the contribution to capacity from the shaft of the pile. The design input parameters have been grouped into three areas that are discussed in turn below; the dimensions of the pile, the interpretation of the ground conditions and the interaction between the pile and the soil.

5.2.1 PILE DIMENSIONS

The parameters under the Engineer's control are the pile dimensions. This is true, however, only to a limited extent. In section 4.5 variation of the pile diameter was observed. Any deviation from the intended diameter needs to be controlled where possible to ensure integrity of the pile while maintaining efficient material usage and should also be allowed for in design to ensure that the design is accurate and efficient. Current practice for pile design utilises a nominal diameter equivalent to the auger size. An approach more representative of the situation would be to consider the actual, as-constructed, diameter. This research has shown that at typically representative sites the average diameters for CFA piles in London Clay range from 4 to 20% greater than the nominal diameter (mean of 10%). Further research is required to verify this through exhumation of CFA piles, or through detailed analysis of variation of the rate of concrete supplied during auger extraction.

Pile diameter varies for a number of reasons, when cohesionless layers are encountered material may be removed by excessive flighting (over-excavation), alternatively the presence of a permeable layer may lead to an outflow of concrete/grout. Either of these cases would lead to shear keys between the pile and soil. Pile diameter may also be larger than intended due to the auger rotating eccentrically. This would result in a more uniform deviation from the intended diameter and is also a more likely explanation in London Clay as the presence of permeable or cohesionless layers is unlikely.

The variation of pile diameter may be a result of variation in ground conditions or of operator influence. The degree of operator influence needs to be quantified. Without such quantification the best possible situation is for strict quality control systems to be put in place to govern and monitor the operation and to reduce the scope for variation.

Pile length is more readily monitored and controlled. Such monitoring is required to ensure that the as-constructed length is the same as the design length. The operator can control the pile length by regulating the advancement of the auger into the ground. Furthermore, software and/or physical restraints can be used that prevent design lengths from being exceeded. For this research, the installed length was equal to the design length.

5.2.2 INTERPRETATION OF GROUND CONDITIONS

The Engineer has no control over the ground conditions present at a site. The interpretation of the site properties (as determined from the site investigation), however, can have a great influence on the pile design and is the area where the skill, experience, understanding and judgement of the Engineer are applied.

In the interpretation of ground conditions, the Engineer makes numerous decisions and judgements based on the site investigation data, previous knowledge, experience and the perceived level of risk. There are many subjective elements to these judgements. This research has developed objective tools and methods to assist the engineer in making such decisions and judgements in pile design.

Bayesian updating can be used to combine previous knowledge of ground conditions for a stratum with the site specific ground investigation data. This technique does, however, rely on possession of a quantification of existing knowledge and site investigation data that can be judged to be both sufficient in quantity and of the same statistical population as the general dataset.

The following discussion considers the steps taken by the Engineer in interpreting ground conditions in current practice and how the process might be influenced by this research.

Before interpretation of the ground conditions is carried out, an assessment of the quality of the data and its sources is required. At present, this is a subjective process, the Engineer may question whether there are a reasonable number of test results and whether the site investigation was carried out by a trusted party. Subjective comparison may also be made with the Engineer's expectation of soil properties from experience, in terms of values, trend and scatter of data. Application of Bayesian updating methods raises slightly different, more objective questions; for example, can the data be shown to be from the same population as the dataset of existing knowledge and is the quality of the site investigation acceptable with reference to current codes and standards?

The first step in interpreting ground conditions is the division of the various strata that occur. This is done by observation of the soil types and by consideration of any banding or interbedding. Seemingly incongruous results may be the result of incorrect selection of strata horizons and this should be considered before other possibilities such as the quality of sampling or testing.

Following the assignment of strata the Engineer seeks to interpret the soil properties (i.e. undrained shear strength) from the test results. Experience and geotechnical knowledge may be applied at this point to seek trends of shear strength with depth that are known or believed to occur in that deposit.

The objective approach used in this research applied the method of least squares to obtain a regression line of shear strength versus depth to represent the test results. This removes the subjective element from the process. This approach provides a precise solution that is reproducible; accuracy, however, may be affected by the quality and quantity of the data. This is explored by application of the approach to a number of design scenarios, these are;

1. There is a low amount of site data available.
2. There are incongruous, rogue test results.
3. There is bias in the test results.
4. There are seemingly good test results.

5.2.2.1 Scenario 1, Limited Site Data

Many cases have been observed where the ground investigation does not provide sufficient quality or quantity of site specific data (Egan 2008). In compiling the database described in section 4.1, the author only included sites with adequate data, 68 sites in total were used whereas 15 sites had to be excluded on the grounds of poor quality and quantity of data. Sites used had from one to eight boreholes with multiple sample points and/or tests at each borehole location. The most extreme examples of inadequate site investigation observed by the author have contained no boreholes, test data or sampling, in this instance a very conservative design was used at tender stage and further ground investigation was proposed to verify the assumptions made.

This presents the Engineer with a clear argument for obtaining more data. In practice, and in the author's experience, it is not always possible to obtain additional ground investigation data. This may be for reasons of cost, practicality or commercial restrictions. It is in such an instance that alternative sources of information are sought. Existing knowledge and experience are often called upon to supplement poor site investigation, this is currently achieved through subjective application of experience. The quantified data presented as part of this research and Bayesian updating are ideally suited to this scenario and provide an objective alternative.

5.2.2.2 Scenario 2, Poor Quality Data

Bayesian updating overcomes the presence of rogue data in the dataset by applying equal weighting to each of the data points in a very large dataset. Similarly the effect of such data points in the site investigation is also reduced by the weight of the data.

In current practice an extreme value has more significance and skews the trend line. In this case it may be necessary for the Engineer to intervene. Firstly it should be verified that the data point belongs to the same strata and not the result of an inclusion or an interbedded layer. Removal of this data point may be justified if it can be shown to be from a different soil type or condition. The rogue results may also be the result of an error in the test procedure, for example an incorrect cell pressure applied in a triaxial compression test. This can be investigated by the Engineer to arrive at a decision as to whether the result is valid. Such investigations and decisions are often made subjectively in the author’s experience.

Adoption of Bayesian updating and objective methods for determining trends, however, reduces the reliance on subjectivity and provides a rigorous structure for the interpretation of ground conditions.

5.2.2.3 Scenario 3, Biased data

Data from ground investigations can display bias that has been introduced as a function of the method of investigation. It has been observed in section 4.1.7.1 that SPT values are often truncated at 50 when full penetration is not achieved, approximately 15% of test results shown in Figure 4.13 are potentially affected. Despite this SPT values are often used in practice in place of, or to supplement, triaxial test data. The source and validity of data needs to be considered by the Engineer. The Engineer may choose to extrapolate values where full penetration has not been achieved, to use only values from shallower regions where the strengths are not so high as to present an issue, or to consider the trend to be bilinear with SPTn increasing with depth until a maximum value of 50 is reached and thereafter remaining constant. Bias can also be introduced to the results of triaxial testing through the difficulty associated with obtaining undisturbed samples. The driving of a sampling tube inevitably leads to some disturbance; this is greater in higher strength material and therefore has a greater effect with depth in London Clay.

5.2.2.4 Scenario 4, Apparent Good Data

A site investigation may provide a large amount of quality data. In this ideal case it may appear that Bayesian updating is not necessary. There remain, however, advantages to using Bayesian updating. The examples presented in section 4.2.1 were applied to sites where the site investigation was considered to be good. For the detailed case study at Woolwich 37 test results were obtained from 8 boreholes while at Wimbledon there were 15 test results from 7 boreholes. Piles could be designed at these sites without the use of Bayesian updating yet advantages of using Bayesian updating have been demonstrated and discussed in terms of efficiency in design pile length (a 3 to 4% cost saving was observed in section 4.2.1.3) and in terms of reduction of the probability of failure. The probability of failure was observed to be reduced to less than 1 (an acceptable value) in section 4.2.1.5; this is a result of Bayesian updating reducing the coefficient of variation (as shown by the more peaked distribution of the posterior distribution shown in Figure 2.14) Essentially, by combining the sources of information through Bayesian updating, the sample becomes a closer representation of the population. To achieve this low probability of failure without updating a higher factor of safety (hence longer, more costly) would be required.

5.2.3 PILE-SOIL INTERACTION

In current design practice for bored piles in London Clay the value used to represent the adhesion is taken from design guidance (LDSA, 2000). This represents a conservative methodology. The guidance is based on rotary bored piles and its applicability to CFA and BD is questionable.

The adhesion factor is often used to represent more than just adhesion. Uncertainty in the site investigation is often a factor in the selection of α. Furthermore, the actual diameter of the pile (as opposed to the nominal diameter) is not considered in the back calculation of α by Skempton (1959), Patel (1992) or LDSA (2000). Rearrangement of Equation 1.3 reveals α to be inversely proportional to the diameter, the variation in diameter observed in section 4.5 is therefore mirrored by a variation in α. Variation in the diameter along the length of the pile, the effect of roughness of the interface or the presence of shear keys is not considered either. Skempton (1959) and Patel (1992) presented values for α averaged over the pile length and have not considered local variation. The α values for BD piles observed in section 2.5 are also averaged over pile length. There is a need for further investigation of local variation of α using instrumented piles to examine the distribution of load along the pile and with exhumation of the pile following testing to determine the actual shape and roughness, both of which were beyond the capabilities of this research for cost and practical reasons.

The pile construction process has an influence on the adhesion, for example construction of a BD pile leads to polishing of the clay adjacent to the pile. The polishing action produces a smoother interface and hence lower adhesion.

5.2.4 Probabilistic Design

Probabilistic design is not currently carried out routinely for piles yet it offers a great advantage in producing efficient designs and in comparing alternative solutions. The common procurement methods for pile design and construction are currently not suited to such analysis and design. Piles (including their design) are commonly procured as a separate entity to other construction activities, with cost being the major consideration. Probabilistic design needs to be considered in a holistic design approach to arrive at the most efficient design for the entire structure. An example to illustrate this is where a pile design with a lower probability of failure is preferable as the structure and sub-structure do not need to be designed to re-distribute loads from underperforming piles and/or less remedial foundation work is required. This piling solution may, in isolation, be more expensive but leads to greater savings when the probability of failure and the cost of the consequence are considered.

5.3 CONTRIBUTION TO EXISTING PRACTICE

This research has introduced an objective approach to the inclusion of previous knowledge and experience related to soil conditions. Adoption of the objective approach removes reliance on engineer's subjective assessment of ground conditions while retaining the possibility for the application of engineering judgement. Engineering judgement is applied through the selection of the dataset with which the site data is augmented. Existing data relating to the shear strength of London clay has been quantified and presented in a format that allows subsequent application to design and combination with site investigation data.

The methods for quantifying existing knowledge and its subsequent utilisation have been successfully put into practice within Rock and Alluvium. The benefit of this to the company is to allow optimisation of the design and the quantification of risk associated with a particular design. The techniques described are available to all practitioners, many of whom will have access to suitable quantitative data.

The research described in section 4.3 has resulted in the formulation and introduction of a universal BD pile classification and description system which has been previously absent from engineering vocabulary. The proposed system removes the reliance on trademark names and descriptions based on proprietary systems. It is anticipated that this classification system will be adopted; some use by practitioners has already been observed. Should further information regarding the nature of the soil displacement emerge, there may be a need to include the degree or form of displacement undergone by the soil in the classification system. The research described here, and the classification system, remain valid and a major benefit of this research is that the classification system can provide a framework to which future sub-categories can be added. The system is capable of evolving to suit the advancing knowledge but for the present provides a much needed point of reference.

Case studies of BD pile installation and pile load tests in stiff clay have been presented, thereby enabling comparison and benchmarking of BD pile types. This is an area where little or no information has been previously published into the public domain. It is intended that this forms a starting point and a base for comparison for future work in this field. The main benefit of this research is that a point of reference has been established against which other BD piles can be compared. The values for α observed and the observations made on BD pile construction can be, and have been, used for preliminary design or feasibility study, however estimates must be refined in light of site specific trials and testing. Such ongoing investigations should be added to the current knowledge so that a cycle of increasing and refining knowledge occurs with the eventual possibility of reaching a position of confidence in predicted values.

5.4 IMPACT ON THE SPONSOR

Existing knowledge relating to shear strength of London Clay has been quantified into a versatile and easily applied database. The database provides the Company with a comprehensive single point of reference for the shear strength of London Clay, this can be used for design and can be used with the Bayesian updating method to improve the interpretation of ground conditions.

Tools have been developed to aid design, in particular the revision of design procedures to include Bayesian updating techniques and objective methods of assessing the ground conditions. This has been implemented alongside the Company's revision of design procedures to comply with Eurocode 7 and is compatible with this as it provides descriptive statistics of the soil parameters from which characteristic values can be calculated. This process therefore ensures that current and future legislative requirements are met but also enables the company to achieve a competitive advantage through employing advanced techniques and its own collection of experiential data to arrive at more sophisticated models of the ground conditions and estimates of associated risk arising from ground variability.

From the research findings, a number of training sessions have been devised and delivered to the technical, design and engineering staff at Rock and Alluvium to enable them to make use of the database and the Bayesian updating techniques. Throughout the development of these techniques a primary requirement was to maintain ease of use for the engineer. The principles are applied through an easy to use method which requires little in terms of additional skills from the Engineer. The success of this has been demonstrated through the training of staff and adoption of the techniques, the impact of which has been seen by the Company where more efficient and rigorous designs have been made possible and this has resulted in winning work of high value. Bayesian updating adds an extra tool for the design engineers at Rock and Alluvium and is currently applied where the degree of risk needs to be quantified or where advanced design optimisation is required.

The BD piling technique has been developed and refined to enable construction in stiff clays. The research has led to an increased understanding of the forces, speeds and rates of pile installation which is advantageous when considering the constructability of piled foundations in particular soils and in predicting production rates and costs. The benefits of this research have been two-fold. Firstly, the observation of rig parameters has provided a benchmark that is used as a quality control measure – where large deviations occur from the values observed in this research, further investigation and testing is implemented to ascertain the cause and effects. The second benefit is in providing typical production rates and guidance on constructability in London Clay, both of which are essential when tendering for a contract and assessing feasibility and costs.

In addition to the benefits described, the research has provided numerous opportunities for publicity for Rock and Alluvium. These opportunities include the papers referred to in this thesis (Papers 1 to 4, Appendices A-D) and a number of publications in the trade press related to the research. These are listed in Table 5.1. In addition to these publications, publicity has been acquired through attendance, and poster presentations, at conferences attended by practitioners in this area of industry and potential client organisations. The research has also led to the author's involvement, representing Rock and Alluvium as a member of the Federation of Piling Specialists, in discussions regarding the redrafting of the LDSA (2000) guidance note on the design of bored piles in London Clay.

Table 5.1. Other Papers and articles related to the research.

Title	Publication	Date
Displacement Augered Piles	Innovation and Research Focus	November 2005
Design and Installation of Bored Displacement Piles	Proceedings, 6th International PhD Symposium in Civil Engineering, Zurich	August 2006
Bored Displacement Piles in Stiff Clay	Ground Engineering	October 2006

5.5 IMPLICATIONS FOR WIDER INDUSTRY

The Bayesian updating technique is simple to employ and could be adopted industry wide. It provides an objective approach to a problem which is traditionally solved subjectively and can be a source of dispute or conflict. An objective approach allows this source of conflict to be removed. The detailed working of the database is not freely available to wider industry. The structure is however easy to replicate and many geotechnical engineering consultancies and contractors will have their own sources of information which can be used in a similar manner.

Alternative designs which have been produced in the quantitative, objective manner proposed can be compared using a probabilistic analysis. The costs associated with a failure can be multiplied by the probability of failure and then be compared with the additional costs of implementing a more robust design with a lower probability.

The use of Bayesian updating and/or the probabilistic comparisons described in this research presents a conceptual challenge for the foundations industry. Probabilistic design recognises that there is always a probability of failure and deals with reducing this probability and mitigating the effects of failure. The piling and foundations industry is, however, used to dealing with perceived certainties and it is difficult to accept any possibility of failure. Greater understanding is required from Clients, and all stakeholders in the pile design and construction processes. By conducting comparisons and considering the cost of rectifying failure compared to the cost of reducing the probability of failure further, the benefits of this approach can be seen. With the advent of BS EN 1997-1:2004 (BSI, 2004), such approaches will be increasingly required and the techniques described in this research will be useful in both quantifying and reducing risk.

The value of this work to industry was noted by Kumar (2008) who wrote,

> "Baxter et al. [Paper 2, appendix B] presented a study in which they used past experience and a case history to enhance the design of pile foundations. It is well understood that the design of pile foundation requires a lot of judgment. Therefore, knowledge from other similar projects in similar ground conditions could result in substantial cost savings when pile foundations are used."

The value of a wider source of information which can be used to augment design using Bayesian updating has been demonstrated. Many SI contractors have vast arrays of data and there is potential for value to be added to ground investigations as well as desk studies through providing this information in SI reports, provided practical obstacles such as the legal ownership of data can be overcome.

Guidance relating to the selection of α for bored piles has been published (LDSA, 2000). There is no comparable information for BD pile design. This research presents useful information in this field. The LDSA (2000) guidance is flawed and could provide for more rigorous design by consideration of actual pile diameter in back analysis of pile performance and should also separate the adhesion and other factors (such as an allowance for the quality of SI and the influence of the pile load test method) that are currently lumped into the suggested values of α.

BD piles are a specialised type of pile of which many variations exist. This research presents a classification system for the description of these piles which adds to the general pile type classifications used in specifications and standards (e.g. ICE, 2007) and by trade bodies such as the Federation of Piling Specialists.

There has been very little published relating to BD piles in the UK, particularly in fine grained soils. This research has provided a benchmark for other BD pile systems and BD piles in different soils to be compared against. A framework for description of the BD pile systems has been produced such that these comparisons can be made in a consistent and easily understood manner. Scheme designs and alternative solutions using different piling techniques can be compared using the observations on performance reported in this research. This is of use to Promoters, Clients, Engineers, Planners and Contractors.

5.6 CRITICAL EVALUATION OF THE RESEARCH

Two types of pile have been described in this research, CFA and BD, these are the main products of the sponsoring company and hence the interest in these types. While the methods for building a database of existing knowledge and using this to augment models of site conditions are applicable to other pile types and even other geotechnical situations, this research, has been conducted from the perspective of application CFA and BD piles.

BD piles are relatively new and there has been little published regarding their construction and performance. This is largely due to the specialised nature of the pile type and its somewhat limited application. This specialised nature and limited application has led to a shortage of available sites where pile load tests and installation monitoring have been carried out. The choice to limit the research to London Clay has further restricted the sites of interest, this decision was driven by the availability of suitable sites and by the commercial interests of the sponsoring company.

Throughout the research, the sites where observations have been made and tests conducted have been live construction sites. Time has therefore been an important constraint for the number of tests conducted, number and length of load cycles, hold periods and age of pile at time of testing. Test sites that were representative of not only the chosen ground conditions but also typical of the conditions encountered during general pile construction were sought. Ideally more piles at each site and a larger number of sites are required. Use of production piles is not always feasible due to contractual restrictions, time constraints and programmed follow on works. The cost of pile production and availability of sites has however limited the number available for this research. In order to add to the research test data, test results have been gleaned from contractual test piles, where possible. This however is a limited resource, for many contracts it is more economical to design piles to a higher overall factor of safety and to omit or reduce the pile load test requirement; this is an accepted and common practice. Unfortunately this can perpetuate over-conservative designs and does not engender an environment of continual learning and improvement. A change in practice for Clients and Contractors from a less short term, financial perspective to a pro-active knowledge and research based approach would lead to a better understanding and long term savings and a more sustainable solution that uses less material (i.e. concrete).

In order that the Bayesian updating techniques described can be applied to a design or analysis situation, it is necessary for the Engineer to hold quantitative previous knowledge of the soil. A judgement must be applied that the prior data and the soil at the site of interest are from the same population. If this is not the case then the techniques are not applicable and are no longer available to the Engineer. The data presented contains many sources of variation which have not been separated. This is largely due to the quality of site investigation data available but represents a limitation of the research.

Throughout this research, the soil type of interest was London Clay. Choice of this soil type as a subject was partly governed by Rock and Alluvium’s market. Not only is there an archive of soil information, piling experience and test results there is interest in greater knowledge for future application. This choice limits the applicability of some of this research to a small geographical area and further work for other strata and geological deposits is required.

A key factor in pile design calculations is the adhesion factor selected. This was not made the central focus of this research because of statutory limitations on the values which can be used in design in the London area (LDSA, 2000). A better understanding of adhesion between pile and soil would be desirable from an academic viewpoint, particularly for pile load test analysis but because of limits set on the values that can be used for design, there is little direct practical or commercial value in such research to a piling contractor.

The as-constructed pile dimensions were estimated from concrete volumes. In order to ascertain an accurate measure of the as-constructed dimensions, a pile would need to be exhumed and measured. The concrete volumes, however, provide a reasonable estimate of an otherwise unquantifiable variable in the pile design and improves the practice compared to the use of nominal diameters.

The BD and CFA pile load tests reported were not instrumented. This has led to a reliance upon assumptions relating to the proportion of load carried by the pile base and the distribution of load along the shaft. The instrumentation of BD and CFA piles is difficult and unreliable due to the construction sequence; the instrumentation would have to be plunged into wet concrete, an aggressive process that is difficult to control and subjects the instrumentation to an unfavourable environment.

5.7 RECOMMENDATIONS FOR FURTHER RESEARCH

There are a number of areas where further research would be of value, both commercial and scientific, that have been identified through this research.

The Bayesian updating has many commercial applications; it is therefore in the interests of those designing geotechnical structures to gather and process information to quantify knowledge and experience in a variety of ground conditions. Further research adopting a similar methodology is needed to derive improved models of ground conditions for other soils, in the UK or elsewhere. This research has presented a dataset for shear strength of London Clay, many other engineering properties can be quantified in a similar manner.

Furthermore similar work should be conducted for other soil types (other clays such as Oxford Clay, or sands and gravels) and for different regions (e.g. London Clay in the Hampshire Basin). Further data are available, within Rock and Alluvium and other geotechnical practitioners, and are currently under utilised, quantification and application to design and analysis work would have a beneficial effect.

BD piles are a specialised technique with very particular applications. Design and construction is currently based on experience, trials and test results. A greater understanding of the process and effects of the soil displacement and subsequent soil behaviour and soil-structure interaction would be of scientific interest and could lead to better practice. This would also further inform the classification of BD piles, which would benefit industry and research through improving communication and discussion of pile types.

Further load testing and back analysis of BD piles, following similar methodology to that described in this research, is required to develop generic empirical factors for design coefficients such as adhesion, α. Furthermore, investigation is required to develop values for α based on the actual diameter of both CFA and BD piles so that design calculations can be more representative of the actual situation. This is critical for the analysis of pile behaviour and is therefore of scientific interest. Application for design may be limited by codes and standards which tend to adopt a cautious approach; however, through further research and publication of results the extent of the conservatism can be better defined and where it is found to be excessive it could be reduced. Different types of BD pile should be investigated, together with the effects of altering the installation parameters, for example does increasing the rate of rotation of the auger have a beneficial, detrimental or negligible effect of capacity and/or load settlement characteristics. This could be conducted by construction of a series of test piles at a site with uniform ground conditions. Piles could be constructed with carefully controlled parameters, varying the parameter under investigation between piles. Subsequent testing should be consistent between the piles under investigation.

Further understanding of the displacements undergone by the soil and the associated stresses, strains and changes in pore water pressures is required to understand the nature and behaviour of this pile type. This could be achieved by installation of instrumentation (such as inclinometers and pressure cells) in the vicinity of the pile location and monitoring changes during pile construction. Laboratory modelling and observation could also be used.

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APPENDIX A PAPER 1

Full Reference

BAXTER, D.J., DIXON, N., FLEMING, P.R. AND CROMWELL, K.A., 2008, Refining characteristic values of shear strength using experience, *Proceedings of the Institution of Civil Engineers: Geotechnical Engineering*, **161**, pp247-257

Abstract

Determination of characteristic values for soil properties forms a critical step in the foundation design process. The refinement of such values to account for not only site specific data but also existing knowledge and previous experience can result in more efficient design and increased confidence. This paper presents a logical, pragmatic approach for the selection of characteristic values of shear strength for the design of piled foundations within the context of Eurocode 7. The process of refining conceptual models of geotechnical properties in a quantitative, objective manner to include previous knowledge and wider experience is described and demonstrated through case study examples. The result of applying the updating methods is to achieve a revised mean which is a weighted average of the site data and the prior knowledge; the weighting is a function of the variability of each set of data. Such refinement relies upon a quantification of previous knowledge; mean values, trends, variations and distributions of data are required and in this paper a dataset for undrained shear strength of London Clay is presented and applied to case study examples. Application of the techniques described leads to a better estimate of the ground properties and a reduction in the risk attached to a design solution.

Keywords

Piles and piling, geotechnical engineering, design methods and aids

Notation

a	estimated minimum value
b	estimated most likely value
c	estimated maximum value
c_u	undrained shear strength
d	depth below ground level
n	number of data points
R^2	correlation coefficient
V_x	coefficient of variation of the derived values
$\bar{x}$	mean of the sample data
X_m	statistical mean of the derived values
X_k	characteristic value
α	factor relating pile adhesion to soil cohesion
μ'	mean of the prior data
μ''	mean of the updated (posterior) data
σ	standard deviation of the sample data
σ'	standard deviation of the prior data
σ''	standard deviation of the updated (posterior) data

1 INTRODUCTION

In geotechnical design, the skill of the engineer lies in determining the ground conditions at a site and how these are best modelled for use in suitable design calculations. It is not surprising therefore, that a large proportion of the design process is related to the site investigation, its design, execution and interpretation. In this paper, this process is considered within the framework of Eurocode 7. The design procedure for Eurocode 7 has been described by Simpson and Driscoll[2] and is summarised in Figure 1.

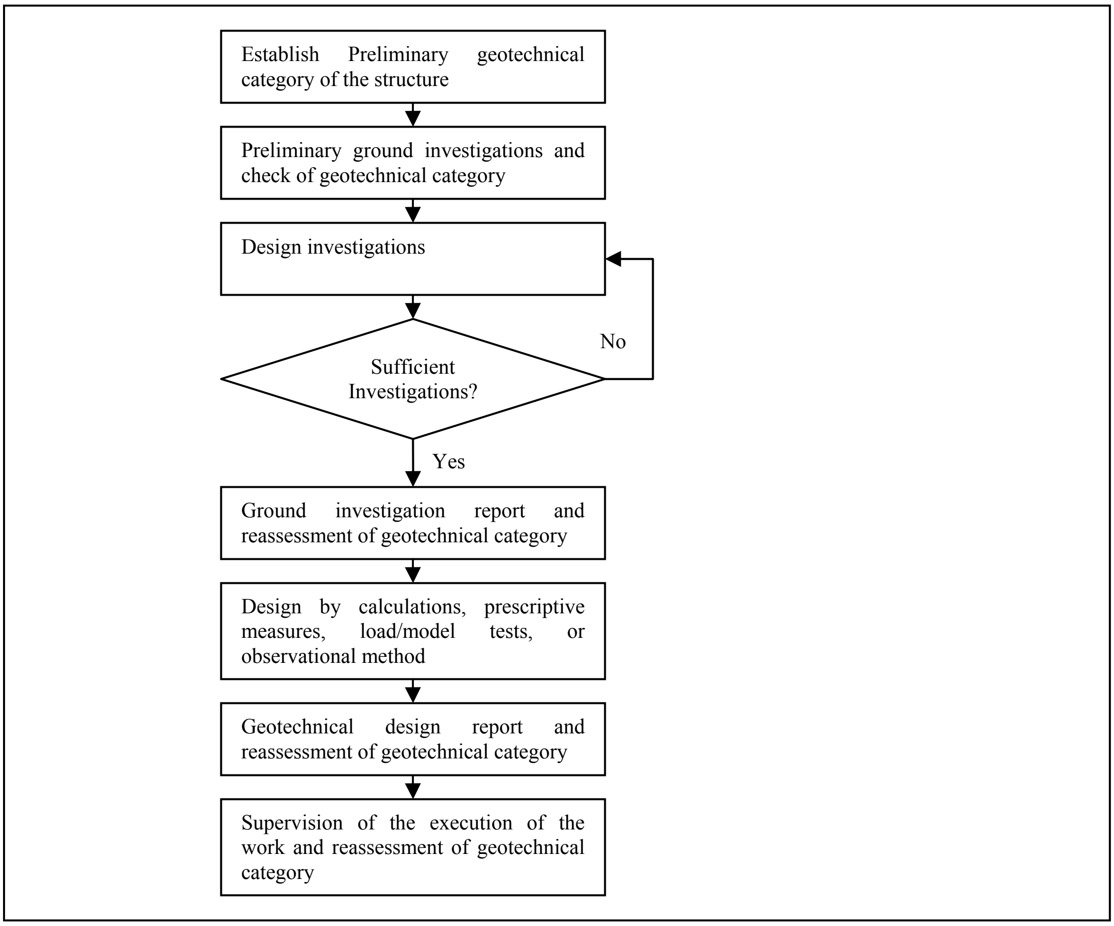

Figure 1. The design process of BS EN 1997-1 (After Simpson and Driscoll[2])

Experience and previously existing knowledge of the ground conditions can be incorporated into the geotechnical model, a process which is often carried out in a subjective manner. In this paper, a quantitative approach to the inclusion of previous knowledge is discussed and applied to examples of bored pile design. To facilitate this, a set of shear strength data from numerous sites has been collated and described. For design purposes, a characterisation of the ground conditions is needed which takes account of variation and the likely deviation which might be expected in the site investigation results. In this paper, a simple statistical approximation is applied to produce a suitable model based on characteristic values of the shear strength at the site. The methods described are then applied to case study examples to demonstrate the influence on design and costs.

2 BACKGROUND

2.1 MODELLING GROUND PROPERTIES

For geotechnical design, it is necessary to construct a model of the ground properties which exist at a site. The usual starting point for this is test results from site investigation. These cannot be used for design calculations in their raw form as account must be taken of variability across the site, the nature of the tests and the manner in which the structure will interact with the soil. The process of determining values for use in design from site investigation measurements follows an essentially linear progression throughout which the designer will make and apply numerous decisions and judgements.

This process for determination of ground properties (following Eurocode 7, Part 1)[1] has been described by Orr[3] and by Frank *et al*[4]. Figure 2 shows the individual steps necessary to establish ground properties which can be used in design. The steps are described below and follow the development of soil property values through four stages – measured values, derived values, characteristic values and design values. Frank *et al.*[4] introduced intermediate steps, most notably the 'geotechnical parameter value' which allows for consideration of existing knowledge and experience. Ground properties are established primarily from site investigation through tests carried out *in situ* and on samples (disturbed or undisturbed) taken for laboratory testing. These are referred to as the 'measured values' and represent the test results (following the application of any test related corrections, which are independent of further analysis). Where test results do not return a value which can be used directly in design, these need to be converted to 'derived values' by applying theory, empiricism or correlations. An example of this is the correlation of SPT blow counts to undrained shear strength through the relationship proposed by Stroud and Butler[5]. The 'geotechnical parameter value' is an intermediate stage which allows for an assessment of the influence of the test, with associated corrections, such as a conversion from axisymmetric to plane strain conditions to better represent the design situation. This is also the point at which refinements can be made based upon published data and general experience or prior knowledge.

The derived values (and geotechnical parameter values) are only representative of the sample and not of the overall distribution of values on the site. The values which describe the properties of the ground at the site, its associated variation, the nature and behaviour of the soil mass and its interaction with the structure are the 'characteristic values'. This characterisation of the site and the geotechnical problem is a vital step in the design and is where the engineer's skill and judgement is applied. The characteristic values are subsequently developed into to 'design values' by the application of safety factors.

2.2 INCLUSION OF PREVIOUS EXPERIENCE AND PRIOR KNOWLEDGE

If information is not sufficient for design, the result is a lack of confidence in the derived values that leads to greater conservatism being called for when obtaining characteristic values. Larger factors of safety would need to be applied. Eurocode 7[1] stipulates that where statistical methods are employed the characteristic value should be selected such that the calculated probability of a worse value governing the occurrence of the limit state considered should not be greater than 5%; the factors of safety have been developed accordingly and this acts as a motivating factor for increasing the quality of site investigation. Selection of other parameters may also be affected by a low confidence level (even though this can disguise the overall allowance for uncertainty). An example of this can be found in pile design where the shaft capacity of a pile is related to the soil strength and pile geometry through the adhesion factor, α. Guidance for pile design in London Clay[6] indicates that a lower (more cautious) value for α should be used where a site investigation does not meet minimum standards. This restriction is intended to create an incentive to carry out quality site investigation. It does, however, make it difficult to ascertain the degree of confidence assigned to a design. Such an approach is not compatible with Eurocode 7, which deals with the uncertainty solely in the characterisation process, with subsequent application of suitable factors of safety dependent on the confidence in the calculations.

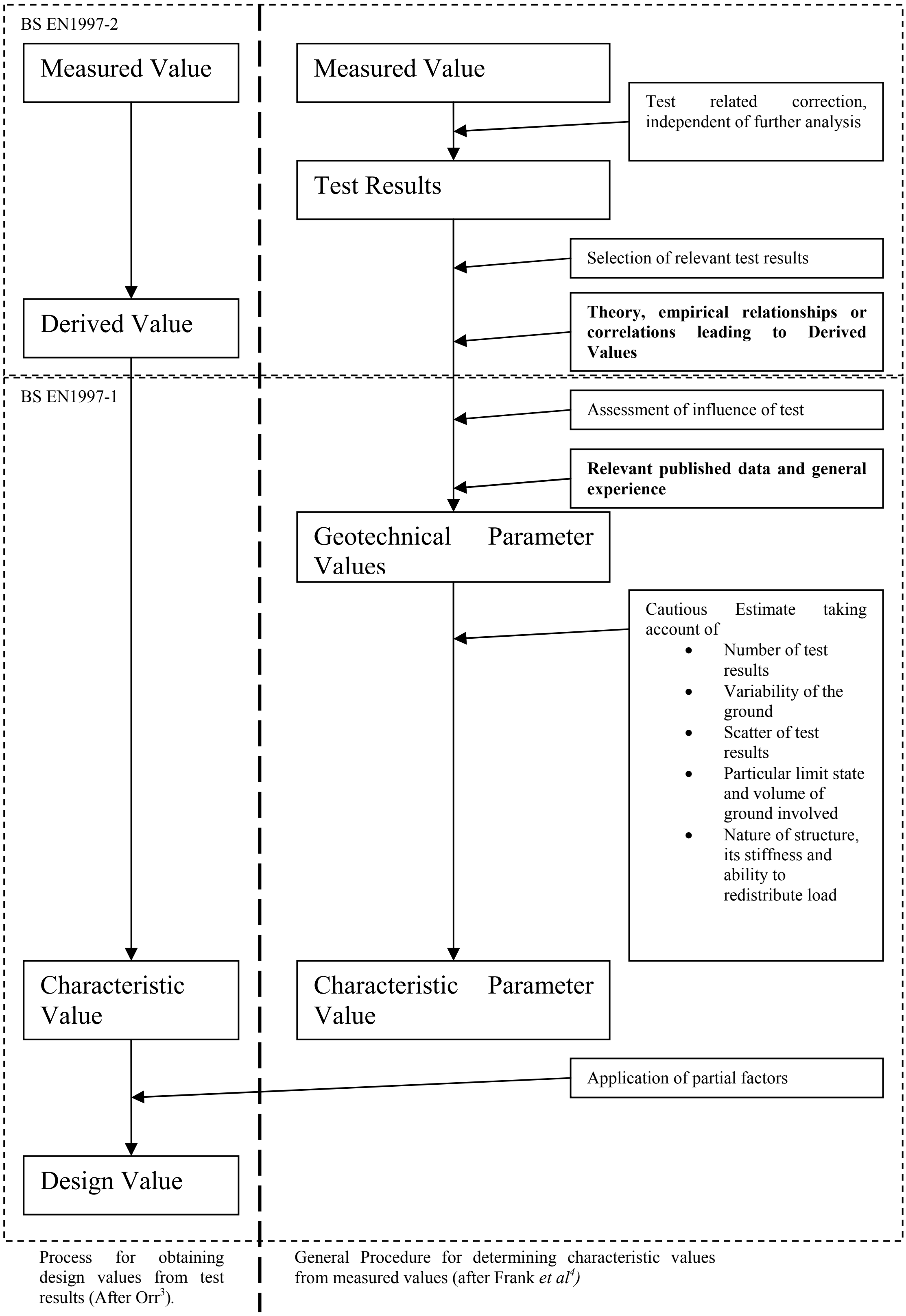

Figure 2. Steps in determining characteristic and design values (After Orr[3] and Frank et *al.*[4])

Where previous experience, published data, or knowledge from nearby similar sites exists, a method of combining this information with the site specific data is advantageous in developing characteristic values. Traditionally, this may have been achieved by application of subjective judgement based on experience. An objective approach is suggested by authors such as Lumb[7] and Tang[8], who have presented methods for the application of statistics, in particular Bayes' theorem, to geotechnical applications. A better estimate of the likely value of the geotechnical parameter values (posterior distribution) can be found by combining information relating to the previous knowledge (prior distribution) and site specific information. Tang[8] observed that the posterior distribution is a product of its prior distribution, which represents the data gathered before the additional test data, and the likelihood function, which represents the information based on the additional test data. This method requires, as a minimum, that summary statistics such as the mean and standard deviation of the previous knowledge are known or can be determined in some way.

If a population is normally distributed, it can be shown, by application of Bayes' theorem that the posterior distribution is normal with the mean value and standard deviation calculated using Equation 2 and Equation 3

Posterior mean, $$\mu'' = \frac{\mu' \dfrac{\sigma^2}{n} + \bar{x}(\sigma')^2}{\dfrac{\sigma^2}{n} + (\sigma')^2}$$ **Equation 2**

Posterior standard deviation, $$\sigma'' = \sqrt{\frac{\dfrac{\sigma^2}{n}(\sigma')^2}{\dfrac{\sigma^2}{n} + (\sigma')^2}}$$ **Equation 3**

Where the prior data has mean μ' and standard deviation σ' and the derived values for the site are represented by the mean $\bar{x}$ and standard deviation σ; n represents the number of data points.

The resulting posterior distribution has a mean which is a weighted average of the prior and sample data means, the weighting being proportional to the variances. The posterior standard deviation will be lower than that for both the prior and sample data distributions as they combine to produce a sharper, more peaked distribution, i.e. data points are more closely grouped about the mean.

2.3 DETERMINATION OF CHARACTERISTIC VALUE

The characteristic value is defined in Eurocode 7, Part 1[1] as a cautious estimate of the value affecting the occurrence of the limit state. The characteristic value must take account of the inherent variability of the soil, measurement errors and the extent of the zone governing behaviour[9], further factors leading to differences between derived values and those governing behaviour are listed by Frank *et al.*[10]. These include, but are not limited to, the presence of soil structure (e.g. fissures), time effects, water softening and the influence of construction activities on the soil.

The characteristic value, used in Eurocode 7[2], is comparable to the 'conservatively chosen' mean which is traditionally used in the British Standard approach for foundation design[11]. The degree of conservatism has conventionally been based on local experience or subjective information. For Eurocode 7[1], an objective measure of the degree of conservatism is required to obtain a specific level of safety. One way in which this can be achieved is through the use of statistical methods.

An array of statistical techniques is available and has been presented by Van Alboom and Menge[13] who concluded that statistics are a useful tool but that care should be taken regarding the adoption of assumed distributions of the population; care should also be taken that sample sizes are sufficiently large. The introduction of statistical techniques can add unnecessary complication to the design process and demands additional skills from the engineer. This can be overcome by using an easily employed approximation for the statistical definition of the characteristic value, such as that proposed by Schneider[14] and described by Equation 4.

$$X_k = X_m(1 - \frac{V_x}{2}) \qquad \textbf{Equation 4}$$

Where X_k is the characteristic value, X_m is the statistical mean of the derived values and V_x is the coefficient of variation of the derived values. X_m and V_x can be estimated from the sample distribution when there are sufficient data to provide an approximation of the population. D'Agostino and Stephens[15] suggest that at least 150 points would be required for a dataset to be representative of the population. Where there is insufficient data, X_m can be determined using the approximation in Equation 5.

$$X_m \approx \frac{(a+4b+c)}{6} \qquad \textbf{Equation 5}$$

Where a and c are the estimated minimum and maximum values and b is the estimated most likely value. Estimations can be based on experience, judgement and published data. Schneider[14] found the coefficient of variation, V_x, of the undrained shear strength of a soil to be in the range 0.3 to 0.5 and recommended a value of 0.4. Many other authors have found similar results; Hooper and Butler[16]found a coefficient of variation of 0.25 to 0.33 for triaxial tests carried out on 100 mm diameter samples of London Clay from a single site. Phoon and Kulhawy[17] report a range for V_x of undrained shear strength of fine grained soils of 0.06 to 0.56, with a mean value of 0.33 based on 38 separate data sets. Lumb[7] observed a range for the coefficient of variation for shear strength of Hong Kong Marine Clay of 0.2 to 0.5.

Schneider[14] demonstrated that approximations of characteristic value using Equation 4 to be consistent with the values estimated by engineers and it has been in use in Switzerland for many years. A sufficiently accurate solution can be obtained without extensive calculations and this simplicity coupled with the flexibility to include an element of engineering judgement makes it a useful tool for practising engineers.

2.4 LONDON CLAY

London Clay has been chosen as the subject for this study. There are two reasons behind its selection. London Clay covers a large geographical area which has a long and well documented history of construction development. There is therefore an existing associated body of knowledge upon which the current research can build and extend. This history of construction can be expected to continue as the area underlain by London Clay is still considered prime for development; the research findings should then be applicable to future construction works. In this paper, the London Clay is treated as a single statistical population; as justified by the following consideration of the deposit.

London Clay is an overconsolidated, stiff, fissured clay. It is considered to be reasonably uniform and its properties have been extensively investigated. It is a blue or grey clay which weathers to brown and contains fossils resembling modern warm-water forms and can contain bands of concretions as well as pyrite or selenite crystals[18].

The behaviour of bored piles in London Clay has been described by Skempton[19]. Skempton analysed pile load tests at ten sites and summarised extensive data relating to the strength of London Clay. This strength data is based on tests of undisturbed samples of 38 mm diameter. Modern site investigation techniques utilise 100 mm diameter undisturbed samples because of the increased likelihood of intercepting a natural fissure and hence obtaining a representative, fissured, shear strength. Patel[20] reanalysed the results presented by Skempton[19], and Whitaker and Cooke[21] and added further results of investigations at 23 sites across the London Basin.

London Clay is a relatively consistent and uniform geological deposit; there is however, variation between sites which leads to the need for individual consideration and site specific investigation[20]. These differences can be partially explained by considering the history of the strata: the sequence of sedimentation, erosion and weathering.

King[22] describes how the marine sedimentation of the London Clay was affected by changes in sea level during the deposition, a fall in sea level being associated with coarser grained material being deposited. The London Clay can be divided into five successive lithological units which reflect the cyclical changes in the depositional environment. Engineering properties can be discontinuous across the boundaries between the lithological units; hence there is merit in knowing the relative positions of the divisions and interpreting property profiles accordingly. In the case of undrained shear strength however there is no discernable discontinuity[23], there is therefore no justification for the additional site investigation necessary to identify the lithological units for a statistical assessment.

Post deposition, the London Clay has been subjected to a complex loading regime. Continuing deposition led to the London Clay being deeply buried. Subsequent erosion of these deposits and of the London Clay during the late Tertiary and Pleistocene times has led to the surviving clay being heavily overconsolidated. The extent of this erosion has been estimated to be a thickness of between 150 and 300 m[24-26]. In places, there has been further re-covering of the London Clay with river terrace deposits (sands and gravels) as well as alluvial deposits. Skempton[19] observed some softening of the clay where there had been erosion and subsequent re-covering with alluvial material and made a correction to sample depth for samples from such areas based on the relative density of overlying materials but only to depths of about 15 m below ground level. Conversely, Patel[20] used only straight line regressions for mean shear strength profiles. Inspection of the data collated for the current paper did not reveal clear evidence for special treatment of the clays which had been covered by alluvial deposits; they are therefore included in their original form and straight line regressions are performed to obtain mean shear strength versus depth profiles.

De Freitas[27] suggests that faulting may also influence the continuity of properties across the basin. The location of faults (and the suggested network of minor faults which would result from the brittle behaviour of the clay) has not been mapped. It is therefore currently impossible to determine the effect that this has on the spatial variability of engineering properties. The lithology may not be continuous but as this has been observed not to have an effect on strength, and as regional trending of the data has not been observed by the authors, the data has been treated as a continuous set.

The weathering of London Clay is apparent to differing depths across the London Basin. Where the clay has been covered with terrace deposits these have afforded some protection and the weathered zone tends to extend to only about 1 m below the top of the clay; elsewhere the weathering may be apparent to depths of 10 m. Chandler and Apted[28] have investigated the effect of weathering on the strength of London Clay. Their results show a change in the strength in effective stress terms; this is related to an apparent reduction in the overconsolidation ratio. In this study, the undrained strength is of interest and inspection of the data revealed that allowance for the weathering of the London Clay did not lead to a significant difference in the mean shear strength profile.

3 QUANTIFYING EXPERIENCE AND PRIOR KNOWLEDGE

Consideration of previous knowledge and experience can assist in obtaining characteristic values which better represent the site conditions or provide greater confidence. In order to apply the objective methods introduced in this paper, a quantification of the existing knowledge or experience is required. In the following section, information relating to the shear strength of London Clay is collated and processed by way of an example of how prior knowledge might be quantified for use in such calculations and characterisation. The information is also useful for probabilistic analysis and design of piles as shown in the case studies.

3.1 CONSTRUCTION OF DATABASE FOR SHEAR STRENGTH OF LONDON CLAY

In order to quantify existing knowledge relating to the mean value and variation of shear strength of London Clay, a database of site investigation data has been constructed. The data presented in this paper is drawn from 68 sites in the London Basin, predominantly in the Greater London area; the locations of the sites are shown in Figure 3. There were results from 947 quick undrained triaxial compression tests conducted on undisturbed samples from 234 boreholes. The data is drawn from investigations by 31 different SI contractors carried out for piling contracts between 2003 and 2006. There were between 1 and 11 boreholes at each site with shear strengths reported from between 1 and 24 samples per borehole. Sites have been selected where 100 mm diameter undisturbed samples were taken from the London Clay for quick undrained triaxial testing.

Shear strengths are recorded against depth below site ground level. The depth to the top of the London Clay is also recorded for each borehole. An example of the strength/depth data for one typical site is shown in Figure 4 with a linear regression trend line; the 37 triaxial test results are from samples taken from 8 boreholes.

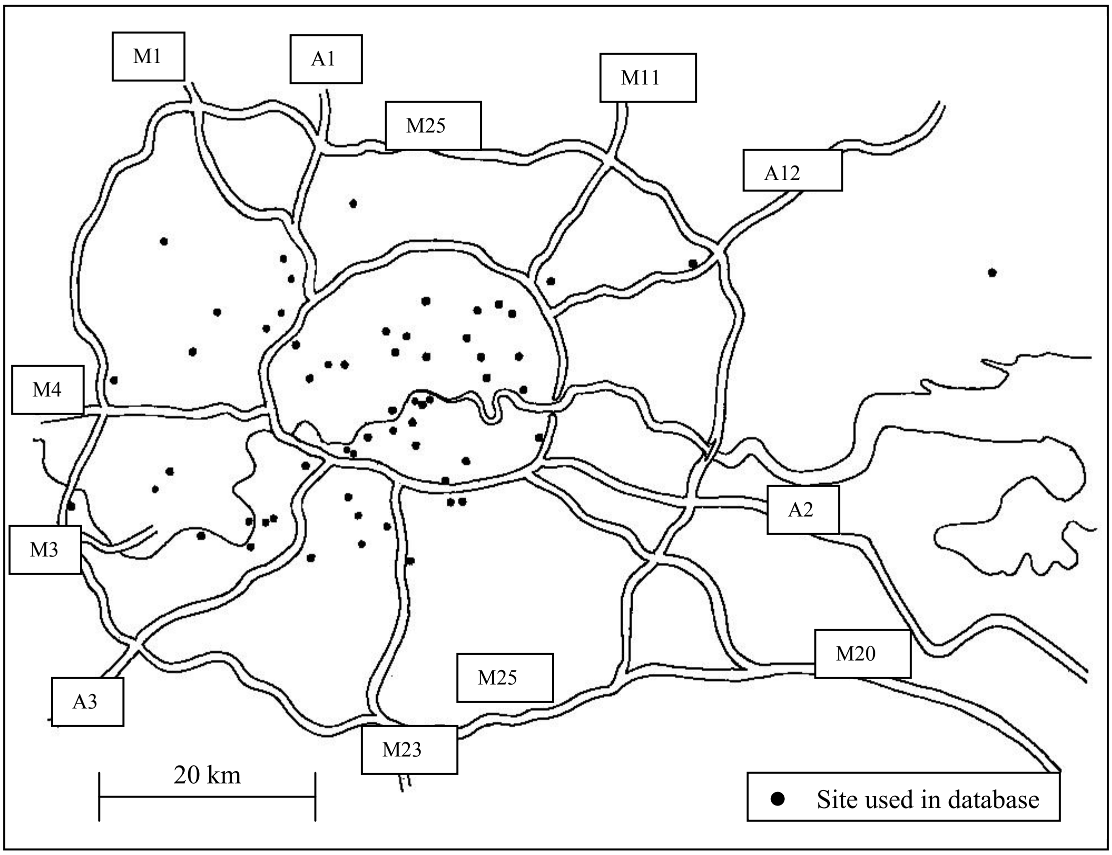

Figure 3. Location of sites in the Greater London area used in database

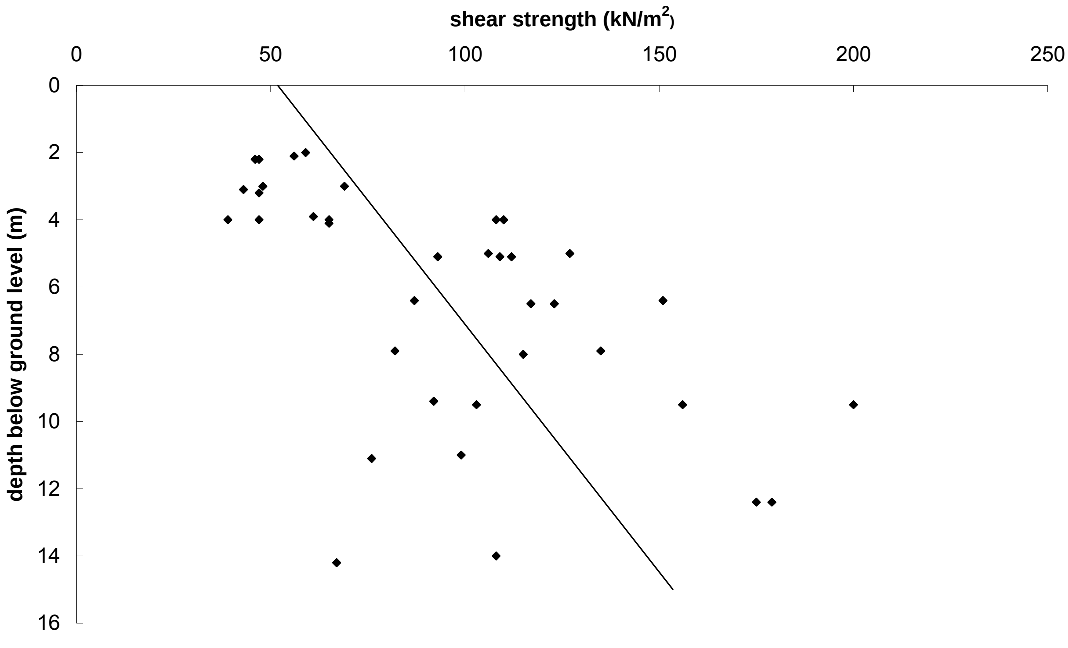

Figure 4. Example shear strength data, with linear regression trend line, from site investigation at a site near Woolwich.

If the collated data from all 68 sites is treated as a single continuous dataset, a regression line can be used to describe the mean value of shear strength of the London Clay with depth. A linear trend for strength against depth below ground surface for the entire dataset is shown in Figure 5; a straight line regression has been performed on the data using the method of least squares. The equation of the line is $c_u = 6.1d + 60.0$ kN/m^2, where c_u is the undrained shear strength and d is the depth below ground level; the fit of the line to the data has a correlation coefficient (R^2) of 0.43. A straight line has been chosen as this provides the simplest representation of the data. This approach is consistent with previous studies (Patel[20], Hooper and Butler[16], Whitaker and Cooke[21]). Other types of line (power, logarithmic, exponential and polynomial) do not provide significantly greater correlation.

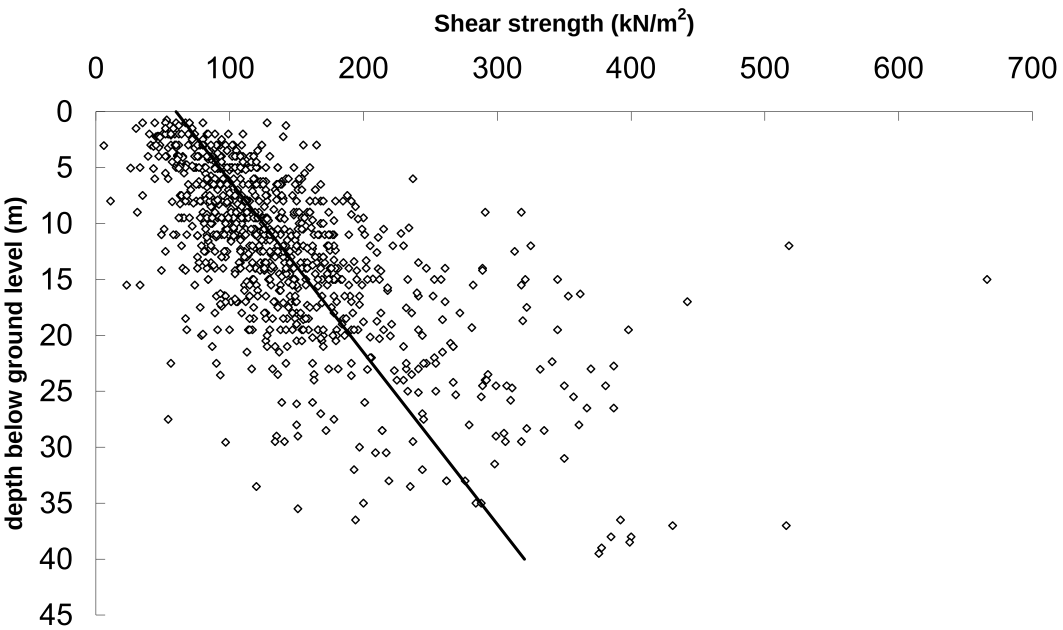

Figure 5. Shear strength against depth for all sites in the dataset, with trend line.

The collated data can be compared to existing published data for the undrained shear strength of London Clay. Patel[20] used the top of the London Clay as a reference point, not ground level, and to facilitate direct comparison the data from this study has been converted into this format. Figure 6 shows mean regression lines of the strength with depth below top of clay for four example sites along with the mean data for the whole dataset presented in this paper, and overlain on this is the envelope of the mean lines presented by Patel[20] for 23 different sites across the London basin. The mean regression line of the data collected in this study is observed to be close to the middle of the range reported by Patel[20], this is consistent with the assumption of normal distribution of shear strength. The individual mean regression lines for single sites shown in this example also fall within the range. The data presented and used in this study is consistent with that previously reported.

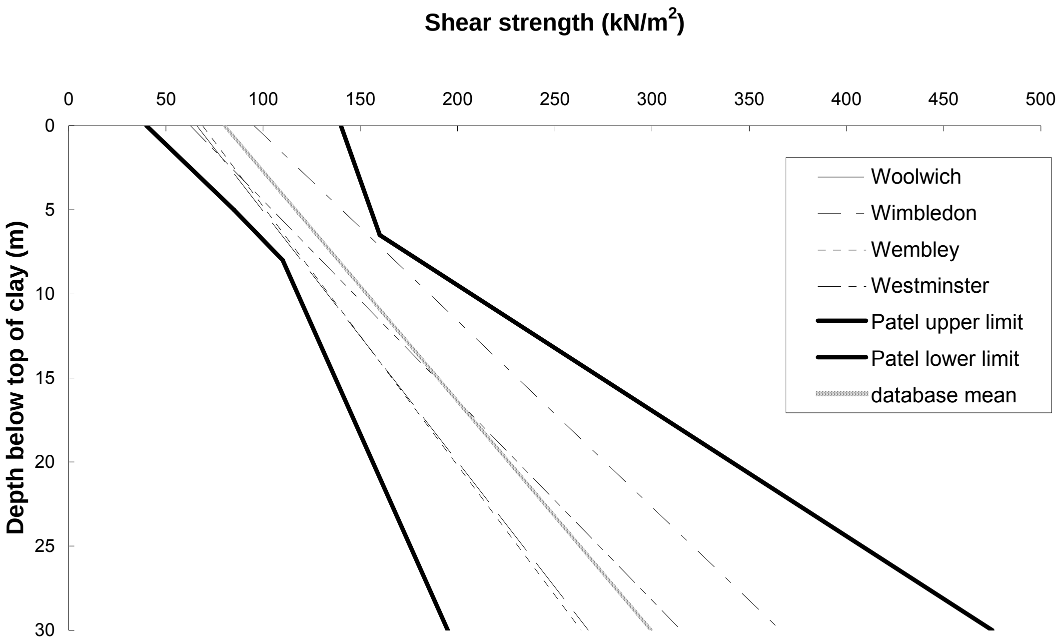

Figure 6. Comparison of site data and dataset mean with data published by Patel[20] (depth measured from top of clay)

An additional benefit of the data presented in this paper is that information relating to the distribution of the data is also available, thus allowing estimates of the coefficient of variation to be made. The size of the database means that it can be assessed and manipulated using statistical techniques. Information relating to the distribution and variation also permits probabilistic analysis to be performed as an alternative or in addition to traditional deterministic analysis.

3.2 DATA PROCESSING

The data in its raw format, or simply summarised by a regression line, cannot be easily employed as prior knowledge to update site specific information using the techniques described in section 2.2. A method of processing the data is required such that a mean and a measure of the distribution can be readily obtained for use in Bayesian updating. The method proposed is to group the strength data by depth to top of sample and to calculate summary statistics for each group; bands of 1 m thickness have been used for the grouping; depths have been measured from ground level. The summary statistics and the number of sites and data points used to generate them are shown in Table 1. The mean values for each band are plotted in Figure 7, overlaid by the mean regression line found previously for all test results (first shown in Figure 5) and a reasonable fit can be observed, particularly for depths between 0 and 20 metres where a greater number of data points are available. The deviations which occur below 20 m are likely to be a result of the low number of test results available at these depths as many boreholes used to build the database did not extend below 20 m, as indicated in Table 1.

The coefficients of variation shown in Table 1 and Figure 8 are in general agreement with those reported in previous literature[14, 16, 17].

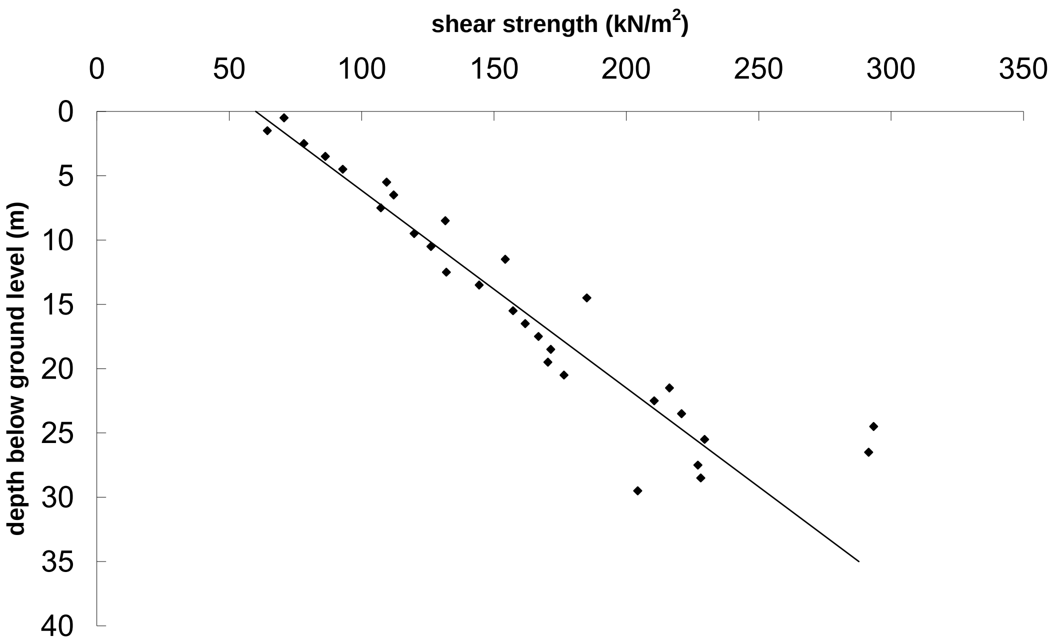

Figure 7. Mean values for shear strength grouped into 1 m thick depth bands, shown with the original regression line for all data points.

Table 1. Statistical analysis of shear strength data grouped by depth for London Clay

Nominal Depth (m)	Depth range (m)	Number of sites	Number of test results	Mean shear strength, X_m (kN/m^2)	Coefficient of variation, V_x
1	0.5-1.5	9	12	70.67	0.46
2	1.5-2.5	14	28	64.43	0.36
3	2.5-3.5	23	38	78.24	0.38
4	3.5-4.5	20	42	86.31	0.26
5	4.5-5.5	34	57	92.89	0.29
6	5.5-6.5	20	31	109.42	0.36
7	6.5-7.5	27	42	112.12	0.25
8	7.5-8.5	43	67	107.22	0.34
9	8.5-9.5	25	38	131.58	0.41
10	9.5-10.5	34	58	119.84	0.31
11	10.5-11.5	42	71	126.19	0.32
12	11.5-12.5	23	39	154.26	0.50
13	12.5-13.5.	31	49	132.04	0.32
14	13.5-14.5	39	64	144.41	0.36
15	14.5-15.5	25	40	185.06	0.51
16	15.5-16.5	19	29	157.17	0.48
17	16.5-17.5	21	37	161.81	0.45
18	17.5-18.5	13	22	166.82	0.35
19	18.5-19.5	18	24	171.46	0.33
20	19.5-20.5	20	37	170.33	0.39
21	20.5-21.5	10	14	176.46	0.33
22	21.5-22.5	2	7	216.29	0.36
23	22.5-23.5	10	20	210.53	0.41
24	23.5-24.5	9	13	220.85	0.30
25	24.5-25.5	5	10	293.40	0.16
26	25.5-26.5	2	7	229.57	0.38
27	26.5-27.5	1	4	291.50	0.36
28	27.5-28.5	2	7	227.00	0.47
29	28.5-29.5	5	8	228.13	0.33
30	29.5-30.5	3	7	204.29	0.42

3.3 DATA DISTRIBUTION

The Bayesian updating relies upon the data having a normal distribution. While most geotechnical parameters can be shown to have normal distributions[12], Van Alboom and Menge[13] warn that for some parameters lognormal distribution may be more appropriate. Hooper and Butler[16], however, demonstrated that the variation of shear strength (from laboratory test results) is well represented by a normal distribution. A subjective assessment of the data used in this paper can be carried out by inspecting the distribution of the test results. The strengths are first normalised for depth using the equation of the mean regression line. This normalised distribution is shown in Figure 9 with the normal distribution predicted from the mean and standard deviation. The distribution is observed to be close to normal and there is no indication of significant deviation which would suggest that an alternative distribution be more apt. Detailed examination of the form of the distribution and the effect of an assumption of normal, or other, distribution is beyond the scope of this paper.

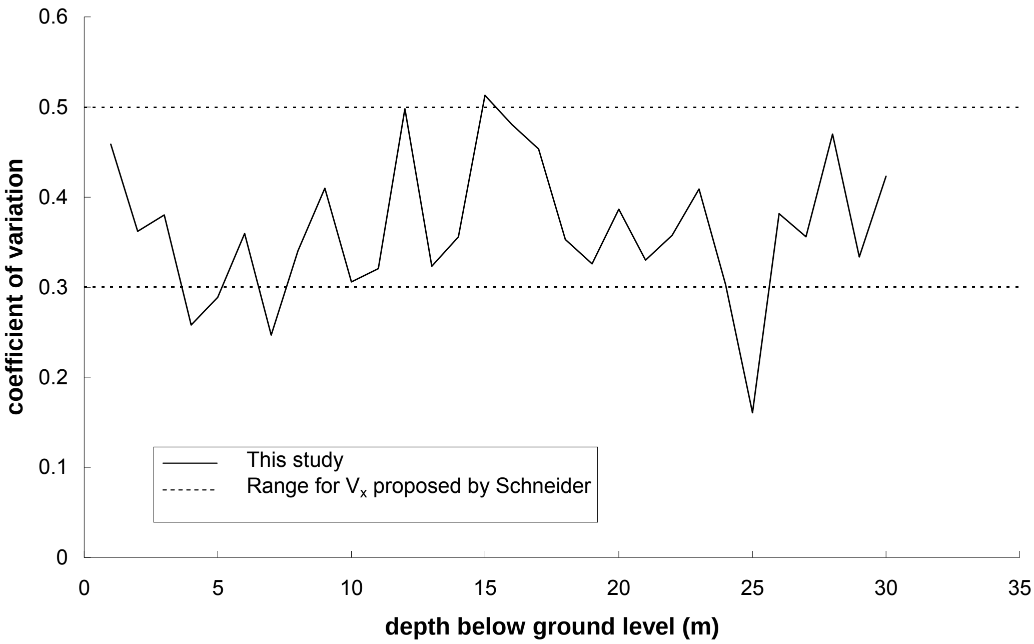

Figure 8. Coefficient of variation (standard deviation/mean) for the gathered data as grouped into 1m depth bands compared to published data[14].

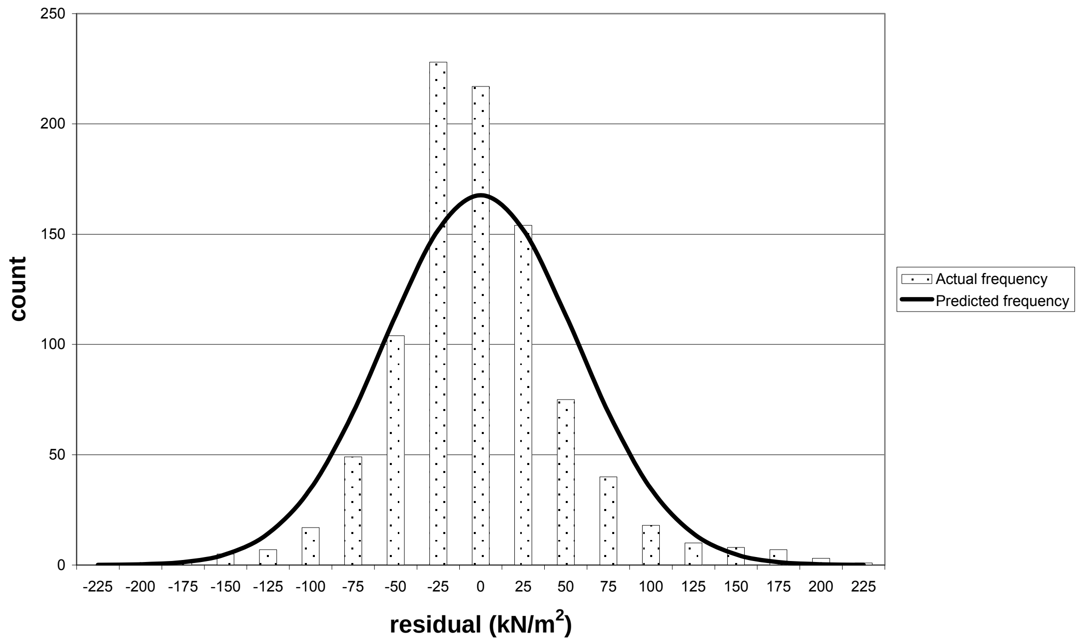

Figure 9. Distribution of shear strength test data (about the mean regression line with depth) from the London Clay database compared to the predicted normal distribution.

4 APPLICATION OF QUANTIFIED EXPERIENCE – CASE STUDIES

4.1 BAYESIAN UPDATING

The information collated in the database can be used as prior knowledge to refine, or update, the 'derived values' for soil properties from a site specific investigation, using Bayesian techniques as proposed by Tang[8]. The process is illustrated through application to a case study of pile design.

Shear strength is assumed to be normally distributed with mean, μ', and standard deviation, σ'. The site specific testing data were used to estimate the summary statistics for the site. A regression can be carried out to estimate the average at a particular depth $\bar{x}$ and the standard deviation, σ, and again a normal distribution was assumed.

The site for this case study is located near to Woolwich, South East London. The site investigation consisted of eight boreholes, from which 37 samples were tested in quick undrained triaxial tests. There were no appreciable superficial deposits reported, London Clay is present from just below the surface to an unproven depth beyond the investigation limits and anticipated pile toe depths. The test results are shown, with mean regression line, in Figure 44. The equation of the mean regression line for the site data is $c_u = 6.8d + 51.8$ kN/m^2.

It is important at this stage to consider the choice of a least squares regression line to represent the data. An experienced engineer may question the position of this line for several reasons such as the grouping of values at shallow depth which all lie below the line, or the large spread of values at greater depth. Using an engineering judgement approach to construct a line, extreme values may be investigated for sampling errors and be subsequently omitted or different weighting may be given to some of the points (such as the cluster at shallow depth). In the objective approach advocated in this paper, there is no room for such subjective weighting of data. A straight line is used as this is the same as that found for the much larger database, as shown in Figure 5. A bilinear summary of the data might seem intuitive because of the low strengths at shallow depth, however, the premise of this approach is that the site specific results are from the same population as the larger database and the straight line can therefore be used to represent this data. Errors may be introduced by the inclusion of all data including the extreme values. The influence of these errors is minimised, however, by applying equal significance to each data point and incorporating the additional data from other sites in the same statistical population through Bayesian updating. It must be stressed that engineering judgement is not completely removed from this process and it should not be applied blindly. The critical application of engineering judgement is in deciding on a suitable database of existing knowledge and the decision as to whether or not the site in question is really part of the same statistical population.

The data from the site specific investigation is refined using the set of data described in section 3.1. Each test result is updated in turn following the method discussed in section 3.2 and illustrated by the example below.

At 2 m below ground level, the depth of the first sample point, the estimated mean strength, $\bar{x}$, from the mean regression line is 65.4 kN/m^2. This is based on a linear regression of the 37 undrained shear strengths against depth, using the method of least squares. The average coefficient of variation for the site, v, has been calculated as 0.3. The standard deviation is found as the product of the coefficient of variation and the mean. The standard deviation for the site at 2 m is 19.6 kN/m^2. The prior information is sourced from the database of results which have been grouped into 1 m thick bands. The data from the 1.5 to 2.5 m band is therefore used for this calculation. From the database, the global mean at this depth is 64.4 kN/m^2; the standard deviation is 23.3 kN/m^2. Applying Equation 2 and Equation 3 provides a posterior mean, at 2 m, of 65.0 kN/m^2 and a standard deviation of 15.0 kN/m^2. This process can be repeated for each test sample location and this produces the posterior undrained shear strength versus depth profile shown in Figure 10. The site specific profile (derived values) and their trend line are also shown for comparison. The effect of applying the Bayesian updating technique is to produce a new mean profile for the site which is closer to the mean for the global population.

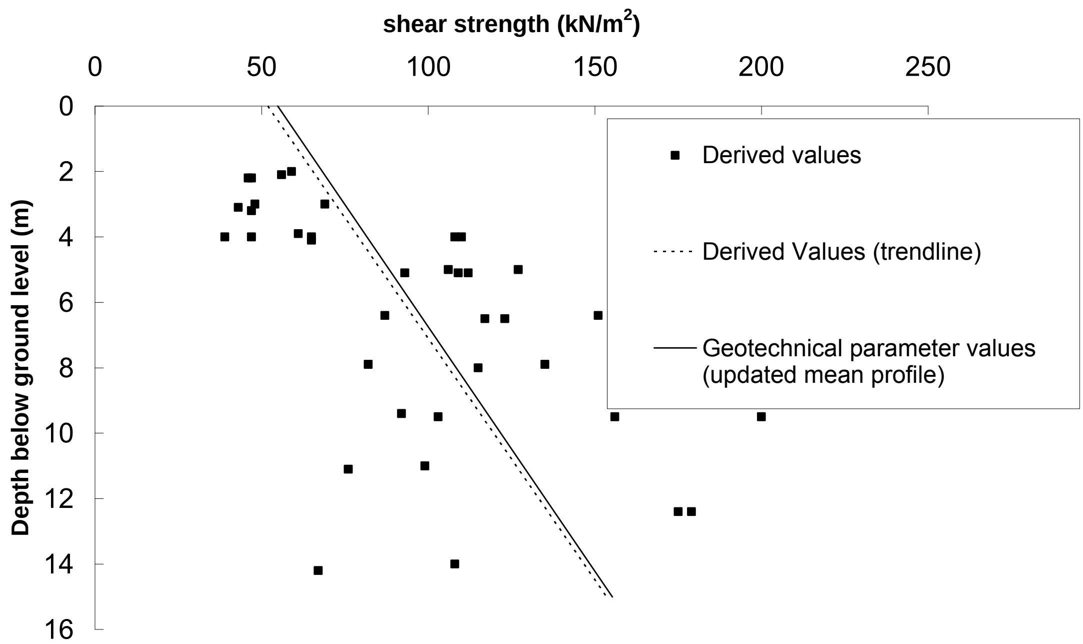

Figure 10. Bayesian updating of derived values of shear strength from the site near Woolwich, using the database as prior knowledge, to generate geotechnical parameter values

4.2 DETERMINATION OF CHARACTERISTIC VALUES

Using the approximation proposed by Schneider[14], characteristic values can be obtained from the summary statistics (mean and standard deviation) defined in section 4.1.

In the case study example, the posterior mean, μ'', at 2 m depth is 65.0 kN/m^2 and the standard deviation is 15.0 kN/m^2 (which leads to a coefficient of variation, V_x, of 0.24). Using Equation 4 this gives a characteristic value of 57.5kN/m^2. This process can be repeated for each value successively modifying each of the points in Figure 7 which represent the derived values to generate a characteristic value. These values can be described by a linear trend shown in Figure 11, the equation of which is $c_u = 65.7d + 51.2$ kN/m^2.

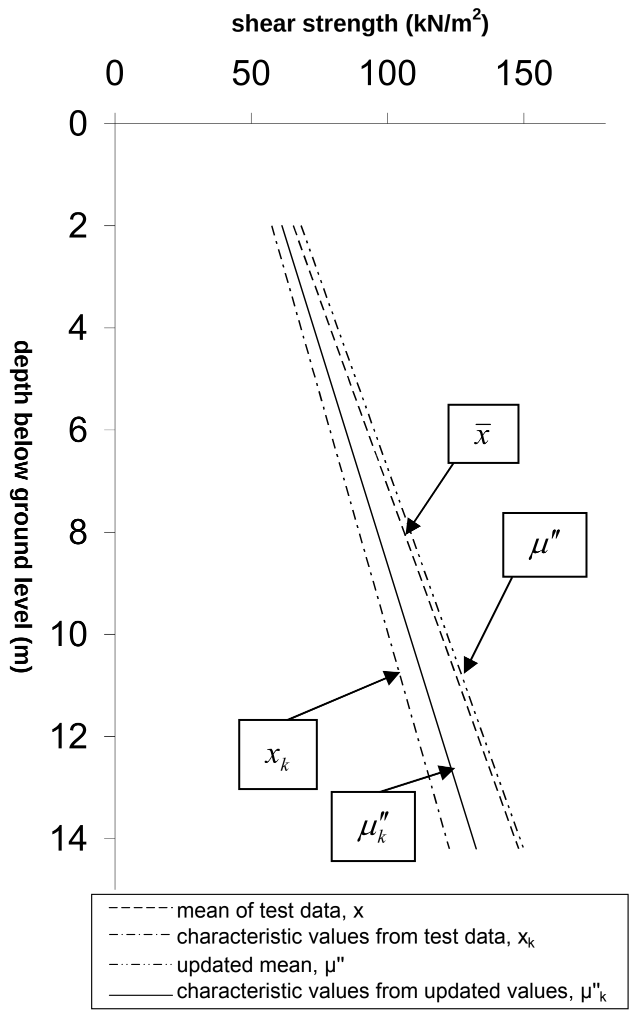

Figure 11. Determination of characteristic values by Schneider's approximation, Woolwich case study.

For comparison, the approximation by Schneider[14] has been applied to the test data without updating using the database information. The characteristic values obtained from the updated mean values are higher partly because of the increase in the mean that resulted directly from the updating but also because of the smaller coefficient of variation associated with the updated values. This is evident from the smaller difference between the mean and characteristic regression lines for the data which has been updated compared to the equivalent difference for the data which has not been updated.

4.3 SIGNIFICANCE FOR DESIGN

An example pile design at the site of the case study demonstrates the effect of this updating process. To carry a compressive, load of 350 kN, a typical solution would utilise continuous flight auger piles of 500 mm diameter. Such a pile would need to be 14 m long (measured from ground level) using the unrefined values. In this design it is assumed that there is no contribution to shaft capacity from the overlying fill material (which extends to 2 m below ground level) and the adhesion factor, α, is taken as 0.5 in the London Clay. If the updated values of shear strength are used, the pile would need to be 13.2 m, i.e. 800 mm shorter. The cumulative volume saved over a number of piles (120 in the case of this example) would produce significant cost savings in terms of materials volume (18.9 m^3 of concrete) and time for construction. Savings are of the order of 3-4% of the actual pile construction costs. Although such significant savings may have been found by fine-tuning and manipulating subjective interpretations of the variables based on experience and engineering judgement (e.g. selection of design line through site shear strength data), the approach presented in this paper allows such savings to be found in a justifiable, objective manner.

The application of Bayesian updating has the effect of moving the site mean towards that of the collated database mean. In the above example this gives a higher strength at any given depth than for the untreated site data and hence shorter pile lengths are obtained.

A second case study shows an example of where the derived values are lower following Bayesian updating than for the untreated site data. The site for this example is in Wimbledon, South West London. The site investigation consisted of 7 boreholes, from which 15 samples were tested. As in the previous example the London Clay is present from just below ground level to an unproven depth beyond the area of interest. The equation of the mean regression line for the site data is $c_u = 9.3d + 61.7$ kN/m^2. Figure 12 shows the mean of the test data, the updated mean values and their associated characteristic values, calculated following the same procedure as the previous example. In this case there would not be direct savings related to a reduction in the pile length as seen above but increased costs. The resulting increase in confidence, however, is discussed below.

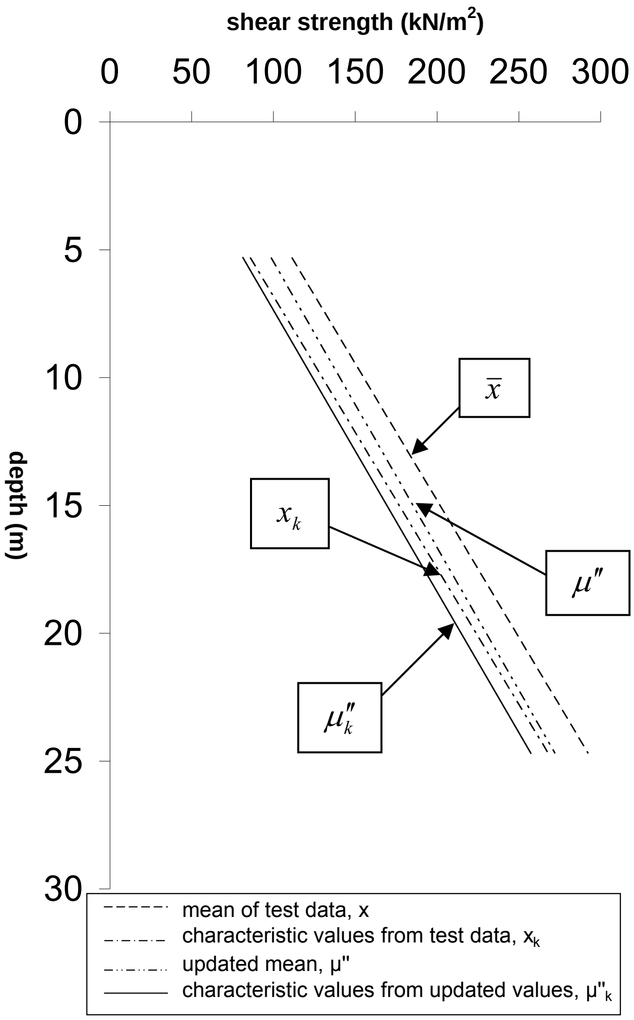

Figure 12. Determination of characteristic values by Schneider's approximation, Wimbledon case study.

5 DISCUSSION

It is essential for the engineer to make a judgement regarding the applicability of Bayesian updating before deciding whether to apply the technique. An assessment must be made of whether the site specific results are from the same population as the database. Where there are significant departures from the database mean values, it should be investigated whether there is a reasonable explanation, such as sampling difficulties. The engineer's judgement remains vital and the techniques presented here should be used as a tool to aid and quantify those judgements.

Bayesian updating can lead to direct savings, as demonstrated in the first case study in this paper which resulted in cost savings of 3-4%. The mean values may, however, be reduced by the updating process. The coefficient of variation will also reduce and this will to some extent counterbalance a decrease in the mean when the characteristic values are calculated. Moreover, application of Bayesian updating leads to a better estimate of the soil properties. Adopting the updated soil strength profile leads to a solution with a lower probability of failure and hence less risk is associated. A probabilistic comparison of alternative solutions can then be made and the most appropriate can be selected. Duncan[29] provides a simple framework for probabilistic comparison of alternative solutions. The product of the reduction in probability of failure associated with more reliable design and the cost of failure should be compared against the additional cost of adopting the more reliable design. In terms of piled foundations, the cost of failure could be the cost of constructing additional foundations following the failure of a test pile; more dramatically, and more likely given the small number of piles that are routinely tested, the cost of failure could be that of remedial works to underpin a structure which had settled by an excessive amount. The question of what constitutes an acceptable probability of failure is subjective and may be specific to the project and its interested parties.

6 CONCLUSIONS

The determination of ground conditions at a site and subsequent modelling of these conditions, allowing for variation and the nature of the interaction between the soil and structure, is fundamental in geotechnical design and is reliant upon the skill of the engineer. The inclusion of previous experience and existing knowledge is an important step in the process of characterising ground conditions as cost and practicalities can lead to even a well planned site investigation yielding only limited information. Wider experience and knowledge from sites with similar conditions is required to obtain statistical estimates of geotechnical properties. Prior information is often incorporated into the assessment of the soils for design by subjective means and while this remains a valid approach, objective methods may be called for.

One possible way to include prior knowledge quantitatively is to employ Bayesian updating. Where a property, such as shear strength, increases with depth a method of grouping data into bands has been applied enabling summary statistics to be calculated for use in the updating. The outputs of the updating process, namely a revised mean and standard deviation, can be used to describe the statistical distribution of properties and to better assess characteristic values. The effect of carrying out Bayesian updating on a set of site investigation data is to produce a revised mean which is a function of the site specific data and the prior information and which is weighted by their relative variations. Bayesian updating leads to more rigorous estimate of the soil properties which would lead to a lower likelihood of failure and a solution with less risk attached. This is of great interest to the engineer particularly if probabilistic comparisons of alternative solutions are to be made.

A large database of the shear strength of London Clay, from 947 quick undrained triaxial compression tests, has been collated and provides a description of the distribution of strengths at a given depth below ground level. Such a database can be used as a quantitative source of prior information in the Bayesian updating process described. Analysis of the database suggests that the coefficient of variation for the shear strength is in the range 0.2 to 0.5. A linear relationship between mean shear strength and depth has been assumed. The distribution of test values around this has been observed to approximate a normal distribution.

Further work on this topic could investigate similar trends and distributions for other properties of soils or to include shear strength data obtained using other site investigation techniques, such as the commonly used standard penetration test. Similar data gathering and analysis for other soils, soil types and regions would also be beneficial in providing prior data for geotechnical design. Sources of prior data are readily available in practice. Engineers and Contractors could make use of the large data resources that they hold from previous works to build similar databases to that described in this paper and to generate the necessary summary statistics.

7 ACKNOWLEDGEMENTS.

This work forms part of a research programme being undertaken at the Centre for Innovative and Collaborative Engineering (CICE), Loughborough University in association with Rock and Alluvium Ltd. and is supported by the UK Engineering and Physical Sciences Research Council.

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APPENDIX B PAPER 2

Full Reference

BAXTER, D.J., DIXON, N., FLEMING, P.R. AND CROMWELL, K.A., 2008, Using Experience And Case History Data To Enhance The Design Of Piled Foundations And Predict Behaviour Characteristics, *Proceedings of the 6th International Conference on Case Histories in Geotechnical Engineering,* Arlington VA, 11th-16th August

Abstract

This paper explores the process of piled foundation design and how it can benefit from the inclusion of previous test data and case histories from nearby or geologically similar sites.

The interaction between the soil and the structure is critical to the behaviour of a pile and is a function of both the ground conditions and the method of pile construction. An accurate model of the ground conditions is required for the design, as is a detailed knowledge of the method of pile installation and its subsequent interaction with the soil.

Where case histories are available they can be utilised to refine the design or to reduce the risk associated with a solution. This is currently often done in a subjective manner by the application of engineering judgement and personal experience. This paper discusses a quantitative method which can be used to employ data from case histories and provide an objective approach to the inclusion of existing knowledge and experience.

Bayesian updating is utilised to improve the model of the ground conditions and subsequently the degree of uncertainty is reduced. The probability of failure has been seen to be reduced by this process, as demonstrated through the application an example situation.

1 INTRODUCTION

The design of piled foundations is dependent upon knowledge of the ground conditions present at a site and how the chosen pile type will interact with the soil. The extent and accuracy of this knowledge and confidence associated with it is then reflected in the degree of conservatism required and the factors of safety which are then adopted.

The principal source of information for pile design is the site investigation, which may be supplemented by preliminary pile load test results. Local experience and a wider knowledge of the ground conditions or soil type can be useful in refining the conceptual model of the ground conditions. This experience is often employed subjectively through application of engineering judgement. Where case history information is available, it is prudent to consider its value to the design. In its simplest form this in done through consideration of case histories of failure as a warning and an indicator that additional conservatism is called for. There is, however, richer and more useful information available from even the most basic case history data and quantitative methods are available for the inclusion of this data in the characterisation of ground conditions, modelling of pile behaviour, quantification of risk and application to pile design.

The data which needs to be extracted from case histories, formats for collating and summarising this information are described in this paper. A method for including this case history data is presented and illustrated through example to demonstrate the benefits of applying the techniques discussed.

2 BACKGROUND

2.1 PILE DESIGN

There are four generally accepted approaches to pile design, which are used to a lesser or greater extent depending upon national standards, local practices, soil type and the site investigation data available. These are a total stress approach, an effective stress approach, empirical correlation with in-situ tests, and application of energy methods. These are all widely understood and accepted as valid design approaches.

2.1.1 TOTAL STRESS APPROACH.

The simplest design method for fine-grained soil, based on current UK practice would be a total stress approach. Pile capacity is related to the pile dimensions and the undrained shear strength of the soil through an empirical factor, α. The factor represents the degree of

softening undergone by the soil during the construction process. Bored pile design often relies on the values for α found by Skempton (1959). CFA piles would achieve values for α which are different from those for bored piles due to the different installation and construction processes. A programme of test pile analysis could yield values applicable to design.
For design purposes, Skempton calculates the shaft and end bearing capacities separately, the sum then represents the total capacity of the pile.

The end bearing capacity is shown to be sufficiently closely approximated by the formula:

$$Q_p = A_p \cdot N \cdot c_p \qquad \text{(kN)} \qquad (1)$$

Where A_p is the area of the base of the pile, N is a bearing capacity factor (generally taken as 9) and c_p is the undrained shear strength of the clay at the base.
The shaft capacity is given by

$$Q_s = A_s \cdot c_a \qquad \text{(kN)} \qquad (2)$$

Where A_s is the area of the shaft in contact with the soil and c_a is the adhesion between the clay and the pile shaft. The average adhesion is some fraction of the clay strength and can be written

$$c_a = \alpha \cdot \bar{c} \qquad \text{(kN/m}^2\text{)} \qquad (3)$$

Where α is less than unity, and not necessarily a constant, and $\bar{c}$ is the average undrained shear strength of the clay along the length of the pile.

Investigations into the magnitude of α have been reviewed above.

2.1.2 EFFECTIVE STRESS METHODS FOR PILE DESIGN.

The effective stress approach is simply explained by Searle (1979). It is similar to the simple friction model of a block sliding on a rough surface.

In terms of stress per unit area for a pile, the skin friction on a pile can be written:

$$q_s = K_s \sigma'_v \tan \phi'_r \qquad \text{(kN/m}^2\text{)} \qquad (4)$$

Where q_s is the shaft frictional stress, K_s is an empirical coefficient relating vertical stress, σ'_v, to horizontal stress σ'_h, and φ'_r is the residual effective stress angle of the soil.

Effective stress methods provide a reasonable, conservative design (Burland and Twine 1988), although there can be difficulties in determining horizontal stress accurately. This approach is often adopted for coarse grained soils where the horizontal stresses are easier to predict. For fine grained soils the effects of cohesion on the soil-structure interface can be difficult to account for. A cast *in-situ* pile also has a non-uniform shape and roughness to its surface which increases its capacity.

2.1.3 EMPIRICAL CORRELATION WITH IN-SITU TESTS.

As previously stated it is common in mainland Europe to design piles by correlation with in-situ test results. Designs may be based on Cone Penetrometer Test (CPT) data and use empirical factors to relate the cone and shaft resistances to the shaft and end capacities of the pile. By way of example Belgian practice is also similar and ninety percent of pile design in Belgium is based on semi-empirical formulae and CPT data (Holeyman, *et al.* 2001).

In essence, the shaft capacity is simply scaled up from that acting on the CPT, Bustamante and Gianeselli (1982) revised this standard approach and proposed values for the coefficient relating to scale, shape, material and installation effects.

Base resistance is calculated from the cone resistance, base area and up to three empirical factors representing shape (β), scale dependent soil shear strength characteristics (ε_b), and the soil type and installation method (α_b) as for the shaft capacity.

2.1.4 ENERGY METHODS FOR PILE DESIGN.

Energy methods for pile design are well documented and readily accepted for driven piles as an alternative design method.

Pile driving formulae are based on an energy balance between the dynamic energy input of the hammer and the static work required to advance the pile. The fundamental pile driving formula given by Fleming *et al.* (1992) is:

$$R = \frac{\eta W h}{(s + c/2)} \quad \text{(kN)} \qquad (5)$$

Where R is the pile resistance, η is the efficiency of the hammer, W is the weight of the hammer, h is the drop height, s is the permanent set of the pile, and c is the elastic movement of the pile.

2.1.5 SUMMARY OF PILE DESIGN APPROACHES.

Whichever approach is chosen, it must rely either directly or indirectly on the results of load testing and the application of empirical values derived from such tests. The inclusion of case history data is therefore implicit in any pile design and hence it is consistent to argue that the formal inclusion of case history data from nearby and geologically similar sites is useful in improving the model of both the ground conditions and the pile soil interaction.

2.2 HOW GROUND CONDITIONS ARE MODELLED

Whichever design approach is adopted, it is necessary to construct a model of the ground properties which exist at a site. It is the construction of this model which gives the engineer the greatest opportunity to influence the design. Constraints exist on other variables but in the selection of suitable soil properties (e.g. undrained shear strength) the engineer has the freedom to include and allow for experience, prior knowledge, judgements and interpretation of the ground conditions. Usual practice is to base this model on test results from site investigation. Test results alone cannot be used for design calculations as account must be taken of variability across the site, the nature of the tests and the manner in which the structure will interact with the soil. The process of determining values for use in design from site investigation measurements follows a straightforward progression throughout which the designer will make and apply numerous decisions and judgements.

This process for determination of ground properties (following Eurocode 7, Part 1 (British Standards Institute 2004) has been described by Orr (1993) and by Frank *et al* (2004). Figure 1 shows the individual steps necessary to establish ground properties which can be used in design. The steps are described below and follow the development of soil property values through four stages – measured values, derived values, characteristic values and design values. Frank *et al.* (2004)introduced intermediary steps, most notably the 'geotechnical parameter value' which allows for consideration of existing knowledge and experience.

Ground properties are established primarily from site investigation through tests carried out *in-situ* and on samples (disturbed or undisturbed) taken for laboratory testing. These are referred to as the 'measured values' and represent the test results (following the application of any test related corrections, which are independent of further analysis). Where test results do not return a value which can be used directly in design, these need to be converted to 'derived values' by applying theory, empiricism or correlations. An example of this is the correlation of SPT blow counts to undrained shear strength through the relationship proposed by Stroud and Butler (1975). The 'geotechnical parameter value' is an intermediate stage which allows for an assessment of the influence of the test, with associated corrections, such as a conversion from axisymmetric to plane strain conditions to better represent the design situation. This is also the point at which refinements can be made based upon published data and general experience or prior knowledge.

The derived values (and geotechnical parameter values) are only representative of the sample and not of the overall distribution of values on the site. The values which describe the properties of the ground at the site, its associated variation, the nature and behaviour of the soil mass and its interaction with the structure are the 'characteristic values'. This characterisation of the site and the geotechnical problem is a vital step in the design and is where the engineer's skill and judgement is applied. It is in the determination of characteristic values that the engineer has the opportunity to include information from case histories.

The characteristic values are subsequently developed into to 'design values' by the application of safety factors.

2.3 METHODS OF INSTALLATION AND CONSEQUENT EFFECTS

Piles may be crudely classified as either displacement or replacement types; according to their method of construction. Within each classification there is a multitude of variations in the type of pile and its method of installation. Tomlinson notes that each type and method of installation will disturb the ground in a different way, the degree of this disturbance and its effect is not well enough understood to be represented solely by soil mechanics theory. Empirical factors are called for to model the pile-soil interaction and effects of installation; these factors have been developed from results of pile load tests and experience.

Whichever pile design approach is selected, it relies upon parameters which have been derived from pile load tests to model the influence of the chosen method of construction of the pile.

2.3.1 DETERMINATION OF CHARACTERISTIC VALUE.

The characteristic value is defined in Eurocode 7, Part 1(British Standards Institute 2004) as a cautious estimate of the value affecting the occurrence of the limit state. The characteristic value must take account of the inherent variability of the soil, measurement errors and the extent of the zone governing behaviour (Dixon *et al.* 2002), further factors leading to differences between derived values and those governing behaviour are listed by Frank *et al.* (2004). These include, but are not limited to, the presence of soil structure (e.g. fissures), time effects, water softening and the influence of construction activities on the soil

The characteristic value, used in Eurocode 7 (British Standards Institute 2004), is comparable to the 'conservatively chosen' mean which is traditionally used in the British Standard approach for foundation design[11]. It is common that the degree of conservatism which is selected by the designer has been based on local experience or subjective information. One way of achieving an objective measure of the degree of conservatism is through the use of statistical methods. Eurocode 7 (British Standards Institute 2004) stipulates that where statistical methods are employed the characteristic value should be selected such that the calculated probability of a worse value governing the occurrence of the limit state considered should not be greater than 5%.

The introduction of statistical techniques can add unnecessary complication to the design process and demands additional skills from the engineer. This can be overcome by using an easily employed approximation for the statistical definition of the characteristic value, such as that proposed by Schneider (1997) and described by Equation 6.

$$X_k = X_m(1 - \frac{V_x}{2}) \qquad (6)$$

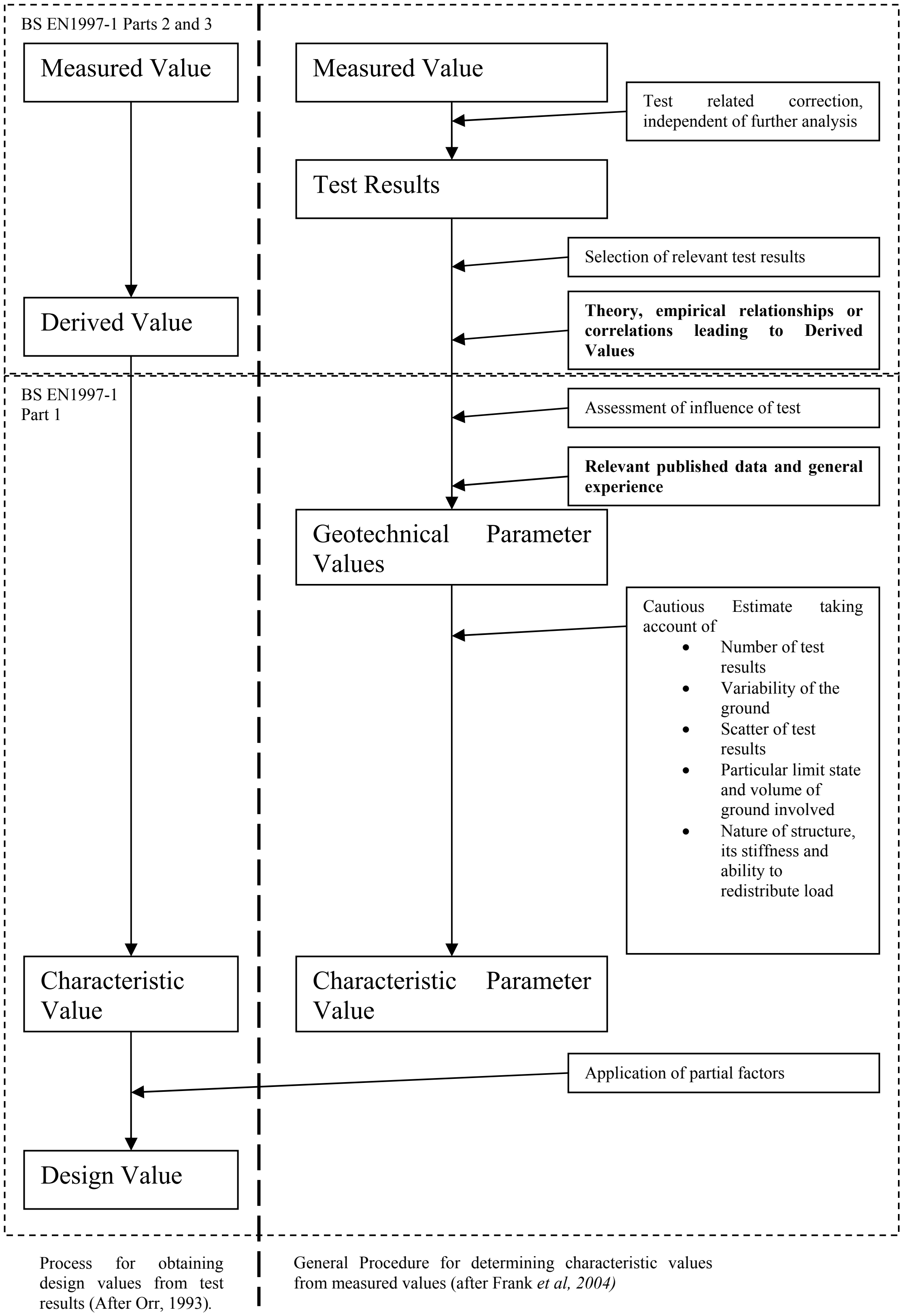

Fig. 1. Steps in determining characteristic and design values (After Orr (1993)and Frank et al. (2004))

Where X_k is the characteristic value, X_m is the statistical mean of the derived values and V_x is the coefficient of variation of the derived values. X_m and V_x can be estimated from the sample distribution when there are sufficient data to provide an approximation of the population. Schneider[13] found the coefficient of variation, V_x, of the undrained shear strength of a soil to be in the range 0.3 to 0.5 and recommended a value of 0.4. Many other authors have found similar results (Hooper and Butler 1966, Lumb 1966, Phoon and Kulhawy 1999) Schneider (1997) demonstrated that approximations of characteristic value using equation 6 to be consistent with the values estimated by engineers and they have been in use in Switzerland for many years. A sufficiently accurate solution can be obtained without extensive calculations and this simplicity makes it appealing to practising engineers.

2.4 ESTIMATES OF RISK AND RELIABILITY IN GEOTECHNICAL ENGINEERING

The selection of factors of safety in geotechnical engineering often stems from experience and empirical values. A calculation or estimate of the probability of failure can permit evaluation of the degree of uncertainty attached to a design and a particular factor of safety, which can vary widely. Duncan (2000) provides a method of estimating the probability of failure using a Taylor series method. This method requires little or no additional information other than that which is typically available and used for a standard deterministic design.

The method can be summarised in the following steps.

1. Determine the most likely value (MLV) of the parameters and the associated factor of safety F_{MLV}. MLV is determined from the known information relating to a parameter and may be based on a statistical average or an experiential judgement.

2. Estimate the standard deviations of the parameters.

3. Calculate the factors of safety with each parameter increased then decreased by 1 standard deviation in turn. In each case calculate the change in factor of safety, ΔF

4. Calculate the standard deviation, σ_F, and hence the coefficient of variation f the factor of safety using Equation 7

$$\sigma_F = \sqrt{(\frac{\Delta F_1}{2})^2 + (\frac{\Delta F_2}{2})^2 + \ldots + (\frac{\Delta F_n}{2})} \qquad (7)$$

5. Determine the probability of failure P_f from F_{MLV} and the coefficient of variation of the factor of safety, V_F, either by calculation or from tables.

3 METHOD FOR INCORPORATING PREVIOUS DATA AND CASE STUDY INFORMATION

3.1 BAYESIAN UPDATING

If information is not sufficient for design, the result is a lack of confidence in the derived values that leads to greater conservatism being applied. Lack of confidence is reflected further along the design process when larger factors of safety are applied. Larger factors of safety are required to yield an acceptable probability of failure.

Where previous experience, published data, or knowledge from nearby similar sites exists, a method of combining this information with the site specific data is advantageous in developing characteristic values. Traditionally, this may have been achieved by application of subjective judgement based on experience. An objective approach is suggested by authors such as Lumb (1966) and Tang (1971), who have presented methods for the application of statistics, in particular Bayes' theorem, to geotechnical applications. A better estimate of the likely value of the geotechnical parameter values (posterior distribution) can be found by combining information relating to the previous knowledge (prior distribution) and the new site specific information. Tang (1971) observed that the posterior distribution is proportional to the product of the distributions of the prior information and the new data (in this case derived values for a site). This method requires, as a minimum, that summary statistics such as the mean and standard deviation of the previous knowledge are known or can be determined in some way.

If a population is normally distributed, it can be shown, by application of Bayes' theorem that the posterior distribution is normal with the mean value and standard deviation calculated using Equations 8 and 9.

$$\text{Posterior mean, } \mu'' = \frac{\mu' \dfrac{\sigma^2}{n} + \bar{x}(\sigma')^2}{\dfrac{\sigma^2}{n} + (\sigma')^2} \quad (8)$$

$$\text{Posterior standard deviation, } \sigma'' = \sqrt{\frac{\dfrac{\sigma^2}{n}(\sigma')^2}{\dfrac{\sigma^2}{n} + (\sigma')^2}} \quad (9)$$

Where the prior data has mean μ' and standard deviation σ' and the derived values for the site are represented by the mean $\bar{x}$ and standard deviation σ.

The resulting posterior distribution has a mean which is a weighted average of the prior and sample data means, the weighting being proportional to the variances. The posterior standard deviation will be lower than that for both the prior and sample data distributions as they combine to produce a sharper, more peaked distribution, i.e. data points are more closely grouped about the mean.

4 QUANTIFYING PREVIOUS EXPERIENCE

Consideration of previous knowledge and experience can assist in obtaining characteristic values which better represent the site conditions or provide greater confidence. In order to apply the objective method introduced in this paper, a quantification of the existing knowledge or experience is required. In the following section, information relating to the shear strength of London Clay is collated and processed by way of an example of how prior knowledge might be quantified for use in such calculations and characterisation. The information is also useful for the probabilistic analysis and design of piles as shown in the case study.

4.1 CONSTRUCTION OF DATABASE FOR SHEAR STRENGTH OF LONDON CLAY

In order to quantify existing knowledge relating to the mean value and variation of shear strength of London Clay, a database of site investigation data has been constructed. (Baxter *et al,* 2007.). The is drawn from 68 sites in the London Basin, predominantly in the Greater London area; There were results from 947 quick undrained triaxial compression tests conducted on undisturbed samples from 234 boreholes. The data is drawn from investigations by 31 different SI contractors carried out for piling contracts between 2003 and 2006. There were between 1 and 11 boreholes at each site with shear strengths reported from between 1 and 24 samples per borehole. Shear strengths are recorded against depth below site ground level.

If the collated data is treated as a single continuous dataset, a regression line can be used to describe the mean value of shear strength of the London Clay with depth. A linear trend for strength against depth below ground surface for the entire dataset is shown in Fig. 2; a straight line regression has been performed on the data using the method of least squares. The equation of the line is $C_u = 6.1d + 60.0$ kN/m^2, where C_u is the undrained shear strength and d is the depth below ground level; the fit of the line to the data has a correlation coefficient (R^2) of 0.43. A straight line is consistent with previous studies (Hooper and Butler 1966, Patel 1992, Whitaker and Cooke 1966). Other types of line (power, logarithmic, exponential and polynomial) do not provide significantly greater correlation.

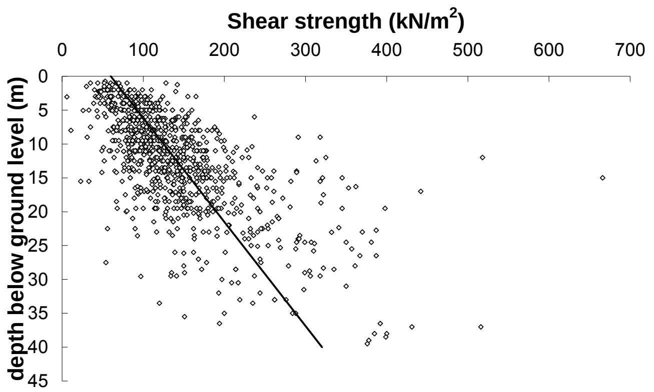

Fig. 2. Shear strength against depth for all sites in dataset, with trend line.

The collated data has been compared to existing published data for the undrained shear strength of London Clay by Baxter *et al.* (2007) and is consistent with that previously reported.

An additional benefit of the data presented in this paper is that information relating to the distribution of the data is also available, thus allowing estimates of the coefficient of variation to be made. The size of the database means that it can be assessed and manipulated using statistical techniques. Information relating to the distribution and variation also permits probabilistic analysis to be performed as an alternative or in addition to traditional deterministic analysis.

4.2 DATA PROCESSING

The data in its raw format, or simply summarised by a regression line, cannot be easily employed as prior knowledge to update site specific information using the techniques described above. A method of processing the data is required such that a mean and a measure of the distribution can be readily obtained for use in Bayesian updating. The method proposed by Baxter *et al.* (2007) and adopted in this paper is to group the strength data by depth to top of sample and to calculate summary statistics for each group; bands of 1 m thickness have been used for the grouping; depths have been measured from ground level. Each depth band extends from 0.5 m above the nominal depth to 0.5 m below. The summary statistics and the number of sites and data points used to generate them are shown in Table 1. The mean values for each band are plotted in Fig. 3, overlaid by the mean regression line found previously for all test results (first shown in Fig. 2) and a reasonable fit can be observed, particularly for depths between 0 and 20 metres where a greater number of data points are available. The deviations which occur below 20 m are likely to be a result of the low number of test results available at these depths as many boreholes used to build the database did not extend below 20 m.

The coefficients of variation shown in Table 1 are in general agreement with that reported in previous literature (Schneider 1997, Hooper and Butler 1966, Phoon and Kulhawy 1999)

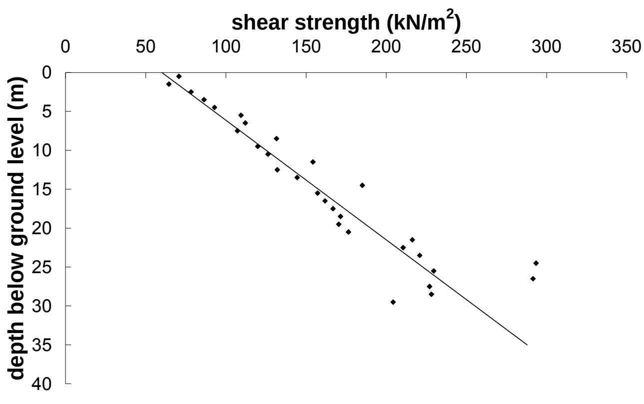

Fig 3. Mean values for shear strength grouped into 1 m thick depth bands, shown with the original regression line for all data points.

5 EXAMPLE APPLICATIONS

The information collated in the database can be used as prior knowledge to refine, or update, the 'derived values' for soil properties from a site specific investigation, using Bayesian techniques as proposed by Tang (1971). The process is illustrated through application to a case study of pile design.

Shear strength is assumed to be normally distributed with mean, μ', and standard deviation, σ'. The site specific testing data were used to estimate the summary statistics for the site. A regression can be carried out to estimate the average at a particular depth $\bar{x}$ and the standard deviation, σ, and again a normal distribution was assumed.

The site for this case study is located near to Woolwich, South East London. The site investigation consisted of eight boreholes, from which 37 samples were tested in quick undrained triaxial tests. There were no appreciable superficial deposits reported, London Clay is present from just below the surface to an unproven depth beyond the investigation limits and anticipated pile toe depths. The test results are shown, with mean regression line, in Fig 4. The equation of the mean regression line for the site data is $C_u = 6.8d + 51.8$ kN/m^2.

The data from the site specific investigation is refined using the database collated by Baxter *et al*. (2007) Each test result is updated in turn following the method discussed in section 3.2 and illustrated by the example below.

At 2 m below ground level, the depth of the first sample point, the estimated mean strength, $\bar{x}$, from the mean regression line is 65.4. The coefficient of variation for the site, σ, has been calculated as 0.3. The standard deviation is found as the product of the coefficient of variation and the mean. The standard deviation for the site at 2 m is 19.6. The prior information is sourced from the database of results which have been grouped into 1 m thick bands. In the case of this example calculation, the depth of the first test sample was 2 m; the data from the 1.5 to 2.5 m band is therefore used for this calculation. From the database, the global mean at this depth is 64.4 kN/m^2; the standard deviation is 23.3 kN/m^2. Applying Equations 8 and 9 provide a posterior mean, at 2 m, of 65.0 kN/m^2 and a standard deviation of

15.0 kN/m^2. This process can be repeated for each test sample location; this produces the posterior undrained shear strength versus depth profile shown in Fig 5. The site specific profile (derived values) and their trend line are also shown for comparison. The effect of applying the Bayesian updating technique is to produce a new mean profile for the site which is closer to the global mean.

Table 1. Statistical analysis of shear strength data grouped by depth for London Clay (after Baxter et al. (2007))

Nominal Depth (m)	Number of sites	Number of test results	Mean shear strength, X_m (kN/m^2)	Coefficient of variation, V_x
1	9	12	70.67	0.46
2	14	28	64.43	0.36
3	23	38	78.24	0.38
4	20	42	86.31	0.26
5	34	57	92.89	0.29
6	20	31	109.42	0.36
7	27	42	112.12	0.25
8	43	67	107.22	0.34
9	25	38	131.58	0.41
10	34	58	119.84	0.31
11	42	71	126.19	0.32
12	23	39	154.26	0.50
13	31	49	132.04	0.32
14	39	64	144.41	0.36
15	25	40	185.06	0.51
16	19	29	157.17	0.48
17	21	37	161.81	0.45
18	13	22	166.82	0.35
19	18	24	171.46	0.33
20	20	37	170.33	0.39
21	10	14	176.46	0.33
22	2	7	216.29	0.36
23	10	20	210.53	0.41
24	9	13	220.85	0.30
25	5	10	293.40	0.16
26	2	7	229.57	0.38
27	1	4	291.50	0.36
28	2	7	227.00	0.47
29	5	8	228.13	0.33
30	3	7	204.29	0.42

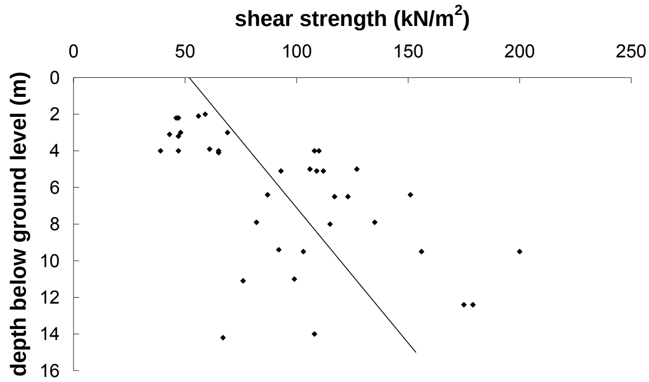

Fig 4. Example shear strength data, with linear regression trend line, from site investigation at a site near Woolwich.

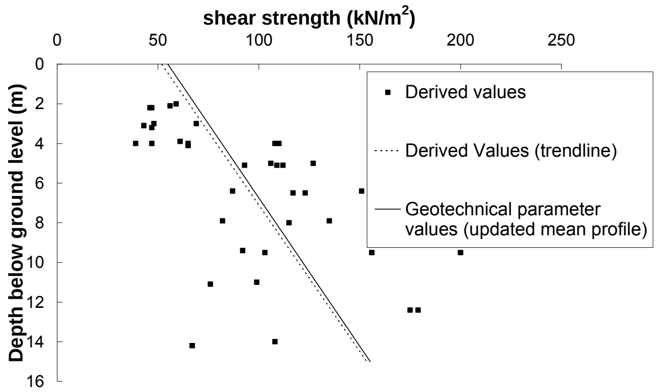

Fig 5. Bayesian updating of derived values of shear strength from the site near Woolwich, using the database as prior knowledge, to generate geotechnical parameter values

5.1 DETERMINATION OF CHARACTERISTIC VALUES

Using the approximation proposed by Schneider (1997), characteristic values can be obtained from the summary statistics (mean and standard deviation).

In the case study example, the posterior mean, μ'', at 2 m depth is 65.0 kN/m^2 and the standard deviation is 15.0 kN/m^2 (which leads to a coefficient of variation, V_x, of 0.24). Equation 6 this gives a characteristic value of 57.5kN/m^2. This process can be repeated for each value and the resulting characteristic values are shown in Fig. 5. These values can be described by a linear trend, the equation of which is $C_u = 65.7d + 51.2$ kN/m^2.

For comparison, the approximation by Schneider (1997) has been applied to the test data without updating using the database information. The characteristic values obtained from the updated mean values are higher partly because of the increase in the mean that resulted directly from the updating but also because of the smaller coefficient of variation associated with the updated values. This is evident from the smaller difference between the mean and characteristic regression lines for the data which has been updated than the equivalent difference for the data which has not been updated.

5.2 SIGNIFICANCE FOR DESIGN

An example pile design at the site of the case study demonstrates the effect of this updating process upon the pile dimensions and the probability of failure associated with the design. A probabilistic design has been carried out using the Taylor Series approach after Duncan, (2000) firstly for the untreated values from the site investigation and secondly for the values which have been updated using Bayes' theorem after Tang.

To maintain clarity and simplicity in this example, only shaft capacity will be considered. The design is for a 500 mm diameter bored pile to carry a load of 500 kN, In this design it is assumed that there is no contribution to shaft capacity from the overlying fill material (which extends to 2 m below ground level). The pile (shaft) capacity has been calculated by application of Equations 2 and 3.

5.2.1 DESIGN USING ONLY NEW SITE DATA.

For a factor of safety, F_{ss}, of 2 a pile length of 17m (15m pile length in London Clay) is required. If these dimensions are fixed, the effect of variation on the factor of safety and the associated probability of failure can be calculated from the values given in Table 2 using the method proposed by Duncan (2000), as shown by Table 3.

Variations in the pile diameter and pile length have negligible effect of the factor of safety achieved and have therefore not been considered. The values selected for the variables are shown in Table 2. The most likely value (MLV) of average shear strength acting over the length of the pile is found from the characteristic profile shown in Fig 6 (without updating). The standard deviation has been estimated from the variation displayed by the site investigation data. The MLV of alpha is taken as the mean value of alpha found by Patel

(1992) (using maintained load tests) for bored pies in London Clay, the standard deviation is also calculated from the same source.

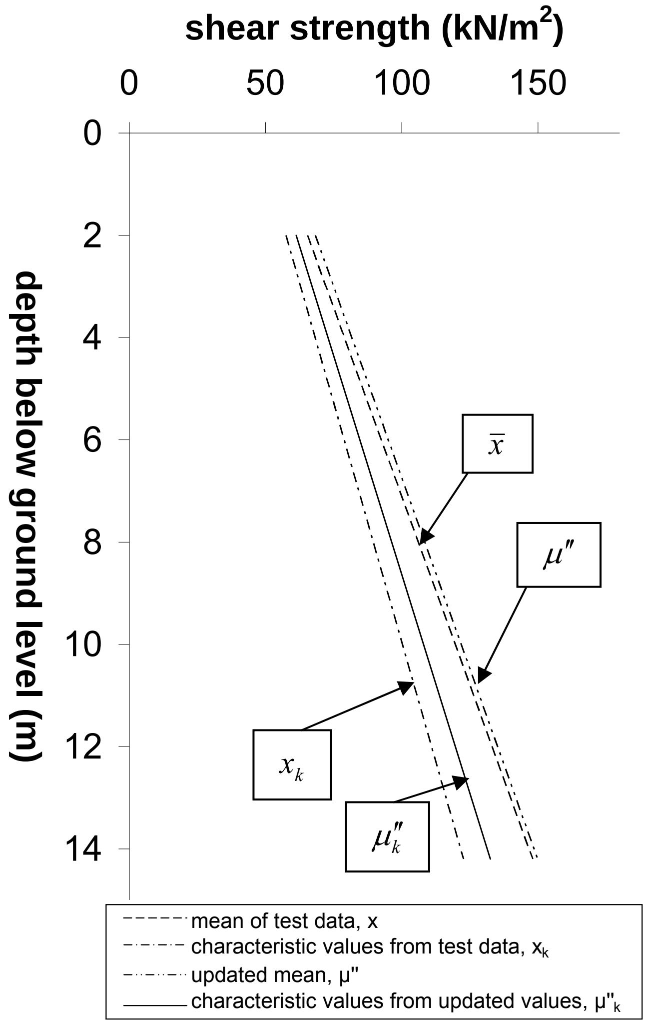

Fig 6. Determination of characteristic values by Schneider's approximation, Woolwich case study.

The standard deviation of the factor of safety is therefore equal to 0.67 and hence the coefficient of variation is equal to 0.33. Using the tables provided by Duncan for the probability of failure based on the Taylor series, the probability of failure P_f is 2.41%. The effect of including case history data upon this probability of failure can be seen by re-evaluating the design using characteristic values calculated from the updated values.

Table 2. Design Variables (site data)

Variable	MLV	σ
Shear strength	97	29
α	0.45	0.06

Table 3. Calculation of probability of failure after Duncan (2000) (site data).

<table>
<tr><th colspan="2">Variable</th><th>Values</th><th>F</th><th>ΔF</th></tr>
<tr><td rowspan="2">Shear strength</td><td>MLV+σ</td><td>126</td><td>2.67</td><td rowspan="2">1.23</td></tr>
<tr><td>MLV-σ</td><td>68</td><td>1.44</td></tr>
<tr><td rowspan="2">α</td><td>MLV+σ</td><td>0.51</td><td>2.33</td><td rowspan="2">0.55</td></tr>
<tr><td>MLV-σ</td><td>0.39</td><td>1.78</td></tr>
<tr><td colspan="5">Standard deviation of F,
$\sigma_F = \sqrt{(\frac{\Delta F_1}{2})^2 + (\frac{\Delta F_2}{2})^2}$ =0.67
Coefficient of variation of F,
V_F=0.67/2.06=0.33
Probability of failure, $P_{failure}$=2.41% (from tables)</td></tr>
</table>

5.2.2 DESIGN USING THE DATA UPDATED WITH CASE HISTORY INFORMATION.

The application of Bayesian updating has the effect of moving the site mean towards that of the collated database values. In this example this gives a higher strength at any given depth than for the untreated site data and hence shorter piles are required. This leads to the required pile length being reduced to 16m in order that the same overall factor of safety is achieved (F_{ss}=2). The standard deviation has been reduced through the Bayesian updating. Table 4 shows the MLV and standard deviation from the updated values.

As before the probability of failure is calculated as shown in Table 5. The probability of failure is now greatly reduced and is below 1%.

Table 4. Design Variables (updated data)

Variable	MLV	σ
Shear strength	103	21
α	0.45	0.06

Table 5. Calculation of probability of failure after Duncan (2000) (updated data)

<table>
<tr><th colspan="2">Variable</th><th>Values</th><th>F</th><th>ΔF</th></tr>
<tr><td rowspan="2">Shear strength</td><td>MLV+σ</td><td>124</td><td>2.45</td><td rowspan="2">0.83</td></tr>
<tr><td>MLV-σ</td><td>82</td><td>1.62</td></tr>
<tr><td rowspan="2">α</td><td>MLV+σ</td><td>0.51</td><td>2.31</td><td rowspan="2">0.54</td></tr>
<tr><td>MLV-σ</td><td>0.39</td><td>1.77</td></tr>
<tr><td colspan="5">$\sigma_F = \sqrt{(\frac{\Delta F_1}{2})^2 + (\frac{\Delta F_2}{2})^2}$ =0.49
V_F=0.49/2.04=0.24
$P_{failure}$=0.36% (from tables)</td></tr>
</table>

6 DISCUSSION

It is essential for the engineer to make a judgement regarding the applicability of Bayesian updating before deciding whether to apply the techniques the process. An assessment must be made of whether the site specific results are from the same population as the database. Where there are significant departures from the database mean values, it should be investigated whether there is a reasonable explanation, such as sampling difficulties. The engineer's judgement remains vital and the techniques presented here should be used as a tool to aid and quantify those judgements.

Bayesian updating can lead to direct savings, as demonstrated in the example in this paper which resulted in a marked reduction in pile length. This may not always be the case as the mean shear strength values may be reduced by the process. The outcome which is of notable interest here is that the application of Bayesian techniques leads to a better estimate of the soil properties this is evident from the reduced coefficient of variation. It is striking that even with shorter pile lengths (as in the example) adopting the updated soil strength profile leads to a lower probability of failure and a solution with reduced risk associated.

A comparison can then be made to select the most appropriate solution. The product of the reduction in probability of failure associated with more reliable design and the cost of failure should be compared against the additional cost of adopting the more reliable design. In terms of piled foundations the cost of failure could be the cost of constructing additional foundations following the failure of a test pile; more dramatically, and more likely given the small number of piles that are routinely tested, the cost of failure could be that of remedial works to underpin a structure which had settled by an excessive amount.

7 CONCLUSIONS

The determination of ground conditions at a site and subsequent modelling of these conditions, allowing for variation and the nature of the interaction between the soil and structure, is fundamental in geotechnical design and is reliant upon the skill of the engineer. The inclusion of previous experience and existing knowledge is an important step in the process of characterising ground conditions as cost and practicalities can lead to even well planned site investigation yielding only limited information. Wider experience and knowledge from sites with similar conditions is required to obtain statistical estimates of geotechnical properties. Prior information is often incorporated into the assessment of the soils for design by subjective means and while this remains a valid approach, objective methods may be called for.

Bayesian updating has been shown to be a useful tool in the quantitative inclusion of case history data and prior knowledge into geotechnical design. Where a property, such as shear strength, increases with depth a method of grouping data into bands has been applied enabling summary statistics to be calculated for use in the updating. The outputs of the updating process, namely a revised mean and standard deviation, can be used to describe the statistical distribution of properties and to better assess characteristic values. The effect of carrying out Bayesian updating on a set of site investigation data is to produce a revised mean which is an average of the site specific data and the prior information and which is weighted by their relative variations. Bayesian updating leads to a better estimate of the soil properties which would lead to a lower likelihood of failure and a solution with less risk attached. Probabilistic comparisons can be made between alternative solutions; this should be of interest to the engineer as it allows comparisons to be made.

A large database of the shear strength of London Clay, from 947 quick undrained triaxial compression tests, has been used as the source of prior information and provides a description of the distribution of strengths at a given depth below ground level.

Further work on this topic could investigate similar trends and distributions for other properties of soils or to include shear strength data obtained using other site investigation techniques, such as the commonly used standard penetration test. Similar data gathering and analysis for other soils, soil types and regions would also be beneficial in providing prior data for geotechnical design. Sources of prior data are readily available in practice, Engineers and Contractors can make use of the large data resources that they hold from previous works to build similar databases to that described in this paper and to generate the necessary summary statistics

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APPENDIX C PAPER 3

Full Reference

BAXTER, D.J., DIXON, N., FLEMING, P.R. AND HADLEY S.H., 2006, Bored Displacement Piles – A United Kingdom Perspective, *Proceedings of the 10th International conference on Piling and Deep Foundations*, Amsterdam, 31st May – 2nd June, pp210-218

Abstract

Use of bored displacement piles has experienced significant growth in the UK piling market sector, largely due to their environmental advantages over other piling methods and through cost benefits. Bored displacement piles are a cast-*in-situ* construction which can be formed with minimal production of spoil and offer low noise and vibration compared to driven displacement piles. A large variety of bored displacement piles have been developed. This paper describes a standard system for their classification. The development of the different types is described along with the factors which have led to their increased use. Consideration is given to processes undergone by the soil during pile installation and design approaches for bored displacement piles are compared. Comparisons are made between the different classifications of bored displacement pile. Comments are also made on the installation of such piles. It is concluded that complex changes occur in the stress states of the surrounding soils. Such effect of the pile installation on the soil should be accounted for and there is a need, therefore, to further refine the design process for these piles.

1 INTRODUCTION

A bored displacement pile is a type of piled foundation formed using a shaped auger to cause lateral displacement of the earth to form a void. A reinforced concrete pile is then formed *in-situ* in the void which has been formed. Many different types of bored displacement pile exist, along with a surfeit of names for the process. The term bored displacement has been selected here as the generic term for such piles as it is descriptive of the process without being limited to, or suggestive of, one particular method or pile shape.

The use of bored displacement piles in the UK is increasing, largely due to the requirement to develop brownfield sites and to avoid the rising cost of spoil disposal. As their use increases, the need to understand and to be able to define and compare pile types becomes more important. There is also a requirement for accurate, efficient design techniques, which allow for the complex processes undergone by the soil during construction.

This paper explores the development of the bored displacement pile and aims to classify the diverse methods used to construct them. Consideration is given to the changes undergone in the soil during the construction and the design implications. Finally there is discussion of some of the important issues surrounding the pile installation, such as suitability of different soil types and production rates.

2 PILE CLASSIFICATION

Piles have traditionally been classified as either displacement or non-displacement types, according to their method of construction. Bored displacement piles clearly fall into the category of displacement piles. However, the use of boring techniques is more commonly associated with non-displacement piles; this leads to bored displacement piles sharing some properties within both pile classification divisions. Fig. 1 shows how bored displacement piles can be related to other pile types.

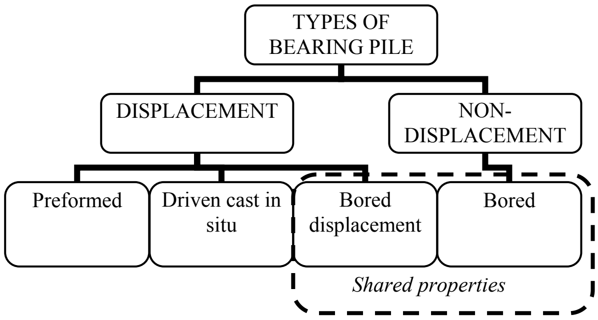


Figure 1. Types of bearing pile

A bored displacement pile benefits from the improvement to the surrounding soil caused by the displacement (and any subsequent action). Like a bored pile, however, it also benefits from having a pile/soil interface which is formed *in-situ*, the irregular nature of which can improve load transfer.

The ambiguity in the nature of a bored displacement pile is problematic with regard to specification of relevant standards. Current standards in Europe for displacement piles such as 'Execution of special geotechnical work. Displacement piles. BS EN 12699:2001' (European Committee for Standardization (CEN), 2001) makes little reference to piles constructed in this, or similar fashion; it concentrates instead on driven displacement methods, yet a bored displacement pile cannot be classified with bored piles. A coherent, consistent approach to the classification of bored displacement piles along with their design and specification is therefore required.

3 APPLICATION OF BORED DISPLACEMENT PILES

Recently in the UK several factors have combined to lead to the increased popularity of displacement piles. Key among these is the increased use of brownfield sites for developments. This is largely due to Government Planning Policy (Department of the Environment, Transport and the Regions, 2000), which restricts the sites available to developers and states that the Government is committed to maximising the re-use of previously developed land. A national target has been set that by 2008, 60% of additional housing should be provided on brownfield sites or through renovation. This is to promote regeneration and to limit the amount of greenfield land being used.

Redevelopment of brownfield sites presents two distinct problems with regards to foundations, namely disruption to neighbours (e.g. from noise and vibration) and the production of unwanted spoil. Often sites neighbouring brownfield developments are still occupied by functioning buildings, whether housing, commercial or industrial property. This often leads to constraints on the levels of permitted noise or ground borne vibration permitted during construction. This can be problematic, particularly for driven displacement piling methods.

The traditional non-displacement methods, however, produce unwanted spoil. Rising costs of haulage, landfill and the associated taxes make spoil disposal an unattractive option. Furthermore, brownfield sites often carry in their soil some form of contamination from their former use. Disposal of spoil arisings from replacement piles then becomes even more expensive and problematic.

Bored displacement piles can offer a solution which addresses these challenges. This and other advantages are summarised in Table 1. The disadvantages are also summarised.

The various bored displacement piling techniques are particularly suited to certain ground conditions because of the degree of displacement involved. Ideal conditions include sites with extensive made ground, alluvium, soft clays, loose sand or chalk. Bored displacement piles can be installed in stiffer and denser soils, depending upon the power available from the installation rig.

Table 1. Selection criteria for bored displacement piles.

Advantages	Disadvantages
The design of reinforcement is not controlled by the effects of handling or driving stresses	Potential disturbance of nearby existing structures due to ground heave
There can be an increase in the relative density of a granular founding stratum	Potential damage to nearby piles in which concrete has not cured sufficiently to resist changing soil stresses during construction
There is minimal spoil production	Concrete cannot be inspected after construction
Pile length can be easily varied to suit local conditions or different load capacities	Large diameters are not available
Installation causes low noise and vibration compared to driven displacement piles	

4 TYPES OF BORED DISPLACEMENT PILES

The classification of bored displacement piles is a surprisingly broad category containing a variety of proprietary pile types. By considering the historical development of bored displacement piles, most pile types can be described. A classification system is proposed to allow comparison between pile types and discussion of their relative merits. The classification system aims to avoid misleading or confusing terminology and the use of proprietary names for systems.

Legrand (2001) describes how the early development of bored displacement piles was brought about principally by technological advances in hydraulic drive systems for piling equipment. Hydraulic systems were developed which could provide sufficient torque and vertical force simultaneously to drive and bore a pile. Gradual refinement of the techniques subsequently followed and a number of bored displacement piling systems have evolved.

Van Impe (2001) identified three stages in the development of modern bored displacement piling methods and describes these as three generations. In the first generation, large diameter tubing with a sacrificial tip is inserted into the ground using a combination of rotary torque and downward force to displace the soil and then withdrawn to permit concreting. Continuous flight auger (CFA) piles can be considered to be a variation of this pile type, a small amount of soil displacement is caused by insertion of the auger. Large amounts of spoil are produced by such piles. The removal of spoil from the bore can negate the improvement to engineering properties of the soil caused by the displacement process. A refinement which causes greater displacement and produces less spoil is to utilise a large central stem with relatively small flighting. The flight then has the sole purpose of converting torque to draw the stem into the ground and displace soil.

The second generation of bored displacement piles includes the addition of a short flight to aid penetration as shown in Fig. 2. This development reduces drag forces by eliminating the continuous sections of flight previously described. The flight displaces soil as the auger is screwed into the ground, in addition to the central stem section.

Both this second generation of auger and the earlier CFA (including large stem) can be used in two different ways to provide differently shaped piles with diverse properties and advantages. The simplest method is to screw the pile into the ground and then extract it without rotation (or with continuing forward rotation). Such an approach produces a larger diameter cylindrical pile shaft. Considerable spoil is produced, although the lateral displacement will reduce this slightly.

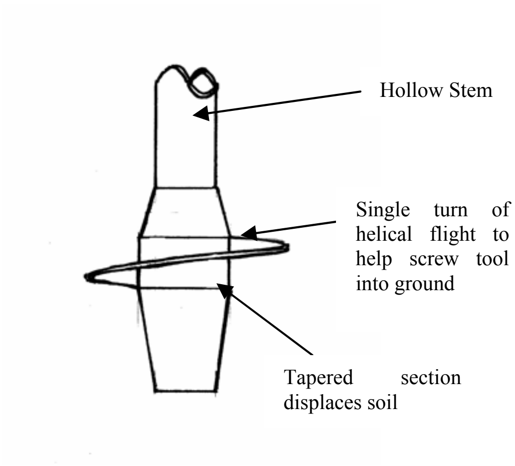


Figure 2. Second Generation bored displacement pile auger.

The alternative is to reverse the rotation as the auger is withdrawn in order that it retraces its path. This produces an irregular shaped pile, with a cylindrical central stem and a continuous helical flange. Cross sections of the two piles are shown in Fig. 3.

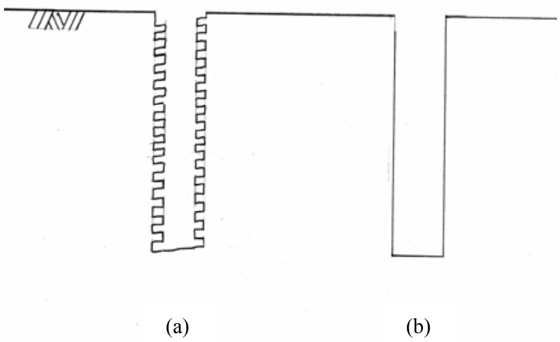


Figure 3. Pile cross sections. a) heliform, b) cylindrical.

The third generation of bored displacement pile was to provide full displacement to the outside diameter of the flights, producing a cylindrical pile. Efficient flight design pulls the auger into the ground; a tapering stem displaces the soil to the maximum diameter. The final section has a left handed flight. This allows the bore to be maintained in loose ground as the auger is extracted. The pile is concreted on withdrawal through the hollow stem. Fig. 4 shows a typical design of such a bored displacement auger.

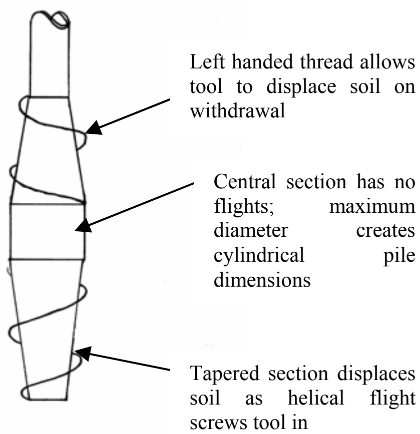


Figure 4. Third generation bored displacement pile auger, to form high displacement, minimal spoil, cylindrical piles

5 MATERIALS AND CONSTRUCTION

Bored displacement piles are cast *in-situ* by pumping concrete through the hollow stem of the auger, much in the same manner as a CFA pile. Concrete mixes need to be of a consistency suitable for pumping. Typically, in the authors' experience, concrete with a target slump of up to 150mm is used. Concrete cube strengths of up to $40kN/m^2$ are used.

Reinforcement is plunged into the concrete following withdrawal of the auger. Typically a 6 or 8 bar cage is inserted. Shear reinforcement is provided in the form of rings or helical links. For the heliform piles, the size of the inner diameter may restrict reinforcement to single central bars.

6 BORED DISPLACEMENT PILE CLASSIFICATION SYSTEM

The development of bored displacement piles, particularly in the UK, has been largely contractor driven. This has led to use of conflicting and confusing nomenclature and association of systems with 'trademark' names. A classification system is required which avoids all such association with proprietary systems and which gives clear description of the piles properties and expected behaviour of the pile and of the soil.

It is considered that the first part of the classification should indicate the degree of soil displacement undergone during the construction. This is consistent with the classification of driven displacement piles and is important in understanding the behaviour of a pile and the state of the soil. A pile formed by inserting and back screwing an auger such as that shown in Fig. 2 would be a low displacement pile, the displacement being limited to the diameter of the stem. A cylindrical pile formed entirely by displacement would be high displacement.

Further classification is provided by a description of the amount of spoil produced by a pile. This will also provide information related to the behaviour of the pile. A third generation pile which fully displaces soil to form a cylindrical pile will perform differently to a cylindrical pile formed by flighting and extracting material using a second generation type. Although similar to the previous categorisation, this classification provides information relating to the proportion of soil displaced to that removed from the bore. It is also particularly useful from a Client, or end-user, point of view as it provides clear information on the tangible effects of the process.

Behaviour and capacity of the pile is strongly influenced by the final shape of the pile formed. The second generation piles can produce heliform piles by back screwing on extraction such that the auger follows the same path as on insertion. Alternatively the same augers can be used to provide a cylindrical pile; this will produce differences in load capacity, permissible internal stresses and settlement characteristics. The classification also allows the pile volume to be deduced form the auger dimensions. This simple classification system is summarised by Fig. 5. Further explanatory notes can then be added to a pile description, for example ‘with enlarged toe’.

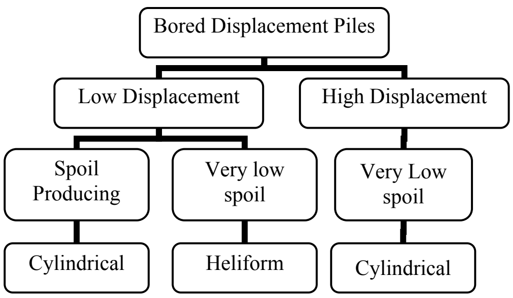
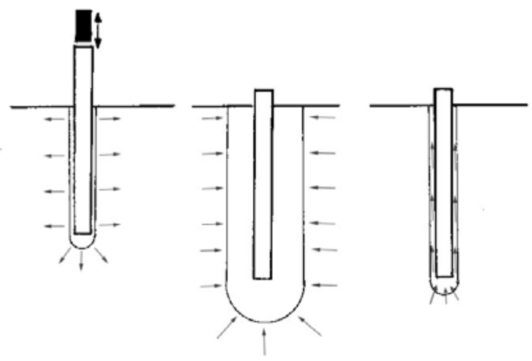


Figure 5. Bored Displacement Pile Classification

7 MECHANISM OF DISPLACEMENT

In order to understand the load carrying behaviour of a pile constructed using bored displacement techniques, it is essential to understand the mechanism of the displacement of the soil during the construction process, its effects and changes in the behaviour of the soil.

Randolph (2003) describes three processes which affect a driven displacement pile’s behaviour. Namely these are the installation, equilibration and loading of the pile. Randolph considered driven piles in both clay and sand. These processes are shown in Fig. 6 and described below, with reference to the bored displacement approach.



(a) (b) (c)

Figure 6. Three main phases during history of a driven pile: (a) installation; (b) equilibration; (c) loading. (After Randolph, 2003)

During installation, soil adjacent to the pile will undergo distortion and remoulding. Residual shear planes will be formed by the disturbance. Soil beyond this remoulded zone will be displaced and subject to a strain field resembling cylindrical cavity expansion.

For a modern bored displacement pile it is necessary to consider the installation phase as three discrete stages. The first stage is the insertion of the auger during which it may be reasonably assumed that the degree and extent of remoulding is significant due to the churning action of the auger. This is followed by a stress relief after the displacing auger head has passed. Finally, the auger head will pass a second time upon extraction to be followed by an injection of concrete, as shown in Fig. 7. Piles which are typically of 300 to 600mm diameter are constructed in this manner. The stem diameter is of the order of 50% of the maximum displacement diameter.

Insertion of the auger causes lateral displacement; the effects of the displacement will resemble those of driving a pile. It should be noted, however, that a driven pile may be affected by the cyclical nature of the hammer driving the pile. This is not the case for bored displacement. A driven pile will also cause a skin of soil to be carried down with the pile as it is driven. For a bored displacement pile the screwing action pushes the soil aside and none is carried down. For penetrations of up to ten times the diameter there will be some surface heave, beyond this the void is formed by outward radial displacement of the soil (Cooke & Price, 1973). Hollingsworth and Imbo-Burg (1992) agree that the main displacement is horizontal with static compaction and minimal disturbance to the soil structure. For cohesive soils which are either saturated or part saturated there will be time effects associated with the generation and subsequent dissipation of excess pore water pressures.

Next there is a relaxation stage once the widest part of the auger has passed. This relaxation period does not occur for a driven displacement pile, and its effects on the soil behaviour provide further argument for separate consideration and classification of bored displacement piles. Monitoring and control of the extent of this relaxation would be very complex and difficult. The final pile/soil behaviour may however depend in part on any changes undergone during this stage e.g. any loosening or expansion that occurs or development of negative pore water pressures. The time allowed for relaxation should therefore be carefully controlled, for a given pile length, to provide the maximum repeatability and minimum variability in pile behaviour. Time to construct a pile is therefore important to quality control.

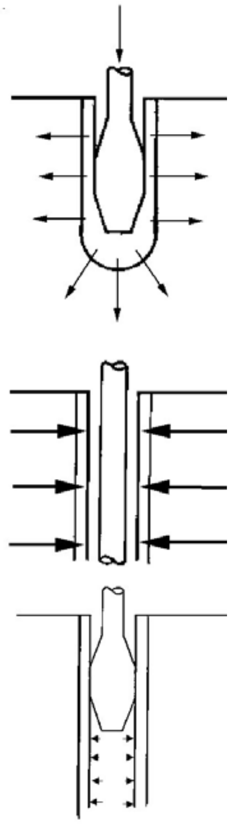
The tool is then withdrawn. Where relaxation towards the hole has occurred the second pass of the auger causes the re-imposition of stresses. The injection of concrete through the base of the auger provides sufficient pressure to maintain the bore and causes stresses to remain in the soil, as described for a CFA pile by Hollingsworth and Imbo-Burg (1992).

The equilibration phase will then begin as excess pore water pressures equalise. This will be rapid for a coarse-grained soil. In a fine-grained soil the soil may undergo consolidation. The timescale for this equilibration will be dependant upon the pile diameter and the coefficient of consolidation; this includes a component of the horizontal permeability of the soil. In the case of a heliform pile there may be further effects from vertical equilibration between the bands of relatively high and low disturbance caused by the alternating diameter of the pile. Equilibration is further complicated for the bored displacement pile by the fact that during this phase the concrete is curing and undergoing constant chemical change, competing for free water.

The final stage described by Randolph (2003) is the loading of the pile whereby the applied load is taken up along the shaft and by the pile toe. The interface of a bored displacement pile can be expected to be different from that of a driven displacement pile and may thus behave differently.

8 CHANGES IN STRESS DUE TO INSTALLATION

Key to understanding the pile behaviour is an understanding of the stress changes undergone by the surrounding soil. Little work has been carried out to measure the changes in stress during the installation of bored displacement piles. However, Lee *et al*. (2004) have studied sand compaction piles in soft clays and similarities can be drawn between these and displacement piles. A sand compaction pile (SCP) is installed using a steel casing through which, instead of injecting concrete, sand is placed and compacted by repeated withdrawal and driving of the casing.



a) Insertion of the displacement auger causes lateral movement of soils and an increase in pore water pressures in fine-grained soils

b) Once the tool has passed the soil is free to relax back towards the centre of the hole. This is necessary to reduce

c) Pressures are restored by the auger during extraction and the concrete injection maintains the integrity of the bore

Figure 7. Installation process of a bored displacement pile

During installation, horizontal stresses were shown to reach a peak as the tip of the tool passes. This reduced as the tip progressed deeper; this has also been described for driven piles by Bond and Jardine (1995). As the tool was withdrawn and the sand placed, the stresses returned to approximately the previous peak value and remained there after the withdrawal of the tool. Fig. 8 shows the stresses measured at a distance of between 1 and 2 pile diameters from the centre of the SCP installation and it can be seen that the increase in horizontal stress was of the order of 30%. The stress changes reduce with distance from the pile such that beyond 3.6 times the pile diameter from the pile centre there is little appreciable effect. In the figure the lines 1 to 4 represent the following four events: 1 the end of the tube passes the sensor, 2 the tube reaches full depth, 3 the end of tube passes on extraction and 4 the pile is completed.

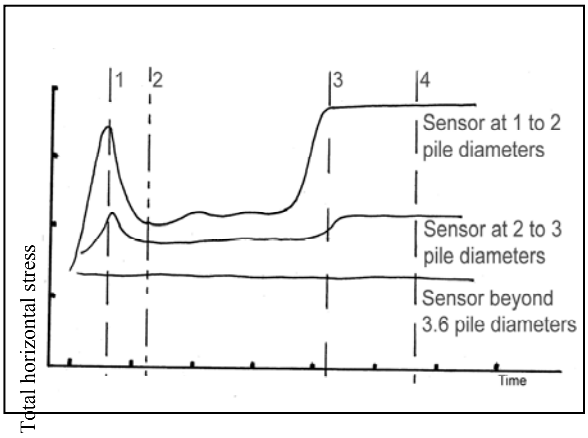


Figure 8. Measured total horizontal stress during construction of model SCP piles (After Lee *et al.*, 2004).

9 BORED DISPLACEMENT PILE DESIGN

Four alternative approaches to pile design are frequently adopted dependent upon soil type and the site investigation data available. These are empirical correlation with *in-situ* tests, a total stress approach, an effective stress approach and application of energy methods. These are all widely understood and accepted as valid design approaches.

Bored displacement piles are often designed for both fine and coarse-grained soil by empirical correlation with *in-situ* tests in mainland Europe, where use of such site investigation techniques is widespread e.g. cone penetrometer. In the UK, however, site investigation is often more suited to other methods. Effective stress methods provide a reasonable, conservative design (Burland & Twine, 1988), although there can be difficulties though in determining horizontal stress accurately. The compaction of the soil due to the displacement can be dealt with through selection of enhanced values for the coefficient of earth pressure and the angle of friction between the soil and the pile.

Energy methods for pile design are well documented and readily accepted for driven piles as an alternative design method.

Van Impe (1998) and Mure *et al.* (2003) have developed measures for the specific energies expended in creating augered piles. Van Impe's formulae are not designed for the assessment of pile capacity. They are intended to provide a measure of the efficiency of a piling technique and allow comparison of different bored displacement piling installation systems.

Mure *et al*. (2003) have sought to relate the energy to load capacity for continuous flight auger (CFA) piles but to date have not been successful in developing an accurate repeatable relationship. Its use is limited to comparisons across a site to identify anomalies or provide additional quality control. Key data captured and used in the analysis includes rate of penetration, torque and downward forces applied, speed of rotation, concreting pressures and flow rates and speed of extraction.

The simplest design method for fine-grained soil, based on current UK practice would be a total stress approach. Pile capacity is related to the pile dimensions and the undrained shear strength of the soil through an empirical factor, alpha. The factor represents the degree of softening undergone by the soil during the construction process. Bored pile design often relies on the values for alpha found by Skempton (1959). Bored displacement piles would achieve different values for alpha due to the changes undergone by the soil during the installation. A programme of test pile analysis could yield values applicable to design. It is suggested that the values would be greater than those found for bored piles due to the improvement to the soil caused by the displacement.

10 INSTALLATION OF BORED DISPLACEMENT PILES

This piling technique is best suited to fill, soft fine-grained soils, and loose to medium dense coarse-grained soils. Installation requires greater torque and downward vertical forces, than comparable CFA or rotary bored piles. This is reflected in the selection of rigs used by contractors.

It is at the installation stage however, that many of the benefits of this type of pile are realised. The lack of spoil arisings provides the Client with a cost benefit through avoidance of haulage, disposal and taxes. Further benefit is gained through the speed of set-up and lack of requirement to clear spoil after pile formation. Higher production rates are therefore common. Rates of up to 300 linear metres per day, and large numbers of piles being completed in a shift (50 piles per day), have been claimed.

If granular materials are encountered, wear of the auger can become a significant issue. Frequent maintenance may be required to maintain the auger surfaces and the overall diameter of the pile. This inevitably adds to overall production costs.

11 DISCUSSION

The UK has witnessed an increased call for bored displacement piles, in order to meet this efficiently and effectively, a better understanding of the piles is required.

There are many variations on the theme of bored displacement piles. The most common types have been described and a classification system has been proposed. This classification system is designed to be easily understood, not only by piling specialists but also by Clients.

When selecting a pile from this classification a number of factors must be considered. The type of soil, its stress state, density or strength will be influential on the decision, but assuming that the site suits the method which approach offers the most efficient pile? The amount of displacement caused by the pile may be influential. In dense or stiff soils, the energy required to install high displacement pile may prove prohibitive. The next choice relates to both the amount of spoil produced and the shape of the pile. A trade-off between these two factors is required. A cylindrical pile will offer higher load capacities but will produce more spoil, thereby incurring higher costs.

By either removing some spoil or by using the high displacement, low-spoil cylindrical bored displacement piling techniques, a cylindrical pile can be formed. These are advantageous due to their higher load carrying capacity. The capacity of heliform piles is limited because of their lower central stem diameter. The outer diameter cannot be used for pile capacity calculations, and if some composite diameter is to be used such as that suggested by Frangoulides (2000) then checks must be made on the shear capacity of the helical flange (which is not reinforced). There are also limitations on the amount of reinforcement that can be installed due to the lower central stem diameter. Internal concrete stresses may also be problematic with the heliform piles as the loads must be carried on the internal diameter.

Low displacement, low spoil, heliform piles can provide quick construction and possibly be suited to a wide range of sites. A high displacement, low or high spoil, cylindrical bored displacement pile, however, offers higher pile capacity and better pile integrity. The low spoil version being preferable, for reasons described.

Design of such piles is most simply carried out by empirical methods, whether relating to *in-situ* tests or effective stress. The choice between these is dependent upon the type of site investigation data available. Using existing empirical values is thought to provide a conservative estimate of pile capacity. The compaction or consolidation processes undergone during the pile installation will lead to better performance and values currently used for design may then prove overly conservative.

12 CONCLUSIONS

Bored displacement pile types are numerous and varied. Products and processes are often labelled inconsistently or confusingly and subject to proprietary systems. A descriptive classification system is proposed for discussion of the different types of pile.

Design of bored displacement piles, is allied to that of other pile types. The effect of the displacement on the soil should, however, be considered. Where empirical correlations are used in design, further work is required to determine the value of coefficients specific to the pile type and soil condition. Information is particularly lacking for the total stress approach often adopted within the UK for fine-grained soils.

The mechanism of the soil displacement is complex and involves increases and reductions in horizontal total stress; this is followed by consolidation as excess pore water pressures equalise. Mapping of the stress paths may provide an understanding of the changes undergone by the soil during piling, but this is complex.

Pile production can be quick, compared to other modes, and can offer substantial cost benefit through the avoidance of spoil production. This must be offset against the cost of increased power required for installation and maintenance required to maintain the displacement tools.

Cylindrical piles offer the highest pile capacities, and those formed with minimal spoil production offer best value to the Client. Heliform piles would only be advantageous where higher installation speeds are required and only relatively low capacities are desired.

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APPENDIX D PAPER 4

Full Reference

BAXTER, D.J., FLEMING, P.R., DIXON, N. AND CROMWELL, K.A., 2008 Bored Displacement Piles In Stiff Clay: construction and performance, *Proceedings of the 2nd BGA International Conference on Foundations,* Dundee, 24th -27th June.

Abstract

Use of bored displacement piles has undergone considerable growth in the UK piling market sector, largely due to their environmental advantages over other piling methods, and through cost benefits. This paper describes the formation of bored displacement piles and reports two case studies which represent the current state of the art in bored displacement piling in stiff clay (London Clay). Data collected from real-time monitoring of the construction of bored displacement piles and pile load test results are presented and discussed. There are currently few published case histories of this type of piling. The construction data and results of the testing programme add to and extend current knowledge of the behaviour and performance of bored displacement piles and provide a point of reference for comparison. Contrasts and comparisons are drawn with continuous flight auger piles in similar ground conditions, where production rates and capacities are seen to be similar; however larger energy input is required during construction.

Keywords

Bored displacement, Case Histories, Piles and Piling, Stiff Clay

1 INTRODUCTION

Use of bored displacement piles has experienced considerable growth in the UK piling market sector, largely due to their environmental advantages over other piling methods, and through cost benefits. The risk as well as the cost of handling, and disposing of, contaminated material is greatly reduced. Noise and vibration are also kept to a minimum, which makes them ideally suited to brownfield developments, particularly where contamination is present.

Traditionally, bored displacement piles have been utilised where the ground conditions permit displacement to occur relatively easily, either loose to medium dense coarse grained soils or soft to firm fine grained soils. The development of more efficient displacement augers and of purpose built bored displacement piling rigs have allowed the use of bored displacement piles to be extended to stiffer and denser soils. In this paper, case studies of the application of high displacement very low spoil cylindrical bored displacement piles in stiff to very stiff London Clay are presented and discussed. Contrasts and comparisons are drawn between bored displacement piles and continuous flight auger (CFA) piles.

The process of forming CFA piles is relatively well understood and has matured as a foundation construction process. Initial mistrust of the CFA process stemmed from the lack of knowledge of the process which was occurring in the ground and the difficulty in observing the bored shaft. Complex and detailed monitoring of the drilling and concreting parameters by rig mounted computers have now allayed many of these concerns. Bored displacement piles are still in their infancy compared to CFA piles. The processes undergone by the soils during displacement are not fully understood, nor are their effects on pile capacity. Bored displacement piles are grouped together with CFA piles[1] because of the similarities in the plant which is used and in the monitoring which is carried out during installation. This, however, can be misleading as the processes undergone by the soil are likely to be very different resulting in different pile behaviour.

There is little published case history data relating to bored displacement piling in the United Kingdom other than promotional material and magazine articles. The case studies presented in this paper form part of a wider research programme being carried out by Loughborough

University and Rock and Alluvium Ltd with the aim of furthering the knowledge of bored displacement piling and refining design and construction of such piles. The intention of this paper is to present case studies of bored displacement piling in stiff overconsolidated fissured clay (London Clay) which can be used for comparison, benchmarking and to promote discussion. Details of a similar CFA pile in comparable ground conditions are also reported to enable comparison.

2 BACKGROUND

2.1 BORED DISPLACEMENT PILING

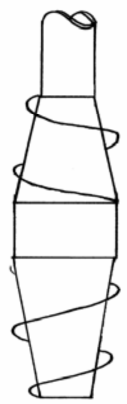
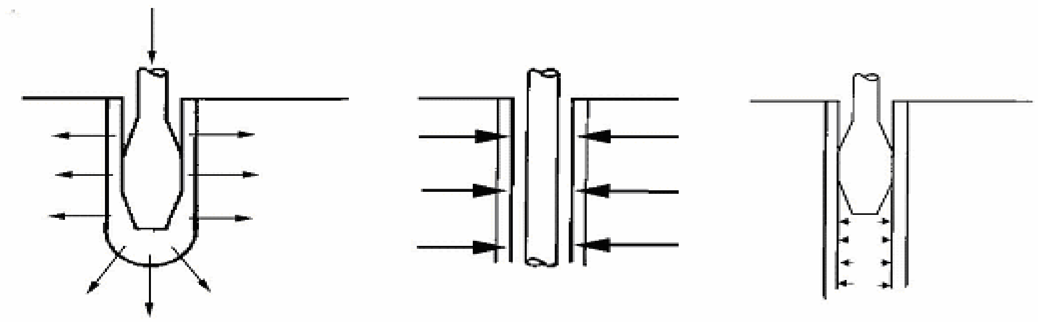


Fig. 1. A bored displacement auger for constructing high displacement cylindrical piles

A bored displacement pile is a type of piled foundation formed using a shaped auger to cause lateral displacement of the earth to form a void. A reinforced concrete pile is then formed in-situ in the void that has been formed. There are several alternative types of bored displacement pile, with varying degrees of displacement and different finished pile shapes; these have been classified by Baxter *et al*[2]. In this paper, high displacement, cylindrical bored displacement piles are considered and described. A diagrammatical representation of the auger is shown in fig. 1. This produces a cylindrical pile shape and not the screw shape associated with lower displacement piles. It is useful to consider the construction process as three stages which are described below and illustrated in fig. 2. These are the insertion of the auger, a stress relief following the passing of the auger head then the withdrawal/concreting phase.

During the first phase, efficient flight design on the leading part of the auger helps to pull the tool into the ground while the tapered stem displaces the soil laterally to the maximum auger diameter. Hollingsworth and Imbo-Burg[3] have described the main displacement as horizontal with static compaction and minimal disturbance to the soil structure. In this paper the construction of bored displacement piles in saturated stiff clay is described, there will be greater disturbance due to the incompressible nature of the clay. There can be no immediate volume change; vertical displacement and heave would then be inevitable. This heave may not be visible at ground level if there are compressible strata above the clay or if the clay is fissured. Once the auger head has passed, the narrower stem allows the soil to relax back towards the hole. This is necessary to reduce the friction acting on the tool, but complicates the stress history of the soil.

When the design depth has been reached the withdrawal and concreting phase can begin. The auger can be removed with or without rotation. When withdrawal is conducted with rotation, an upper, left-handed flight causes the displacement of any soil which has fallen in or relaxed towards the hole during the construction. This is particularly useful for loose materials. During withdrawal, concrete is pumped through the hollow stem of the auger at a pressure sufficient to maintain the bore dimensions.



a) Insertion of the displacement auger causes mostly lateral movement of soil	b) Once the tool has passed, the soil is free to relax back towards the centre of the hole	c) Pressures are restored by the auger during extraction and the concrete injection maintains the integrity of the bore.

Fig. 2. Installation process of a bored displacement pile

3 GROUND CONDITIONS

Two sites have been selected for examination, both are sites where the ground conditions are predominantly London Clay and where high displacement, very low spoil, cylindrical piles have been installed and tested. The sites are typical of those which might be encountered in the London Basin, in each case bored displacement techniques were used to construct the test piles described in this paper and working piles to act as load bearing foundations for the relevant developments. The same plant was used to construct piles at both sites.

London Clay covers a large geographical area which has a long and well documented history of construction development. There is therefore an existing associated body of knowledge upon which the current research can build and extend. This history of construction can be expected to continue as the area underlain by London Clay is still considered prime for development; the research findings should then be applicable to future construction works.

London Clay is an overconsolidated, stiff, fissured clay. It is considered to be reasonably uniform and its properties have been extensively investigated. It is a blue or grey clay which weathers to brown and contains fossils resembling modern warm-water forms and can contain bands of concretions as well as pyrite or selenite crystals[4]

3.1 PROJECT 1 - WIMBLEDON

The first site is situated in Wimbledon. The soil typically consists of approximately 1.5 m of loose granular fill material overlying 1.5 m of alluvial deposits, largely reported as silty sandy clay. Below 3 m the site is London Clay to unproven depth. The clay is weathered to approximately 5 m.

Seven investigative boreholes were carried out at this site for the purpose of determining the characteristic geotechnical properties of the soils. *In-situ* SPT tests have been carried out and have been correlated to values for undrained shear strength using the relationship proposed by Stroud and Butler[5]; these are shown in fig 3. The depths are measured from ground level.

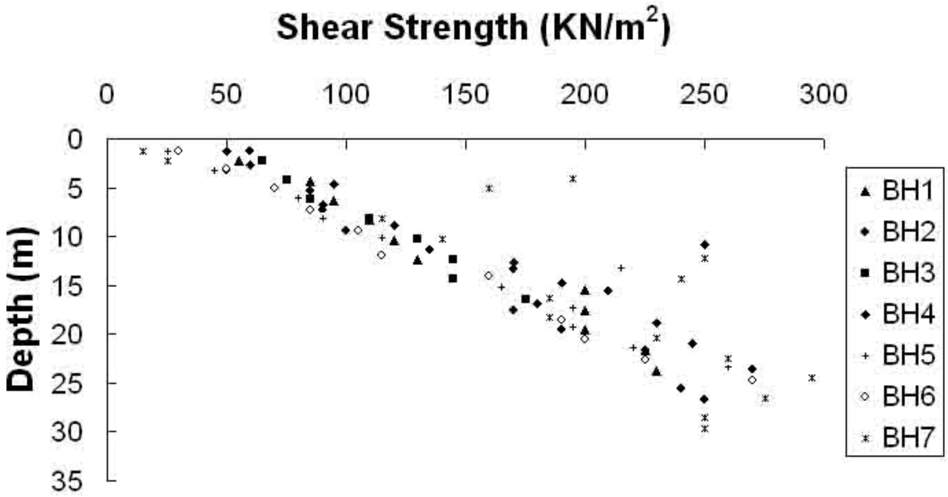


Fig 3. Undrained shear strength (calculated from SPTn using correlation by Stroud and Butler[5]) against depth below ground level for site at Wimbledon

3.2 PROJECT 2 - CHESSINGTON

The ground conditions at this site are largely similar to the Wimbledon study and generally consist of up to 0.7 m of fill, over London Clay. The clay is weathered to a depth of 6.5 m. There were 5 investigative boreholes at this site with *in-situ* SPT tests which have again been correlated to values for undrained shear strength [5]; these are shown in fig. 4. The depths are measured below a site datum which is equal to ground level at the highest point.

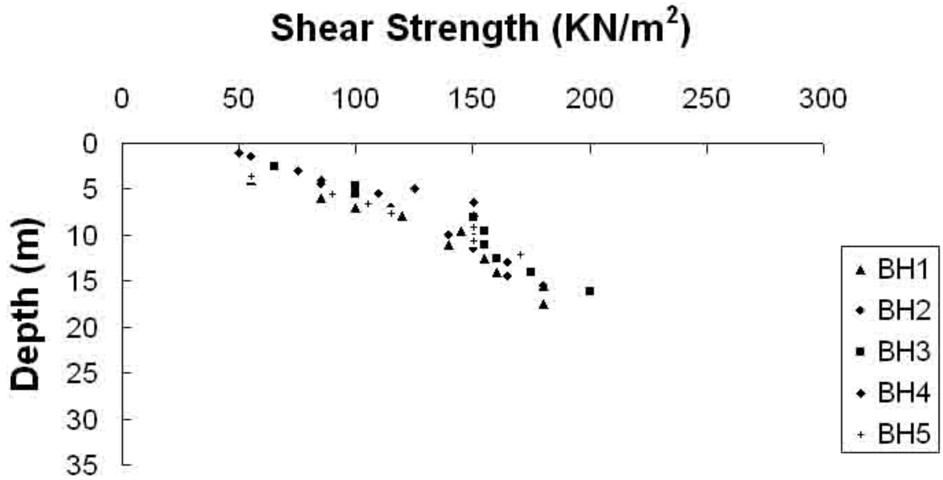


Fig 4. Undrained shear strength (calculated from SPTn using correlation by Stroud and Butler5) against depth below site datum for site at Chessington

4 PILE CONSTRUCTION

The works at Wimbledon consisted of approximately 1900 piles of 400 mm nominal diameter. Working loads on the piles varied widely from 200 kN to over 1000 kN with pile depths of over 18 m being required. This meant installing bored displacement piles into the very stiff London Clay. Production of the working piles was found to be quick and efficient, typically production rates of approximately 20 piles (350 linear metres) per rig day were achieved; this was similar to those which might have been expected for continuous flight auger piles in the same conditions. To enable comparison of pile types, a small number of CFA piles were also constructed at this site.

The works at Chessington consisted of approximately 350 bored displacement piles of 400 mm nominal diameter. Working loads on the piles ranged from 125 kN to 1000 kN with pile depths of over 16 m being required. Expendable test piles were constructed at both sites, using bored displacement techniques. Typical examples are reported below. The test piles in each case were 400 mm in diameter, at Wimbledon the test pile was 16 m in depth, at Chessington 16.2 m.

Figure 5 shows information recorded by the continuous rig monitoring during the auger insertion stage. For comparison, data from a CFA pile construction at the Wimbledon site has also been plotted on the same axes. The CFA pile was also 400 mm in diameter, and was 18 m in depth. The CFA pile was constructed with the same plant and equipment as the bored displacement piles, with the exception of the auger.

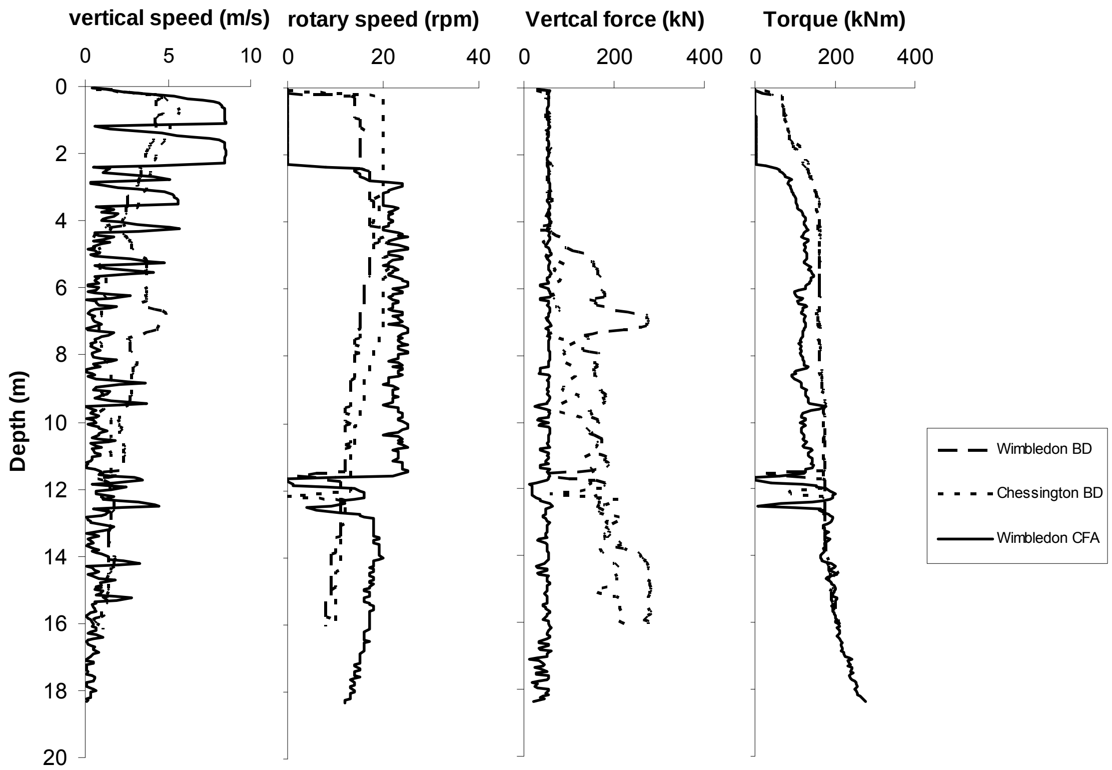


Fig 5. Piling rig logs from the drilling phase of the piles

4.1 ENERGY TO CONSTRUCT PILE

A measure of the energy used to install the pile can be obtained from the rig monitoring data using the formula proposed by Van Impe (2)[6] for the screwing in energy, E_s, shown in equation 1.

$$E_s = \frac{N_d v_i + n_i M_i}{\Omega v_i} \tag{1}$$

Where N_d is the vertical thrust of the auger, v_i is the vertical speed of the auger, n_i is the rotational speed of the auger, M_i is the torque applied and Ω is the area of the borehole. In each case the subscript "i" refers to the screwing in phase. Summary statistics of each parameter for each of the piles are listed in Table 1. An average screwing in energy is calculated from these mean values to facilitate comparison.

Table 1. Summary statistics of drilling phase parameters, Wimbledon site

Parameter	Bored Displacement		CFA
	Chessington	Wimbledon	Wimbledon
Mean Downward speed (mm/min)	1.81	2.39	1.11
Mean Torque(kNm)	161.2	169.2	153.9
Mean Rotary Speed (rpm)	15.82	13.07	19.35
Mean Pull down Force (kN)	111.3	173.2	49.3
Mean Screwing in energy (kJ/m^3)	1073	1501	748

5 PILE PERFORMANCE

Each of the test piles was subjected to a load test of the maintained load (ML) type, carried out in general accordance with the ICE Specification for Piling and Embedded Retaining Walls[1]. The load settlement behaviours are shown in fig 6.

Under application of the final load the Wimbledon bored displacement pile failed to attain the required settlement rates and the pile had therefore failed by this point. The measured capacity is therefore taken from the last point where the settlement requirements were met, 1618 kN. The actual capacity will lie somewhere between these last two points.

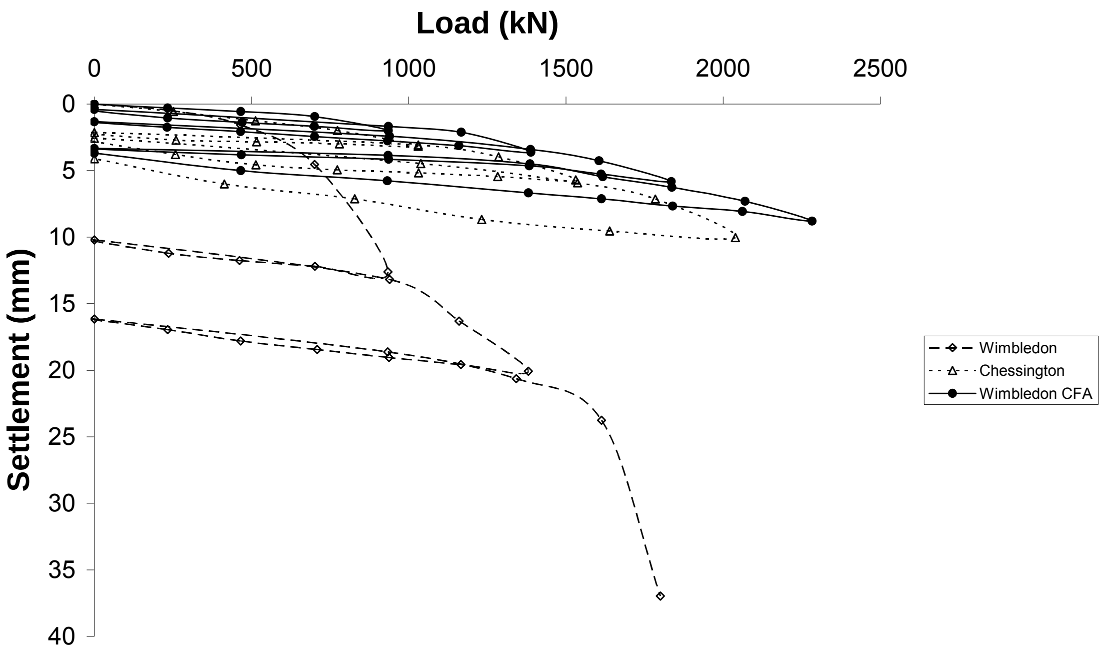


Fig 6. Load settlement behaviour of the bored displacement test piles and a comparable CFA pile

In the case of the Chessington bored displacement test pile, the maximum bearing resistance of the soil has not been exceeded; the test pile has therefore not reached geotechnical failure. The highest capacity demonstrated by this test pile is 2000 kN. Limitations on the allowable concrete stress and structural integrity of the pile precluded testing to higher loads, however, it was not anticipated that significantly higher capacity would be achieved. Using the extrapolation method proposed by Chin[7], the capacity was found to be approximately 2200 kN.

The CFA pile at the Wimbledon site described above was also subjected to a maintained load test. The results are shown overlain on fig 6. The capacity of this pile is demonstrated to be in excess of 2280 kN.

6 DISCUSSION

The two sites selected here are typical of sites in the areas of the London basin where the London Clay is present from near to the surface as there are limited superficial deposits. The Strength of the London Clay increases linearly with depth.

Although bored displacement piles are often categorised along with CFA piles, it must be considered that the pile is formed by very different processes. A CFA pile is formed by removal and replacement of material. The host material is cut and removed; there may be a net loosening of the soil. A bored displacement pile does not remove and replace soil but displaces it to form the pile. There will therefore be issues relating to the lateral displacement of the soil as well as any vertical displacement which occurs due to the incompressibility of the soil. Fissures may be closed up and the clay will doubtlessly be remoulded through the displacement process. The pore water pressure regime will also be affected. Heave may occur as a result of the displacement process and protection against this should be designed into the pile[1].

6.1 PILE ENERGY

The energy required to form a pile is indicated by Van Impe's screwing in energy. A CFA pile uses less energy during the construction of the void, which is to be concreted *in-situ* to form a pile. At the Wimbledon site the mean screwing in energy of a CFA pile is approximately half of that for a bored displacement pile. This is despite the fact that the CFA pile reported is longer by 2 m and therefore stiffer clays are encountered. This is a result of the energy required to displace the soil.

The energy used to construct the bored displacement piles reported in this paper differs between the piles at the two different sites. Pile construction is a subjective process, techniques employed by individual piling operatives may vary as they, and the plant, reacts to the particular ground conditions. In order to advance an auger, different combinations of vertical force, torque and rate of auger advancement can be employed and may differ between operators, rigs and auger designs and direct comparison is therefore difficult. Further work is required to ascertain the most effective and efficient combinations in different soil conditions. This would result in guidance for the best construction practice.

6.2 PILE LOGS

When analysing the logs of the pile construction there is a notable interruption to all of the parameters at about 11 to 11.5 m in all three cases. This is simply a function of the necessary pause in the drilling to allow for the engagement of an auger extension.

In both cases shown in this paper there is an increase in rotary pressure required with depth until the practical maximum operating limit is approached. At both sites this occurs at about 4 m, coinciding with the auger reaching clays which are stiff. The rotary speed remains roughly constant until the practical maximum rotary pressure is reached; once this has been achieved the rotary speed falls with depth (and increasing soil strength). In order to maintain progress of the auger into the ground it is necessary for the operator to engage the pull down winch. The pull down pressure required is constant until the maximum rotary pressure has been reached; beyond this point the downward pressure required begins to increase with depth.

The vertical (downward) speed of the bored displacement pile auger is, in both cases, higher than for the comparable CFA pile. This is a result of the need to pause to allow soil to travel up the flight for removal. The CFA pile log shows an erratic pattern which is symptomatic of this repeated slowing of the auger to allow the soil to travel up the flight. The vertical speed of the bored displacement piles reduces as the depth increases; this is a result of the increasing stiffness of clay which is encountered. The rotational speed is consistent for the bored displacement piles up until the point where the soil becomes sufficiently stiff that vertical force is required to advance the auger (and where the torque approaches its practical maximum limit). Rotational speed is sacrificed for the additional power required. The vertical force is essentially constant through the softer, upper material and is comparable to that used throughout the CFA pile construction. This is essentially the self weight of the auger.

The production rates of bored displacement piles and CFA are similar; downward speed is similar as is the time taken to concrete a pile. CFA piles can achieve faster rotation for a given rotary pressure as cutting the soil uses less energy than displacement in these ground conditions. The main observable difference is in the pull down pressures which are much lower for CFA. This is because the continuous helical shape of the auger transforms the torque into a downward force which effectively pulls the auger into the ground. Although the flight on the lead section of the displacement auger has a similar effect, it does not provide sufficient downward force to advance the displacement auger downwards and displace soil laterally. The additional energy required for the displacement comes from the application of the downward force.

6.3 PILE PERFORMANCE

The load tests reveal that the performance of the pile is comparable to that of a bored or CFA pile in similar ground conditions. Further information is required from more extensive field tests to allow general design approaches and empirical design parameters, such as the adhesion factor, α, to be defined. At present it is good practice to carry out preliminary test piles at each site for the purpose of determining design parameters for the bored displacement pile in the particular ground conditions, even in ground as well documented for other pile types as London Clay.

The settlements of the two piles shown in these case studies are not excessive but are not consistent with each other. Higher settlements for the Wimbledon bored displacement pile are not simply explained. Further investigation is necessary to determine whether this is a function of the way in which the pile was formed. There are also issues associated with the dissipation of the pore water pressures and how the pile capacity will vary with the age of the pile. Further testing and experience is necessary to allow serviceability limit state design based on the short and long term settlement characteristics of bored displacement piles.

The CFA pile load test results showed a higher capacity than for the bored displacement pile. This is not surprising as the pile is both longer and extends into stiffer material. Following testing, piles were exposed by careful excavation around the pile into the London Clay. The CFA pile demonstrated a distinct helical ridge which had been left by the auger head. The bored displacement piles had smoother shafts with no such regular ridging. The CFA pile may have benefited from this unintentional shear key which had been created as a by product of the pile formation.

When constructing bored displacement piles in ground conditions such as those described in this paper, it should be noted that ground heave can occur and has been observed. This is a result of the displacement of the (saturated) clay. Precautions should be taken to avoid damage to nearby structures and the new piles. Pile spacing and construction sequencing should also be carefully planned to avoid undue influence of piling operations on newly formed piles (particularly those where concrete has not yet set).

7 CONCLUSIONS

There are few case histories of bored displacement piling in the United Kingdom to date. The case histories presented in this paper are intended to provide a point of reference for comparison, discussion and future development of this variety of pile. Case histories of bored displacement piles in stiff fissured over-consolidated clay (London Clay) have been presented which demonstrate that this is a viable foundation solution. Bored displacement piles are often grouped with CFA piles because of the similarities in the plant used to construct them and the nature of the pile being cast *in*-situ with no observation of the bore being possible; it is important to observe that the behaviour of the piles will be very different as the processes undergone by the soil will be very different.

Bored displacement piles can be constructed in stiff clays at rates which are comparable to continuous flight auger piles but there are practical considerations and limitations which must be accounted for. Such practical solutions must include an assessment of the heave which is likely to occur, further investigation is required to determine the magnitude and effect of such heave, the area around the pile which is affected and the influence on neighbouring foundations as well as the effect on the pile itself. At the two sites used for the case studies, the minimum spacing between piles was limited to three times the pile diameter. Pile construction was sequenced such that adjacent piles were not installed until the earlier pile had developed a reasonable concrete strength. There were no observed ill effects from the displacement process on neighbouring piles.

The energy used to construct bored displacement piles is greater than that for CFA piles and the force required to displace the soil increases with the strength of the host soils. Despite the additional energy required, a bored displacement pile is an attractive environmental solution as there is minimal spoil to transport and dispose of and contamination is left *in-situ*. Bored displacement piles require application of a greater downward force to force the auger into the ground, the screwing action does not draw the auger down to the same extent as for a CFA pile. Further examination of the effect of the different combinations of speed and force applied to advance the auger could yield a definition of the best practice for efficiently or effectively producing a pile. This is a fundamental step in the understanding of the effect of the installation upon behaviour and capacity which in turn is critical in building confidence in the technique.

The design of bored displacement piles should currently be based on the results of site specific pile load testing. A greater knowledgebase is required to develop indicative empirical design parameters for bored displacement piles. Further case studies of pile performance in this, and other, soil types should be encouraged to permit contrasts and comparisons to be drawn and to draw attention to the key issues relating to the design and installation of such piles. Time related effects on the capacity of the pile need careful consideration and extended investigation.

The capacity of bored displacement piles is of a similar magnitude to that achieved by CFA piles. CFA piles may have better load carrying characteristics because of the rough interface created by the boring process. A bored displacement pile is less likely to have this rough profile as the repeated turning of the auger tends to leave a smoother face.

8 ACKNOWLEDGEMENTS

This research forms part of a research programme being undertaken by the Centre for Innovative and Collaborative Engineering (CICE) at Loughborough University and Rock and Alluvium Ltd and is supported by the Engineering and Physical Sciences Research Council.

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