15 Aug 2008, 1:30 pm - 3:00 pm

Using Experience and Case History Data to Enhance the Design of Piled Foundations and Predict Behaviour Characteristics

David J. Baxter
Loughborough University, United Kingdom

Neil Dixon
Loughborough University, United Kingdom

Paul R. Fleming
Loughborough University, United Kingdom

Ken A. Cromwell
Loughborough University, United Kingdom

6th International Conference on
Case Histories in Geotechnical Engineering, Arlington, VA, August 11-16, 2008

USING EXPERIENCE AND CASE HISTORY DATA TO ENHANCE THE DESIGN OF PILED FOUNDATIONS AND PREDICT BEHAVIOUR CHARACTERISTICS

David J. Baxter
Loughborough University
Loughborough,
Leicestershire,
United Kingdom

Neil Dixon
Loughborough University
Loughborough,
Leicestershire,
United Kingdom

Paul R. Fleming
Loughborough University
Loughborough,
Leicestershire,
United Kingdom

Ken A. Cromwell
Rock and Alluvium Ltd.,
Wallington,
Surrey,
United Kingdom

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.

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.

BACKGROUND

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.

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.

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.

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.

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.

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.

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.

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.

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

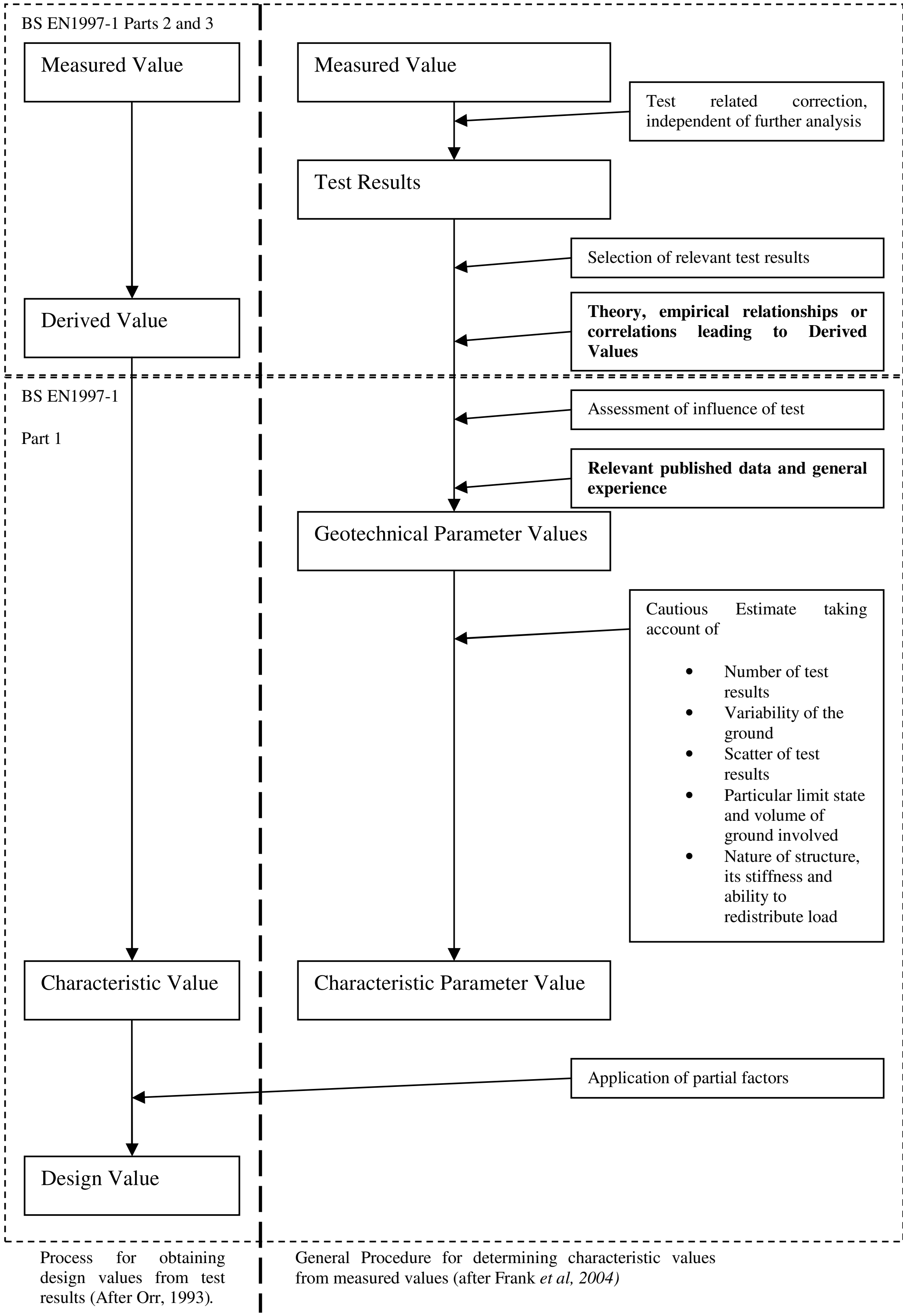

Fig. 1. Steps in determining characteristic and design values (After Orr (1993)and Frank et al. (2004))

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.

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

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 (6)

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.

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

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

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.

METHOD FOR INCORPORATING PREVIOUS DATA AND CASE STUDY INFORMATION

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.

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

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

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.

QUANTIFYING PREVOUS EXPEREINCE

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.

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.

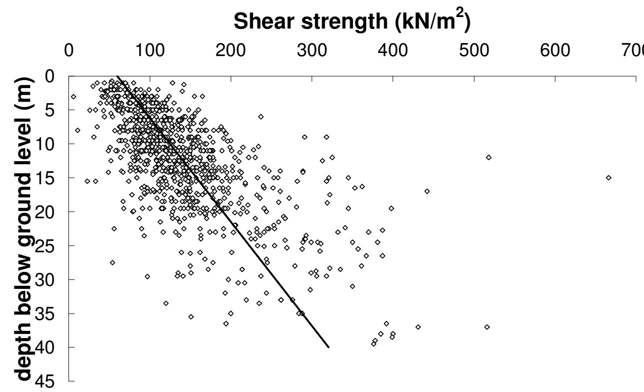

Fig. 2. Shear strength against depth for all sites in dataset, with trend line.

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.

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)

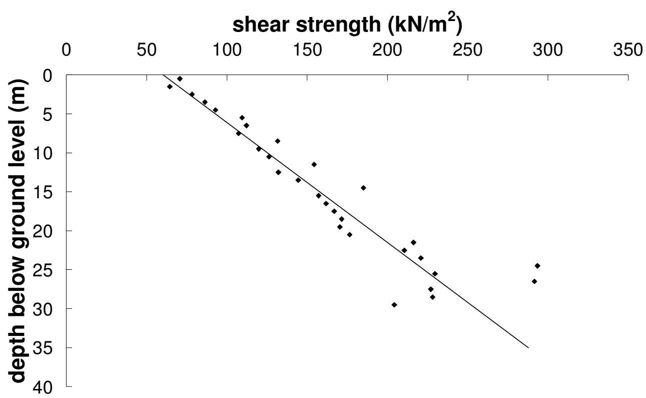

Fig 3. Mean values for shear strength grouped into 1 m thick depth bands, shown with the original regression line for all data points.

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.

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

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 0 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.

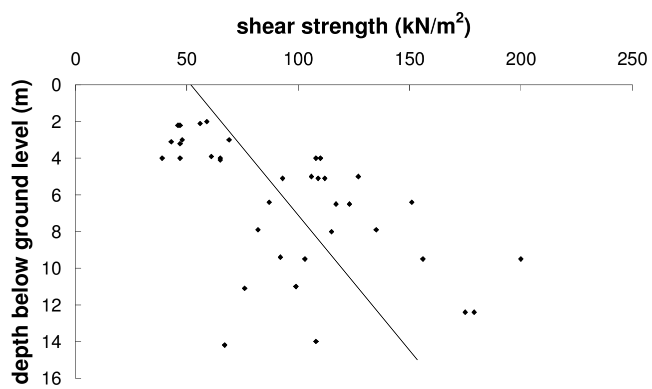

Fig 4. Example shear strength data, with linear regression trend line, from site investigation at a site near Woolwich.

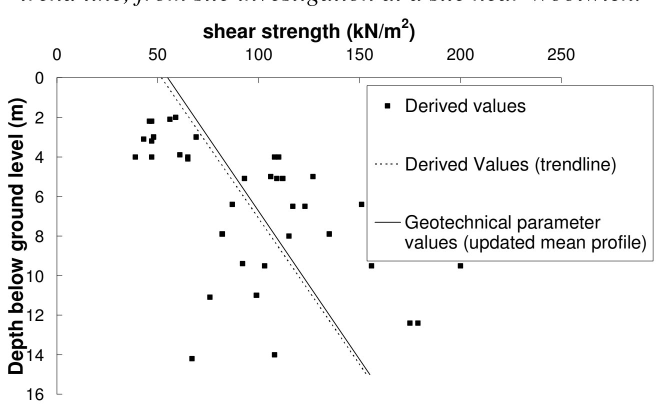

Fig 5. Bayesian updating of derived values of shear strength from the site near Woolwich, using the database as prior knowledge, to generate geotechnical parameter values

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.

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.

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.

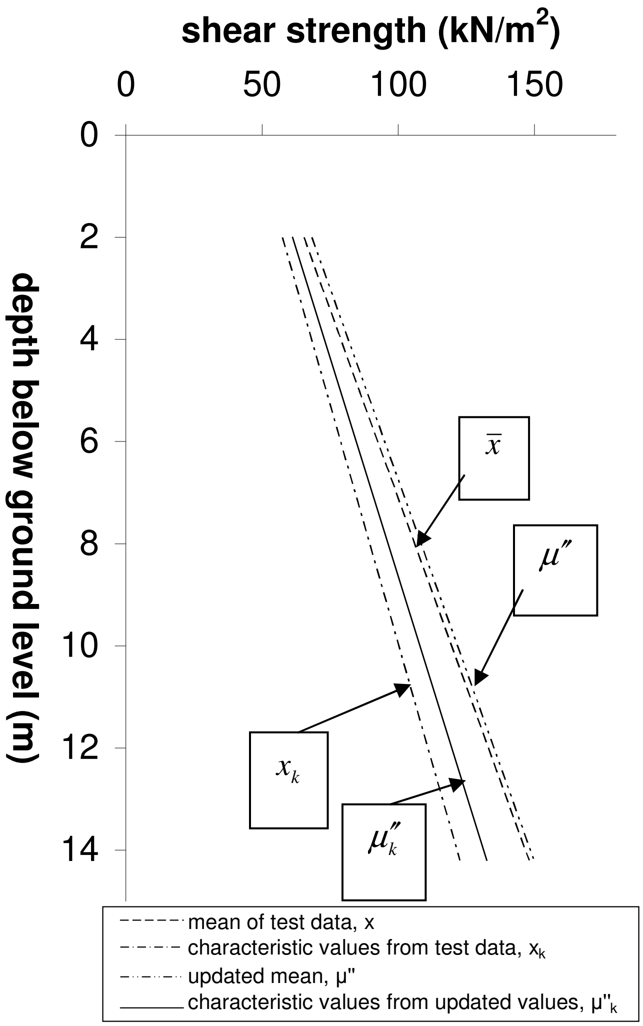

Fig 6. Determination of characteristic values by Schneider's approximation, Woolwich case study.

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).

Variable		**Values**	**F**	**ΔF**
Shear strength	MLV+σ	126	2.67	1.23
	MLV-σ	68	1.44	
α	MLV+σ	0.51	2.33	0.55
	MLV-σ	0.39	1.78	

Standard deviation of F,

$$\sigma_F = \sqrt{\left(\frac{\Delta F_1}{2}\right)^2 + \left(\frac{\Delta F_2}{2}\right)^2} = 0.67$$

Coefficient of variation of F, V_F=0.67/2.06=0.33

Probability of failure, $P_{failure}$=2.41% (from tables)

<u>Design using the data updated with case history information.</u> 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)

Variable		**Values**	**F**	**ΔF**
Shear strength	MLV+σ	124	2.45	0.83
	MLV-σ	82	1.62	
α	MLV+σ	0.51	2.31	0.54
	MLV-σ	0.39	1.77	

$$\sigma_F = \sqrt{(\frac{\Delta F_1}{2})^2 + (\frac{\Delta F_2}{2})^2} = 0.49$$

$V_F=0.49/2.04=0.24$

$P_{failure}=0.36\%$ (from tables)

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.

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

REFERENCES

British Standards Institute [2004]. *"BS-EN 1997-1:2004 Eurocode 7: Geotechnical Design, Part 1 Geotechnical Design."*. BSI. London.

Burland, J.B. and D. Twine [1988]. "The shaft friction of bored piles in terms of effective strength", *Proc. First International Geotechnical Seminar on Deep Foundations on Bored and Auger Piles*, Balkema, Rotterdam, pp411-420.

Baxter, D.J., N. Dixon, P.R. Fleming, K.A. Cromwell [2007]. "Refining characteristic values of shear strength using experience." Submitted to P. I. Civil Eng-Geotec.

Bustamante, M. and L. Gianeselli [1982]. "Pile bearing capacity prediction by means of static penetrometer CPT", *Proc. Second European Symposium on Penetration Testing,* Balkema, Rotterdam, pp493-500.

Dixon, N., W. Blumel, C. Stoewahse, P. Kamugisha and J.R.V. Jones [2002]. "Geosynthetic shear behaviour: Part 2 Characteristic values for use in design." Ground Engineering, (March 2002), pp. 49-53.

Duncan, J .M. [2000]. "Factors of safety and reliability in geotechnical engineering". J. Geotech. Geoenviron., Vol. 126, No. 4, pp. 307-316.

Fleming, W.G.K., A.J. Weltman, M.F. Randolph, and W.K. Elson [1992]. *"Piling Engineering*". 2nd Edition, Blackie Academic and Professional, Glasgow.

Frank, R., C. Bauduin, R. Driscoll, M. Kavvadas, N. Krebs Ovesen, T. Orr, and B. Schuppener [2004]. *"Designers' Guide to EN1997-1 Eurocode 7: Geotechnical design - General rules",* Thomas Telford, London.

Holeyman, A., C. Baudin, M. Bottiau, P. Debacker, F. De Cock, E. Dupont, J.L. Hilde, C. Legtrand, N Huybrechts, P Menge, J.P. Miller and G. Simon [2001]. "Design of axially loaded piles - 1997 Belgian Practice," in *Screw Piles - Installation and Design in Stiff Clays* (A. Holeyman ed.), Balkema, Rotterdam, pp63-88.

Hooper, J.A. and F.G. Butler [1966]. "Some numerical results concerning the shear strength of London Clay." Géotechnique, Vol. 16, No. 4, pp. 282-304.

Lumb, P. [1966]. "The variability of natural soils". Can. Geotech. J., Vol. 3, No. 2, pp. 74-97.

Orr, T.L.L. [1993]. "Probabilistic characterization of Irish till properties", in *Risk and reliability in ground engineering*, (B.O. Skipp ed.), Thomas Telford, London, pp12 6-133.

Patel, D.C. [1992]. "Interpretation of results of pile tests in London Clay", in *Piling; European practice and worldwide trends,*(M.J. Sands, ed.), Thomas Telford, London, pp100-110.

Phoon, K.K. and F.H. Kulhawy [1999]. "Characterization of geotechnical variability." Can. Geotech. J., Vol. 36, No. 4, pp. 612-624.

Schneider, H.R. [1997], "Definition and determination of characteristic soil properties", *Proc. 14th International Conference on Soil Mechanics and Foundation Engineering,* Balkema, Rotterdam, pp2271-2274.

Searle, I.W. [1979]. "The design of bored piles in overconsolidated clays using effective stress", in *Recent developments in the design and construction of piles,* ICE, London, pp355-3632.

Skempton, A.W., [1959]. "Cast in-situ bored piles in London Clay." Geotechnique, Vol. 9, No. 4, pp. 153-173.

Stroud, M.A. and F.G. Butler [1975] "The Standard Penetration Test and the Engineering Properties of Glacial Materials", *Symposium On Engineering Properties of Glacial Materials,* Midland Geotechnical Society, pp117-128.

Tang, W.H. [1971]." A Bayesian evaluation of information for foundation engineering design", *International Conference on Applications of Statistics and Probability to Soil and Structural Engineering*, Hong Kong, pp173-185.

Whitaker, T. and R.W. Cooke [1966]. "An investigation of the shaft and base resistances of large bored piles in London Clay", in *Large Bored Piles* (S.C. Packshaw and K.H. Brittain, eds), ICE, London, pp7-49.