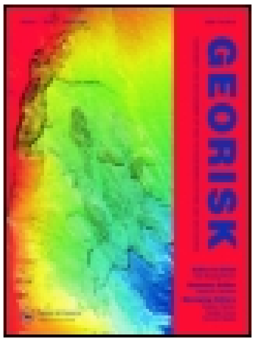

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ABSTRACT
The availability of reliable field data is critical for the advancement of geotechnical engineering. This is particularly the case for piled foundations; due to the substantial geotechnical uncertainties. The settlement (performance) predictions from established analytical methods may deviate from field measurements by as much as an order of magnitude. This paper provides a statistical assessment of the uncertainty of predictions of pile performance under axial loading using an openly accessible geotechnical database of pile load tests from the United Kingdom. The collected database information was classified by pile type, location, test data quality and availability of geotechnical data. With reference to the data from fine-grained soils, two analytical models were employed to predict foundation settlement. The settlement prediction performance was then studied statistically and the model bias and error compared with reference to the aforementioned categories to identify the impact of different sources of uncertainty and evaluate the use of both models for future geotechnical practice. The two models investigated generally over-predict settlement, which is likely due to conservative selection of key model parameters, such as soil strength.

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KEYWORDS
Piles & piling; full scale tests; site investigation; settlement; soil/structure interaction

1. Introduction

For axially loaded piles, failure is typically associated with excessive settlement. The available literature on theoretical simulations is vast and encompasses three families of models: approximate analytical formulations in one or multiple dimensions (Randolph and Wroth 1978; Baguelin and Frank 1980; Scott 1981; Mylonakis and Gazetas 1998; Mylonakis 2001; Guo 2012; Anoyatis 2013; Vardanega, Williamson, and Bolton 2012a; Vardanega et al. 2018; Crispin, Leahy, and Mylonakis 2018), elastic Green's functions/boundary-element type solutions (Butterfield and Banerjee 1971; Poulos and Davis 1980; Kaynia 1982; El-Marsafawi 1994), finite-element models (Ottaviani 1975; Baguelin and Frank 1980) and various empirical or semi-empirical schemes (Seed and Reese 1957; Coyle and Reese 1966; Kraft, Ray, and Kagawa 1981). Despite the variety and the sophistication of available models, engineers "*may never be able to estimate axial pile capacity in many soil types more accurately* [on the average] *than about ±30%*" (Randolph 2003, 848), due to inevitable aleatory uncertainty, which makes piling engineering a challenge even for geotechnical specialists. While the statement of Randolph (2003) refers to pile capacity, it is a salutatory reminder of the very real challenge in predicting to a high degree of accuracy the performance of piled foundations.

Reduction of uncertainty in pile settlement predictions is a fundamental geotechnical engineering need, and therefore, conducting full-scale field tests to establish performance is often necessary on large and/or complex projects. Although such tests are not uncommon in practice, they are expensive, time-consuming and in some cases difficult to carry out and interpret. This frequently leads to conservative design practice which, while acceptable from a safety perspective, does come at a lower value for money. To identify the causes of design conservatism, both system uncertainty and parameter uncertainty should be considered (Bolton 1981), which can be difficult when the availability of geotechnical data is low. This can be addressed by developing open databases of high-quality field data. The significance of such online tools has been recognised by various investigators from different countries who have compiled databases with pile data. These include databases from North America

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(Paikowsky et al. 2004), Western Europe (Galbraith, Farrell, and Byrne 2014), North Africa (AbdelSalam, Baligh, and El-Naggar 2015) and South-East Asia (Ong et al. 2021). Global databases have also been recently compiled for tests in sands by a Chinese-UK-Australian consortium (Yang et al. 2015, 2016) and the Deep Foundation Institute and UC-Irvine (Find A Pile.com, Lemnitzer and Favaretti 2013) for lateral-load tests. Phoon and Tang (2019a) have recently combined two databases to study capacity of static load tests on steel piles founded in various soil types. The joint industry research project Pile-Soil Analysis (PISA) has produced significant results and reports detailing premium quality site data (Burd et al. 2020a, 2020b; Byrne et al. 2017, 2020; Zdravković et al. 2020a, 2020b).

In the United Kingdom, a significant volume of piling is carried out on firm clayey sediments. Many results of full-scale pile tests have been reported for London Clay (e.g. Whitaker and Cooke 1966; Patel 1992), but the raw data is often not openly accessible and is held by various entities. Designers mostly rely on early empirical and analytical models from classic sources such as Skempton (1959), Tomlinson and Woodward (2015), Salgado (2008), and Fleming et al. (2009) to carry out routine design work. The parameters involved in these models are based mostly on judgment and often only a limited number of tests. As codes of practice become more performance oriented, high quality data are essential for calibrating design procedures. Where data is particularly powerful is in calibrating key model parameters e.g. "α-values" (see, for instance: Skempton 1959; Patel 1992; Salgado 2008).

Combining the data collected from load tests on firm clay sediments in the UK is essential for better calibration of geotechnical design models, reducing uncertainty and assessing the potential for foundation re-use by better estimating "reserve capacity" in existing construction. Such data is vital for validating (or falsifying) theoretical models. To this end:

- A large, openly accessible UK pile test database categorised according to pile type, location, test data quality and availability of geotechnical data was assembled.
- Two easy-to-implement analytical models for non-linear pile settlement by Vardanega, Williamson, and Bolton (2012a); updated in Vardanega et al. (2018) and Crispin, Leahy, and Mylonakis (2018) were employed to predict the settlement of piles embedded in fine-grained deposits.
- The settlement predictions of the two models were then compared with measured results from the database and the computed model bias and errors assessed for the aforementioned categories.
- Conclusions as to which parameter uncertainties have the most impact on settlement prediction performance are highlighted.

2. Database assembly

A considerable amount of geotechnical data has been obtained since the emergence of soil mechanics as a discipline during the early twentieth century. Much of this information is dispersed and not compiled in a usable manner, either published in a scattered array of conference proceedings, reports, books, dissertations and journals (or in stored box files in the offices of geotechnical engineering consultants). Compiling an opensource electronic database that is UK specific allows for detailed analysis of foundation performance informed by the considerable high-quality geotechnical characterisation data that has already been funded by significant research investments (e.g. Gasparre et al. 2007a, 2007b; Hight et al. 2007 and Kamal et al. 2014). High quality testing can be achieved, reported, and employed in analysis and design, as recently demonstrated by the PISA project publications (e.g. Burd et al. 2020a, 2020b; Byrne et al. 2017, 2020; Zdravković et al. 2020a, 2020b). Earlier efforts include the Imperial College Method for piles (Jardine et al. 2005, 2006). However, for standard construction works such data is rarely available.

The DINGO project (Vardanega et al. 2021a, 2021b) collected over 500 pile load tests from the UK into an open-source database. The industry data and the publications searched for data from the literature is available in the final project report by Vardanega et al. (2021b) which also includes full details about the building and presentation of the database.

The collected data is comprehensive including various kinds of pile tests (constant rate of penetration (CRP), maintained load (ML), Osterberg style), pile types (bored, driven, cast in situ, continuous flight auger (CFA), straight shafted, under-reamed) and pile materials (concrete, steel, mixed, composite, etc.), locations across the UK and soil types (clay, chalk, sand, superficials, etc.). Nevertheless, some information such as pile Young's modulus for bored piles, pile axial strains and near-pile excess pore water pressure with depth, and time lag between completion of construction and field testing could not be obtained.

A summary of the variety of types of data is shown in Figure 1, where the pile tests included in the DINGO Database are subcategorised by: "Geology", "Data source", "Pile Type" and "Era of construction" (epoch). No further subcategorisation has been attempted due to the amount of data available. A summary geology map of the DINGO database tests in fine-grained deposits is given in Figure 2. Figure 2 shows there is good geographical coverage of tests across the UK and a good spread of results in the London area. In this paper, the 186 tests conducted in fine-grained soils were analysed in detail.

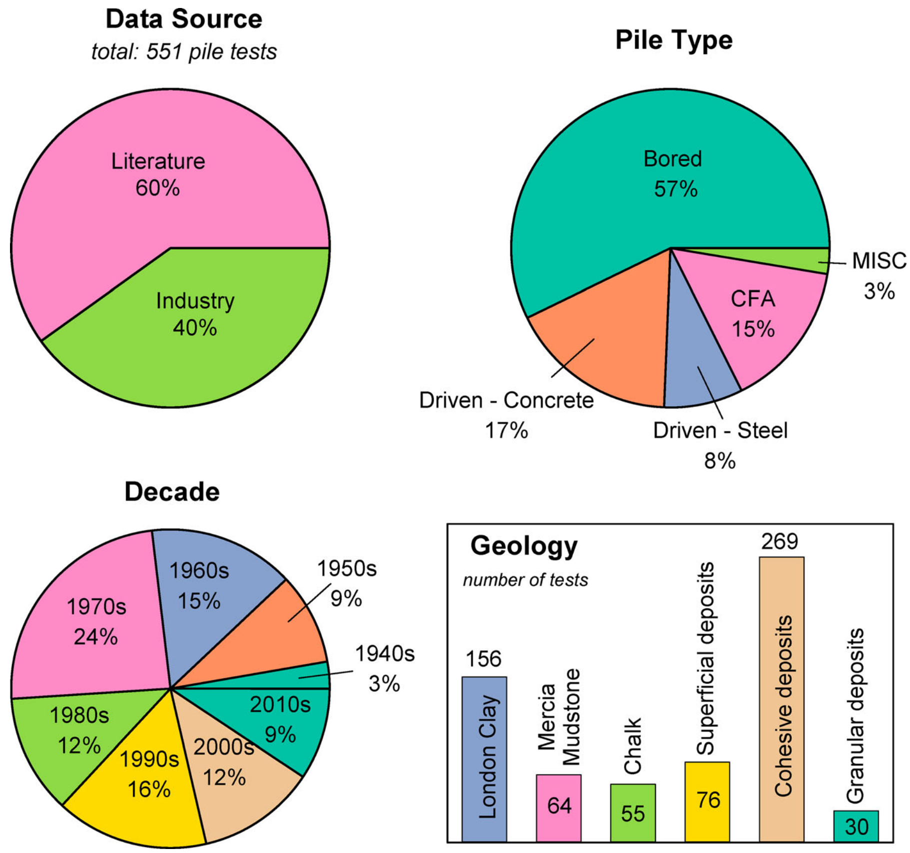

Figure 1. Distribution of the DINGO Database pile test in the main subcategories. (Some geology categories are overlapping, i.e. a single pile may be in multiple categories.)

3. Model presentation

Two analytical approaches for pile settlement are reviewed in this section. Both were chosen for their relative ease of use and ability of implementation with hand calculations or a spreadsheet. These closed-form solutions have been employed for simulating axial pile response and performing comparisons with the test data:

Model 1 is a practical closed-form solution suitable for hand calculation (Vardanega, Williamson, and Bolton 2012a; Vardanega et al. 2018), based on mobilised strength design (MSD) principles. Model 1 introduces a uniaxial power-law constitutive model for shear stress, a function of shear strain, that allows accounting for material non-linearity in the soil surrounding the pile, and pile compressibility, without sacrificing simplicity in application. The formulation does not, however, account for pile tip resistance and, therefore, gradually loses accuracy as the pile base engages. A summary of the formulation is provided in Figure 3.

Model 2 is an analytical closed-form elastoplastic solution encompassing depth-dependent soil stiffness and strength in the form of elastic-perfectly plastic "*t-z*" curves, an elastoplastic tip resistance described by a bilinear force-displacement, and soil yielding propagating unilaterally from the surface towards the base. Based on these assumptions, a closed-form solution for pile settlement is derived (Crispin, Leahy, and Mylonakis 2018). The method can easily handle layered profiles such as those encountered in the study at hand and is suitable for spreadsheet or hand calculation. A summary of the formulation is provided in Figure 4. Additional information on the models is provided in the original publications (Vardanega, Williamson, and Bolton 2012a; Vardanega et al. 2018; Crispin, Leahy, and Mylonakis 2018) and in the follow-up papers of: Vardanega (2015), Voyagaki et al. (2019) and Crispin, Vardanega, and Mylonakis (2019). A wider set of "t-z" curves is given in Bateman et al. (2021).

3.1 Factor of safety, F

To present the results in a rational manner an assessment of the factor of safety is needed. To this end, the settlement at the pile head w_o was be normalised with the shaft diameter D_s and the applied head force P expressed as a fraction of the nominal bearing capacity P_u for undrained conditions, which, for compression piles, can be estimated using Equation (1):

$$P_u = P_{u,s} + P_{u,b} = \alpha\, \bar{c}_u\, \pi\, D_s\, L + N_c\, c_{ub}\, \pi\, D_b^2/\,4 \quad (1)$$

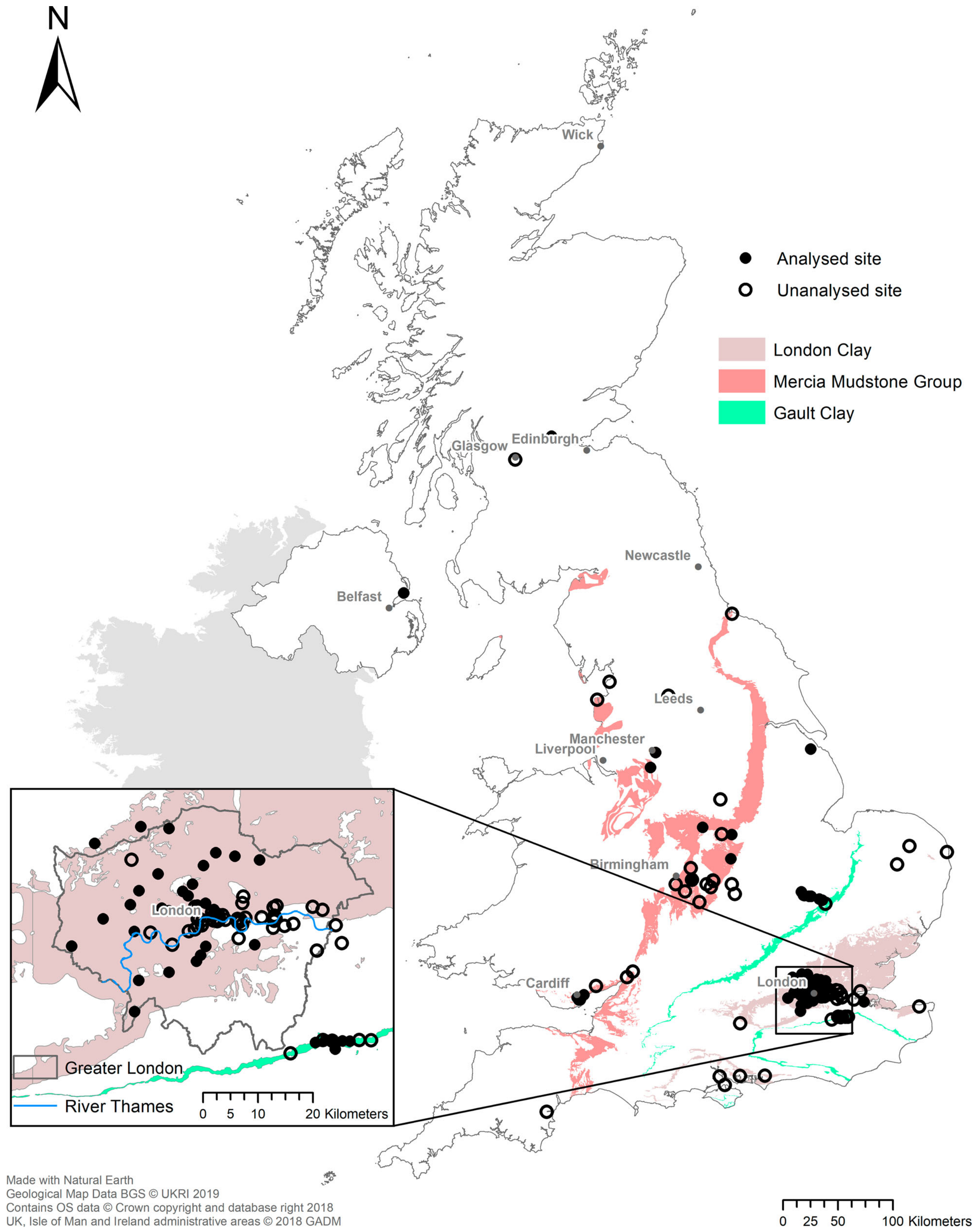

Figure 2. GIS geology map showing the geographic distribution of the DINGO Database tests for fine-grained deposits.

where $P_{u,s}$ and $P_{u,b}$ are the ultimate shaft and base resistances, respectively. $\bar{c}_u$ is the mean undrained shear strength over the pile length, c_{ub} is the corresponding value at the tip, D_s is the pile shaft diameter, D_b is the pile base diameter, L the pile length, and N_c is the bearing capacity factor for undrained conditions in clay. By

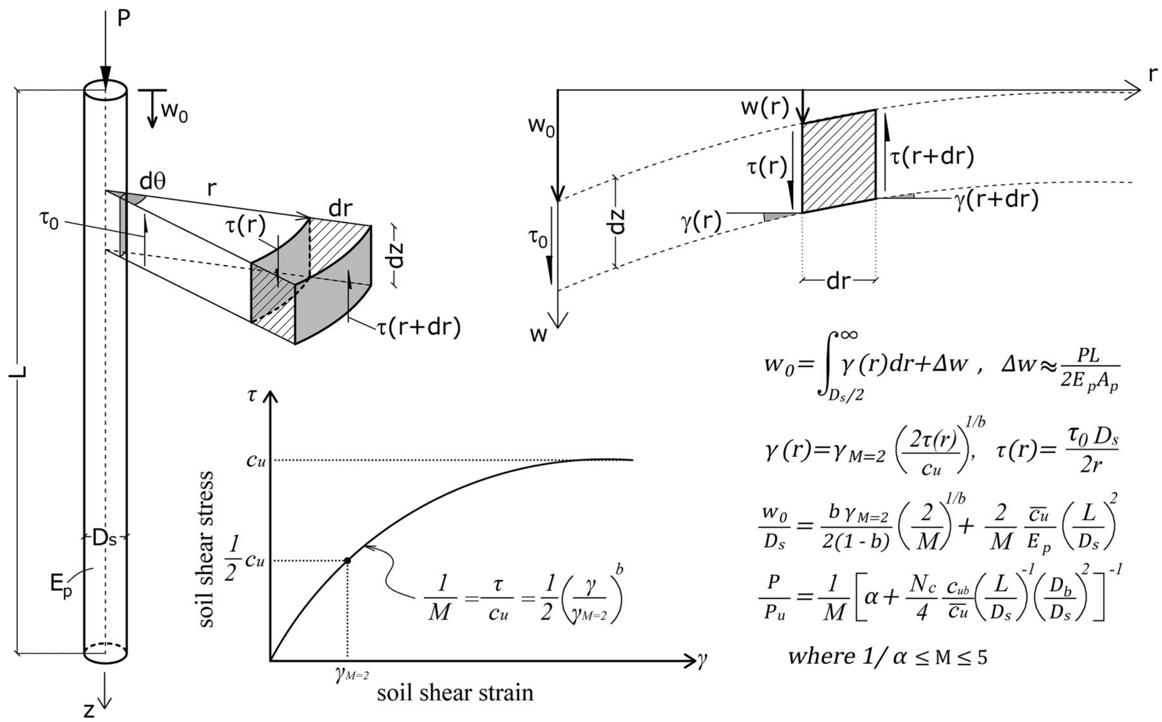

Figure 3. Summary of analytical solution for nonlinear pile settlement considering shearing of concentric cylinders around the pile, pile compression, and non-linear stress-strain relation (strength mobilisation function from Vardanega and Bolton 2011a), hereafter referred to as "Model 1" (see Vardanega, Williamson, and Bolton 2012a; Vardanega et al. 2018 for further details on the development of the model). Symbols are defined in the notation list.

casting Equation (1) in the form (P/P_u) the inverse of the design Factor of Safety (F_{total}), Equation (2) can be shown as:

$$F_{\text{total}} = P_u / P \tag{2}$$

4. Parameter selection

A summary of the 186 analysed tests conducted in fine-grained deposits under undrained conditions is shown in Table 1. The test sites and corresponding details are reported in the Appendix. The full set of information related to the test data can be accessed via the DINGO Database (Vardanega et al. 2021b).

To demonstrate the variability of force-settlement response in the soil deposits studied in this work some fundamental model parameters have been assigned. In some cases, the values for London Clay have been assigned to the whole database due to lack of published information on the other deposits (Table 2).

This study does not purport to calibrate the values given in Table 2. The main aim of the demonstration is: (a) to compare the field data against a baseline prediction to identify basic trends, (b) validate or falsify different theories as to the causes of the trends, and (c) compare the predictions of Models 1 and 2 to assess the relative importance of input parameters (mainly material properties) on modelling assumptions. The results of a sensitivity study on the importance of the model parameters is provided in the Supplemental data. The authors contend that sensible variations of the parameters listed in Table 2 will not result in the general trends shown later in the paper changing significantly. Although values in Table 2 are sensible to represent most of the data available in the database (e.g. suitable for certain deposits such as London Clay), one should accept that site by site and deposit by deposit one may assign different values. However, for instance, the use of $\alpha = 0.5$ while a London Clay parameter has been used in other UK deposits (e.g. for Gault Clay and Oxford clay, Brettell et al. 2021).

It should also be stressed that the nominal failure load in Equation (1) (i.e. instead of the actual measured failure load) is adopted to derive the instantaneous factor of safety when interpreting the data, as a large number of the tests have not been carried out to failure. This would often be a requirement for employing a load test in design, however the focus of this study is on the performance of settlement predictions, therefore any test to a suitable load level has been included. While rigorous capacity prediction methods are available (e.g. Salgado 2008; Fleming et al. 2009; Guo 2012; Viggiani, Mandolini, and Russo 2012; Poulos 2017), this predicted load is mainly employed to select load levels at which to

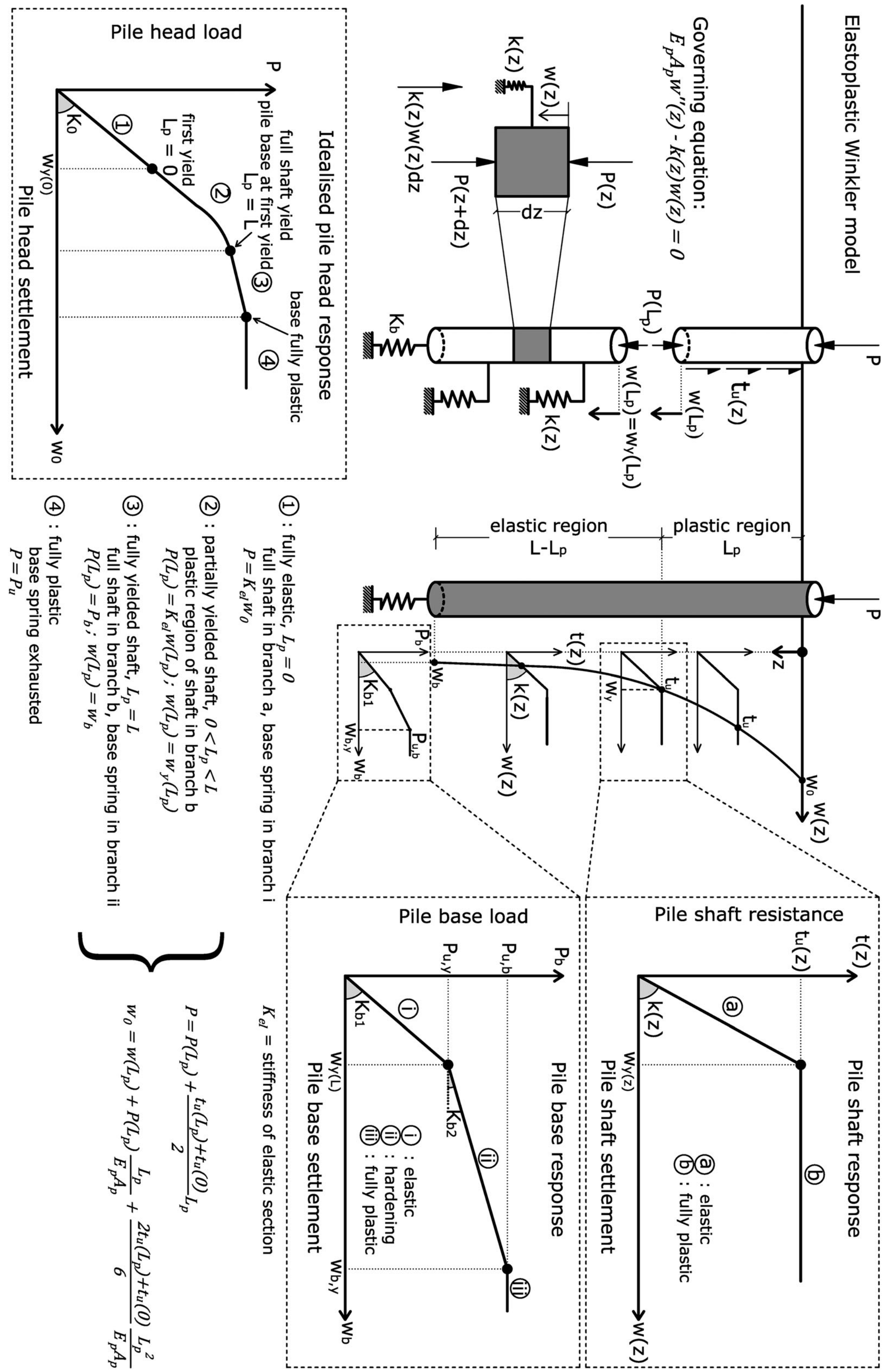

Figure 4. Summary of analytical solution for elastoplastic pile settlement considering elastic – perfectly plastic shaft resistance and bilinear tip resistance, hereafter referred to as "Model 2" (modified after Crispin, Leahy, and Mylonakis 2018, see also Voyagaki et al. 2019 for further details regarding Model 2). Symbols are defined in the notation list.

Table 1. Summary statistics of the DINGO pile test database included in this study.

Statistics of pile tests in fine-grained soils			
Site locations	57	Pile diameter (*m*)	0.15–1.8
Number of test piles	186	Pile length (*m*)	3–39
		Pile slenderness ratio (L/D_s)	6–125
Pile types	Bored, straight shafted and under reamed	Max. applied load (*MN*)	27
Construction method	Maintained Load, Constant Rate of Penetration	Max. settlement (*mm*)	310
Geology[a]	AMC, BAN, GLT, KC, LC, LMBE, MMG, OXC, PUCM, WBY + *various superficial deposits*	Max. normalised settlement w_o/D_s (%)	15

[a]See the Supplemental data for explanation of the Geology codes.

compare predicted and measured settlements. Therefore, this simple method with no additional input parameters is suitable for the analysis presented in this paper. Finally, even for the tests that have been carried out to failure, matching the measured failure load by means of Equation (1) would require calibration of the strength model parameters, which lies beyond the scope of this paper.

4.1 Soil parameter source

Recognising the critical role of the method of parameter determination on pile response (Poulos 1989, 1999, 2004), the analysed tests have been categorised by the source employed to determine the soil parameters, as shown in Table 3 (updated from Vardanega et al. 2021a). This is split into the source of the soil strength parameters and the soil deformation parameters. Most of the data were Category III (78% of the subset in fine-grained soils) tests that include routine site-specific laboratory strength test data. High-quality deformation data was available from the same deposit (Category B) for the piles in London Clay, which accounted for over half of the data (56% of the subset). However, for a small percentage of the pile tests (4% of the subset) only soil description and/or geology information was available (Category IA). It should be noted that the nature of the dataset available for analysis means that Model 1 and Model 2 cannot be evaluated using site specific soil stiffness parameters as these could not be obtained.

As prediction of pile performance requires both soil strength and deformation parameters, certain assumptions are required to allow piles with only Category I, II and/or Category A data to be analysed. Published literature on the soil deposit, such as CIRIA C570 (Chandler and Forster 2001) and C47, (Davis and Chandler 1973) for piles in Mercia Mudstone, was employed where available to estimate soil strength from a Category I description. Category II SPT data was correlated to undrained shear strength using Stroud (1974). Sites with only Category A deformation data were analysed using the London Clay deformation parameters provided in Vardanega and Bolton (2011b).

Two examples are provided in Figures 5 and 6 for the extreme cases of a Category IIIB and Category IA source respectively. The test pile TP1 shown in Figure 5, is from site R37-01 in the DINGO Database and was first reported by Patel (1989, 1992). The $c_u(z)$ line (Figure

Table 2. Key Model Parameters.

Model Parameters[a]	
Factor of Safety (F_{total})	Typical value of 2.5 is used
Concrete Pile Elastic Modulus ($E_{p,\ concrete}$)	30 *GPa*
Steel Pile Elastic Modulus ($E_{p,\ steel}$)	200 *GPa*
Adhesion factor (α)	0.5
Tip bearing capacity factor (N_c)	9
Ultimate skin friction per unit length (t_u)	$\alpha\ c_u\ \pi\ D_s$
Soil shear modulus (G_s)	320 c_u
Winkler modulus (k)	$2\ \pi\ G_s/\ln\ [5\ \rho\ (1-v_s)\ L/d]$ where ρ is the ratio of the mean and maximum $G_s(z)$
Base spring stiffness (K_b)	$2\ G_s(L)\ D_b/(1-v_s)$
Shear strain at 50% strength mobilisation ($\gamma_{M=2}$)	7×10^{-3}
Soil non-linearity exponent (b)	0.6
Soil Poisson's ratio (v_s)	0.5

[a]Sources: Meyerhof (1976), Randolph and Wroth (1978), Poulos and Davis (1980), Fleming (1992), Patel (1992), Salgado (2008), Vardanega and Bolton (2011b); Vardanega et al. (2012b), LDSA (2017).

Table 3. Classification of the DINGO data set based on data quality category for the fine-grained soils subset (updated from Vardanega et al. 2021a).

Strength Parameter Source			Deformation Parameter Source		
Category	Description	No. of Tests	Category	Description	No. of Tests
I	Soil descriptions and geology	7	A	High-quality lab test data from different sites in a similar deposit	80
II	In situ test data (e.g. SPT tests)	32	B	High-quality lab test data from different sites in the same deposit	106
III	Routine lab test data from on site (e.g. UU triaxial tests)	147	C	In situ test data (e.g. pressuremeter tests) or high-quality lab test data from on site	0
IV	High-quality lab test data from on site (e.g. CIU triaxial tests on undisturbed samples)	0	D	In situ test data and high-quality lab test data from on site	0

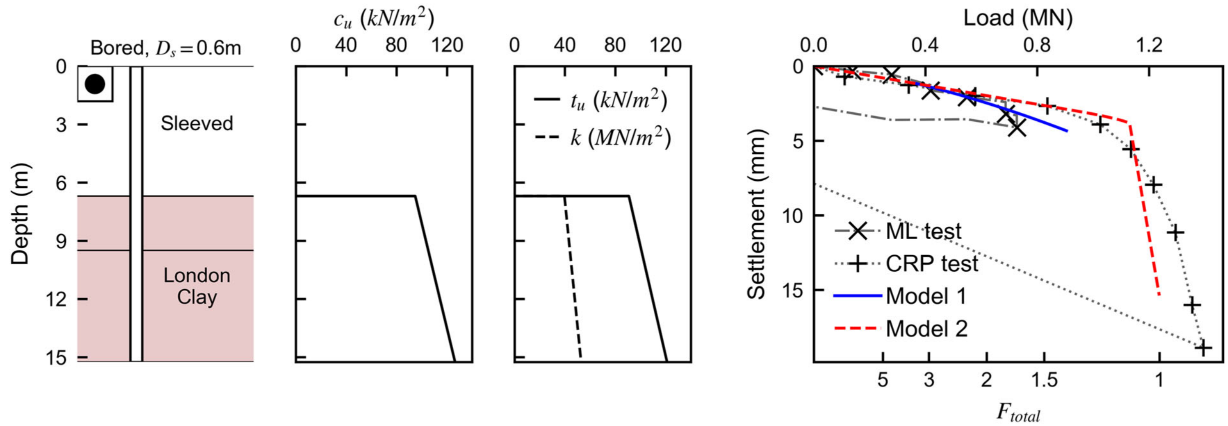

Figure 5. Site R37-05, pile TP1 (a) Soil profile; (b) undrained shear strength variation with depth; and (c) model parameters derived from the soil investigation data for a database site classified as high quality (IV) and; (d) corresponding predicted load-settlement curves plotted against test data.

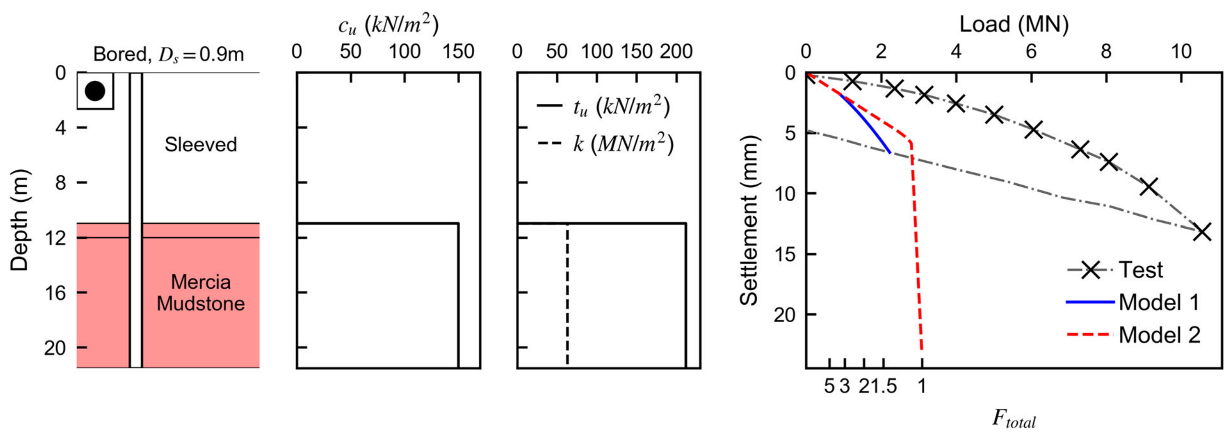

Figure 6. Site R24-01, pile P1(a) Soil profile; (b) undrained shear strength variation with depth; and (c) model parameters derived from the soil investigation data for a database site classified as low quality (I) and; (d) corresponding predicted load-settlement curves plotted against test data.

5(b)) is based on triaxial test data from unconsolidated undrained (UU) triaxial compression tests on 100 mm samples from the site. In addition, the founding strata is London Clay, for which published correlations between deformation parameters and strength parameters are available (Vardanega and Bolton 2011b). Therefore, this pile is designated as having Category IIIB data.

The maximum shaft resistance per unit length (t_u) and Winkler spring stiffness (k) for Model 2 are shown in Figure 5(c). These are both defined in Figure 4. Figure 5(d), shows the corresponding load-settlement curves for each analytical method (Model 1: Vardanega, Williamson, and Bolton (2012a); Vardanega et al. (2018); Model 2: Crispin, Leahy, and Mylonakis (2018)) plotted against the measured experimental data. As evident in Figure 5(d), both methods give very good predictions of the pile response.

An example where site investigation data do not suffice for direct predictions of the c_u profile is given in Figure 6. The pile test shown is pile P1 from site R24-01 in the DINGO Database and is a test in a site in Mercia Mudstone (MMG), in Cardiff, reported by Kilborn, Treharne, and Zarifian (1989). Available information is limited to soil descriptions, geology and location of the site. The pile was, therefore, designated as Category IA and the c_u profile was roughly estimated making the following assumptions based on CIRIA C570 (Chandler and Forster 2001): MMG was designated as grade IV and the SPT value from CIRIA C570 (table 3.3 page 29) was used with a Stroud factor of 5 (Stroud 1974). The corresponding predicted load – settlement curves are shown in Figure 6(d), where the agreement with the test results is rather poor.

A full set of the undrained shear strength profiles and model parameters used for the pile tests in this study, as

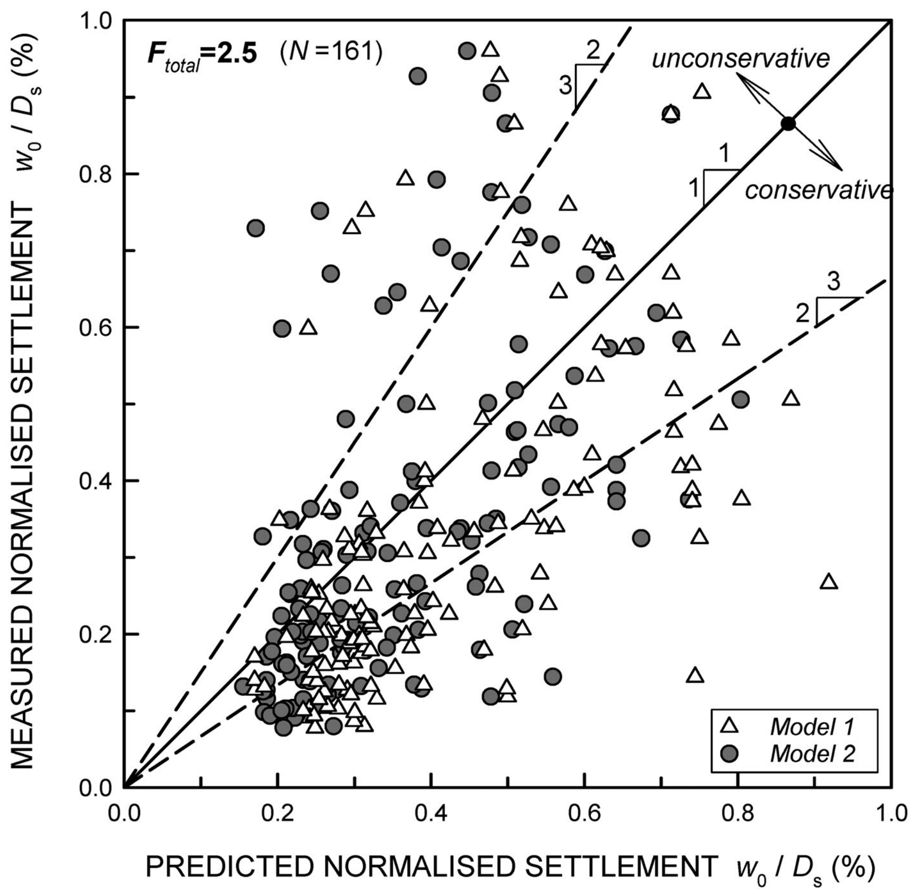

Figure 7. Predicted versus measured pile head settlement plot for the full set of data analysed in this study (model parameters from Table 2).

well as measured and predicted load – settlement curves, is provided in the Supplemental data. The instantaneous factor of safety, F_{total}, (equal to the inverse of P/P_u) is also provided.

5. Results

The measured settlements at specific safety factors have been interpolated and compared with predictions from both Model 1 and Model 2. These are shown in Figures 7–14 for different safety factors, geology categories, pile installation methods, data quality, methods of analysis, pile material properties, and epoch of the tests Note that only the range 0-1% of w_0/D_s is shown on Figures 7-14 (see the Supplemental data to obtain the full set of data points). Also note that the number of data points (N) indicated on Figures 7-14 refers to the full set of data analysed in this study. Figures 7 and 8 show results for different values of the instantaneous factor of safety, computed based on Equations (1) and (2), ranging from 1.5 to 5, which are associated with as many as 186 field tests (the number of tests decreases as the applied load increases since in many cases the tests were not carried out to failure). As evident in Figure 7, points located above the 1:1 line indicate unconservative predictions and vice versa. At low applied loads (F_{total} = 5), both methods seem to provide somewhat conservative predictions. As the load increases, some of the settlement data get significantly overpredicted. However, it is possible that where the capacity is significantly overpredicted by Equation (1), some significantly under-predicted settlements may not visible in the results. This is due to the (expected) absence of measured settlement values above the actual failure load to compare with the relatively low prediction of settlement.

The mean measured normalised settlement is 0.7% and the median is 0.3%. As both the mean and median pile diameter is 0.6 m, this gives settlements of 4 and 2 mm respectively. These low values imply that current pile designs are conservative when considering settlement and, therefore, there is possible reserve capacity available for foundation reuse. However, the standard deviation of the measured normalised settlement is 1.5% which, although it is affected by some significant outliers, highlights the need for settlement prediction in pile design.

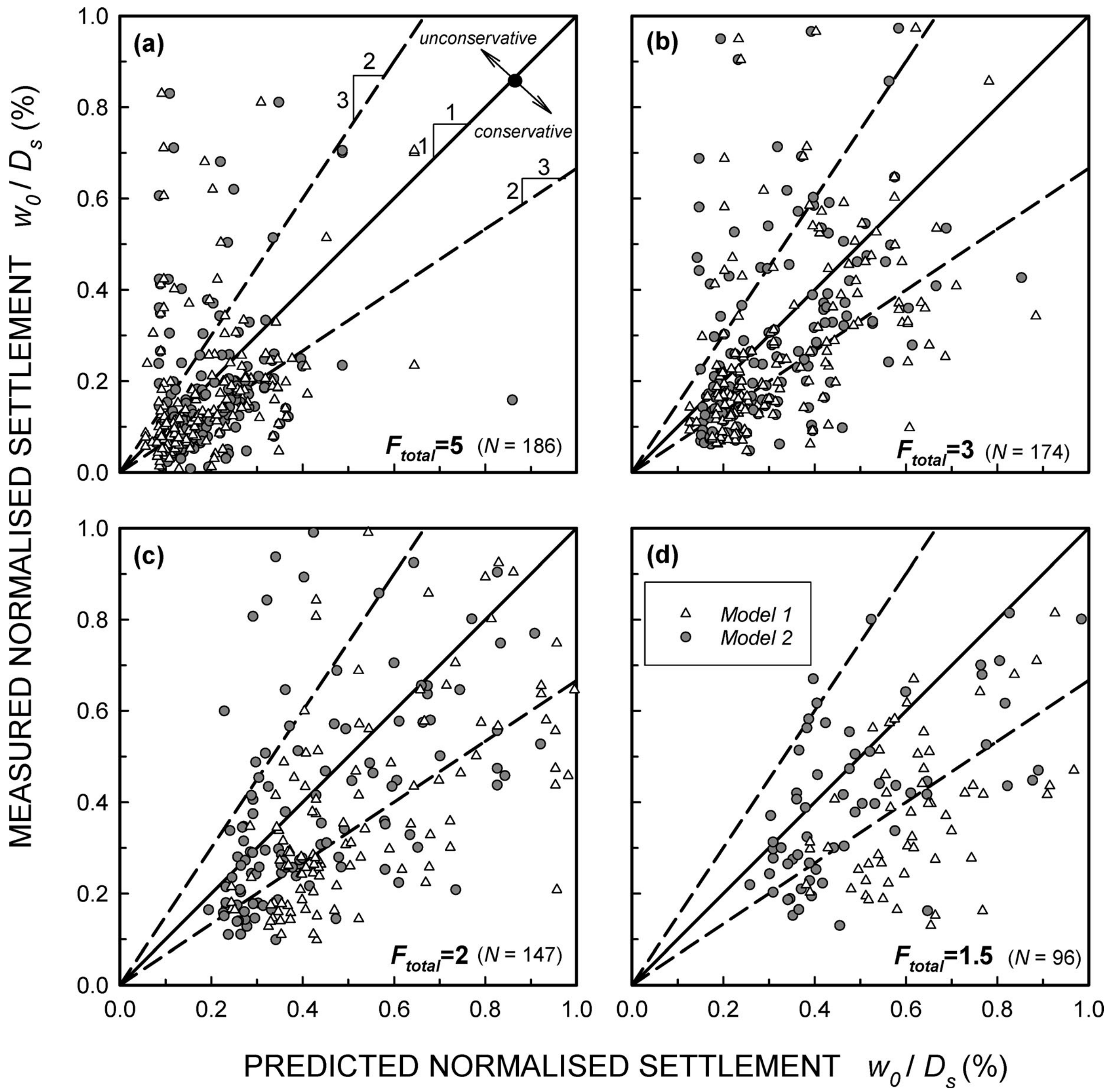

Figure 8. Predicted versus measured pile head settlement plot for different values of factors of safety for the full set of data analysed in this study (model parameters from Table 2).

For applied loads up to 40% of ultimate ($2.5 < F_{\text{total}} < 5$) and despite the scatter of data, a clustering in measured normalised settlements is observed in Figures 7 and 8, centred near 0.1% for $F_{\text{total}} = 5$, near 0.15% for $F_{\text{total}} = 3$ and near 0.2% for $F_{\text{total}} = 2.5$. This suggests proportionality between settlement and applied load within the specific dataset. Accordingly, the following simple fitted formula can be considered:

$$w_0/D_s\ (\%) \ \approx\ \eta_1/F_{\text{total}}\ \pm\ \eta_2 \qquad (3)$$

in which η_1 and η_2 are fitted coefficients. Based on the above observations, $\eta_1 \approx 0.6$, $\eta_2 \approx 0$; the above formula goes through the mid-range of the dataset with approximately 50% of the points above and below. This expression can be used as a rule-of-thumb predictor of pile settlement in clay under undrained conditions for $P/P_u < 40\%$.

Figure 9 reports results for four different methods of parameter determination (Categories IA, IIA, IIIA and IIIB) based on 161 field tests split according to Table 3. It appears that there is no systematic improvement in the accuracy of the predictions when moving from Category IA to Category IIIB. Specifically, the tendency towards conservative predictions improves when moving from Category IA to Category IIA, yet worsens in Category IIIA and improves again (slightly) in Category IIIB. No improvement in the quality of the predictions is observed between Category IIA (involving in-situ strength data) and IIIB (involving laboratory strength

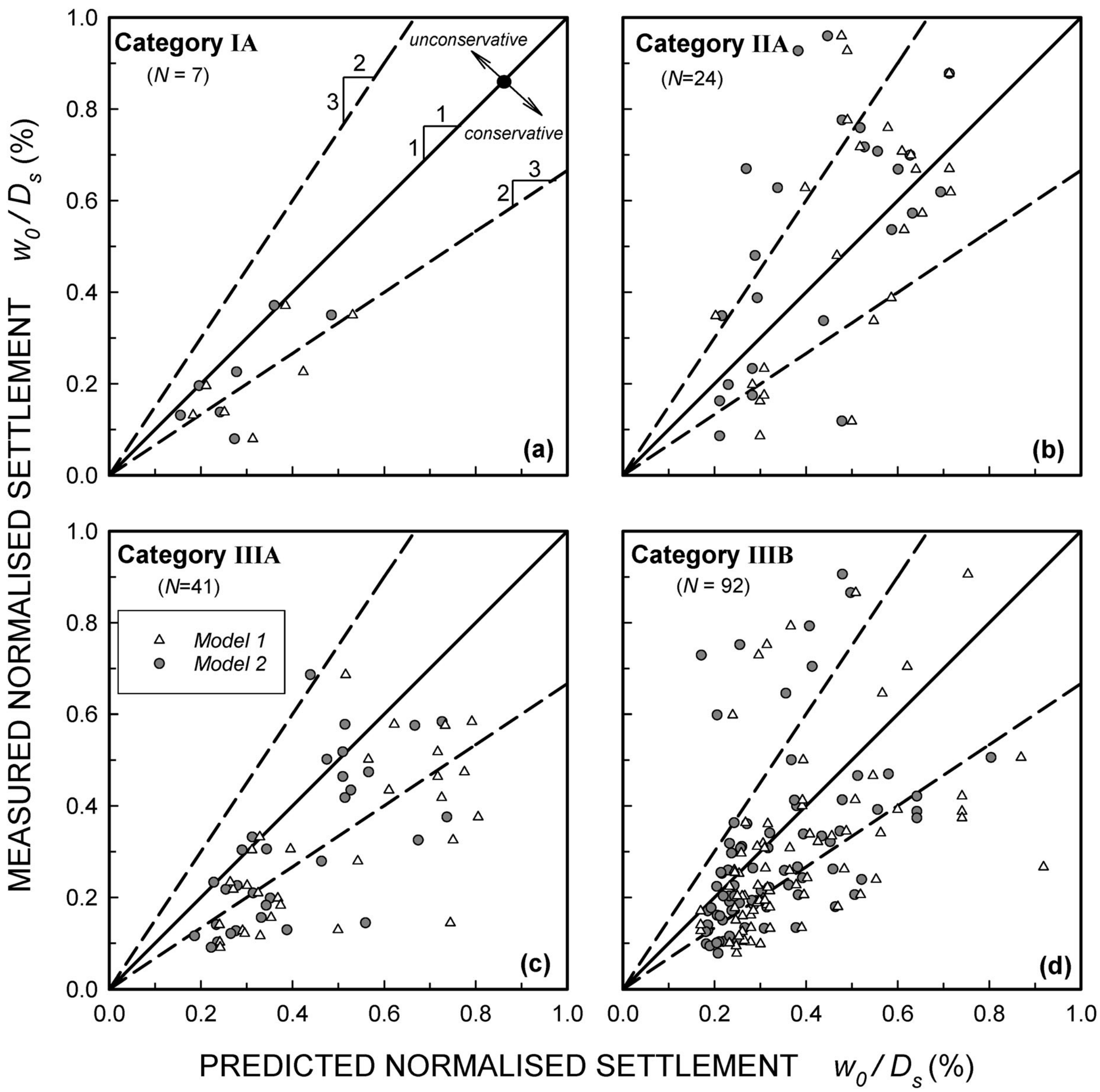

Figure 9. Predicted versus measured pile head settlement plot organised by soil parameter source (categories described in Table 3, model parameters from Table 2).

and stiffness data). These observations may suggest that laboratory data may underestimate clay strength (and, indirectly, stiffness) due to sample disturbance, and provides support as to the use of in-situ strength data in pile settlement predictions in clay.

Figure 10 presents data for four different geology categories (London Clay – LC; Gault Clay – GLT, Mercia Mudstone – MMC, and Others). The observed trends are analogous to those in Figures 7–9, with the settlement predictions being somewhat on the conservative side by both methods. An exception is observed with the Mercia Mudstone where there is no apparent bias in the settlement predictions, especially at high loads, and the Gault Clay where the overprediction in settlement is shown to be more considerable (see Figure 10).

Figure 11 shows results for the epoch of the tests, grouped in two groups: "old tests" dating from the 1950s to 1980s, and "new tests" dating from the 1990s to 2010s. No clear difference in the quality of the settlement predictions is observed between the two groups, with a tendency for somewhat conservative predictions from both methods being apparent, especially at low loads. This observation suggests that there is not systematic variation with time in the quality of the predictions influenced, say, by better sampling in the field, better testing in the laboratory (e.g. due to the introduction of digital measurements in the 1990s), better equipment in the field etc.

It should be recognised that the method of installing piles, (e.g. drilling with or without support fluids) is

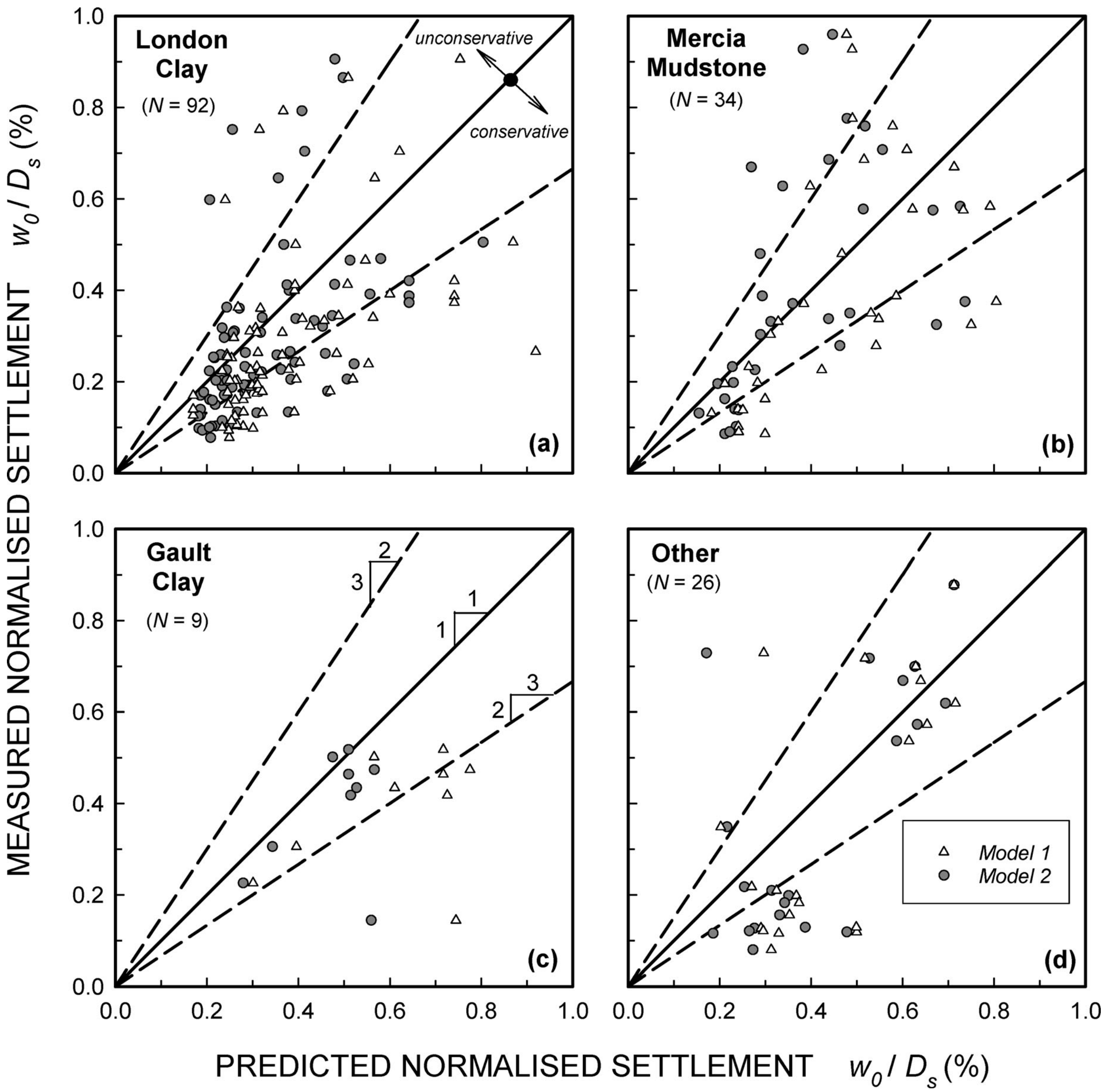

Figure 10. Predicted versus measured pile head settlement plot organised by geology category (parameters from Table 2).

another variable in regard to the performance of piles and why often theory has to be replaced with empirical methods from test pile data. In Figure 12 data are split into four groups based on installation method ("Bored", "Driven-Concrete", "Driven-Steel", and "Continuous Flight Auger"). As about 82% of the 161 tests are associated with bored piles, the aforementioned observations refer mostly to the specific group.

Previous studies have subdivided pile response according to the slenderness of the pile, L/D_s (e.g. Patel 1992). In Figure 13 the tests are split based on a slenderness of 25 – the calculated median slenderness value for the piles with tests that reached $F_{total} = 2.5$. The settlement predictions for the slender piles ($L/D_s \geq 25$) are, in general, slightly more conservative compared to the predictions for the less slender piles ($L/D_s < 25$). A cluster of points is visible for the less slender piles; however, this cluster only shows a small variation in predicted values. There is a similar spread of measured values in both plots. This could be due to the smaller range of pile lengths in this slenderness category.

Figure 14 shows data from all tests split in two groups referring to Model 1 and Model 2, for a nominal safety factor of 2.5 ($P/P_u = 0.4$). No notable differences are observed in the quality of settlement predictions between the two methods, which indicates that the predictions depend mainly on soil properties and to a lesser extent on the method of analysis. Poulos (1999, 13) states that "The selection of geotechnical parameters also plays

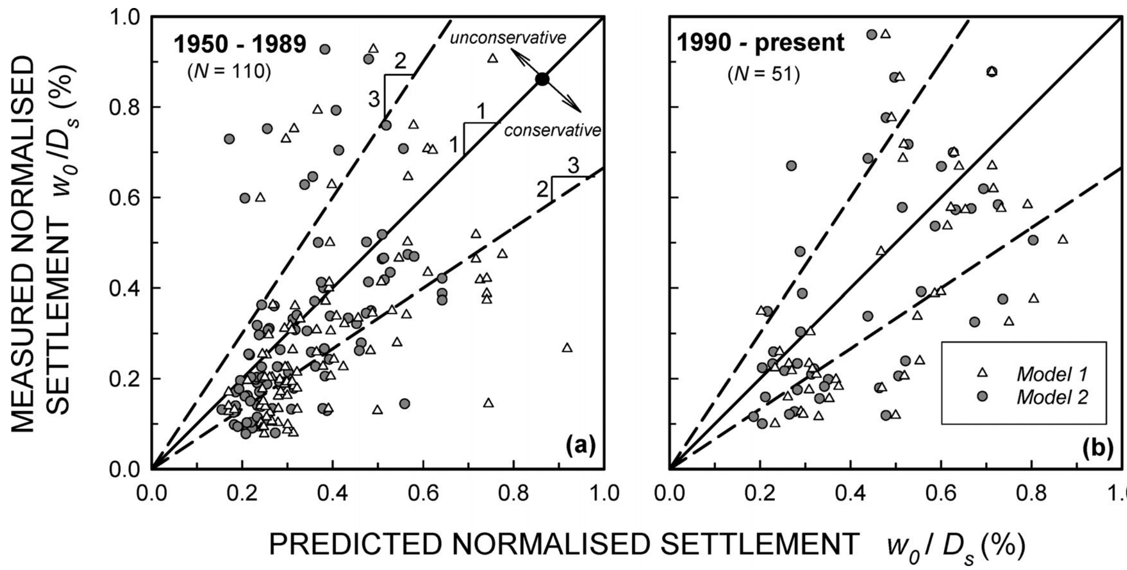

Figure 11. Predicted versus measured pile head settlement plot organised by epoch of the tests (parameters from Table 2).

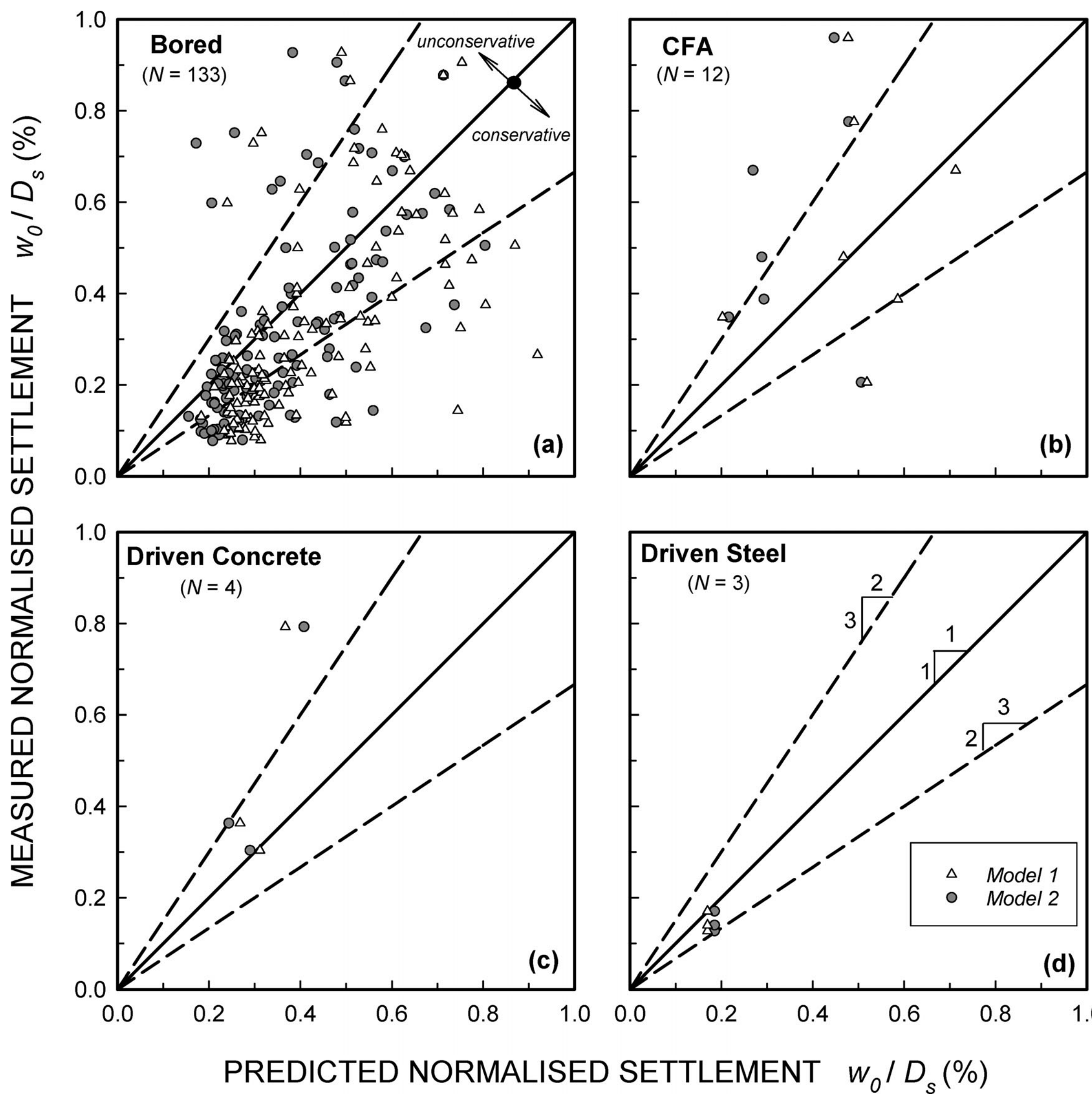

Figure 12. Predicted versus measured pile head settlement plot organised by construction method (parameters from Table 2).

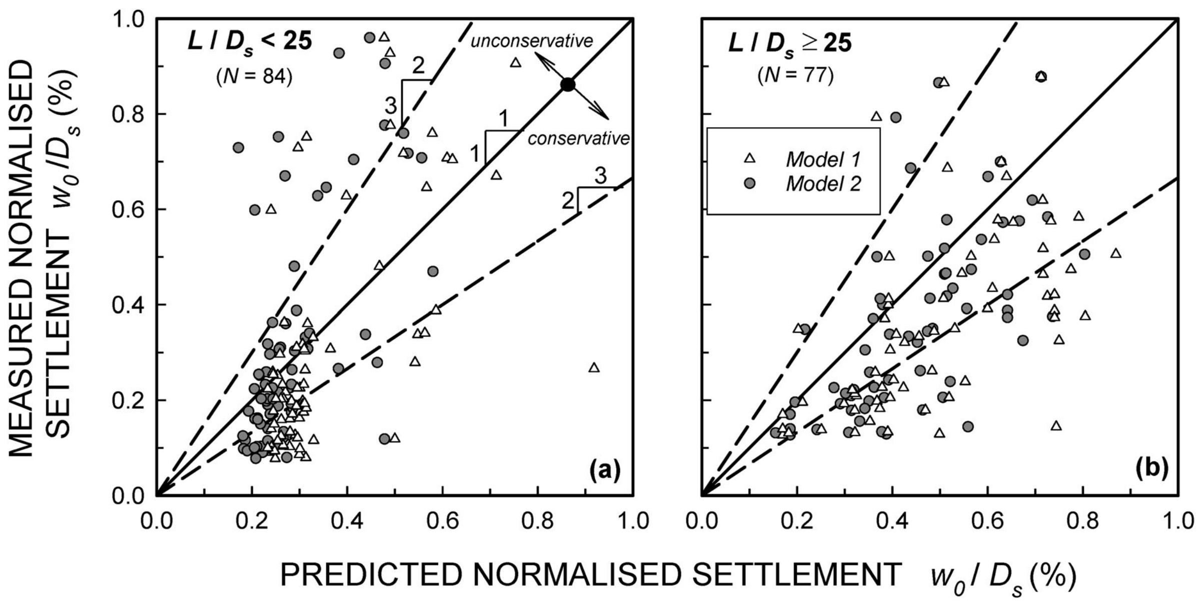

Figure 13. Predicted versus measured pile head settlement plot organised by slenderness of the test pile (parameters from Table 2).

a major part in the success or otherwise of a prediction, and may outweigh or mask any shortcomings of the method used". Model 2 provides slightly smaller bias towards conservative settlement predictions, but this comes at a price of increased complexity in the analysis, especially before shaft capacity is exhausted.

The predictions of the two models used in this paper are assessed in terms of (1) bias and (2) error. Bias is calculated considering the model factor B defined as the ratio of the measured settlement to the calculated settlement and given by Equation (4) (e.g. Phoon and Tang 2019b; Tang and Phoon 2021).

$$B = \frac{Measured}{Predicted} \tag{4}$$

Phoon and Kulhawy (1999a, 1999b) present a detailed review of statistical metrics and method that can be used to evaluate geotechnical uncertainty and variability and in part emphasise the importance of the coefficient of variation (COV) parameter. The mean and COV (dispersion) for the bias and absolute error are summarised in Table 4. A mean and COV of

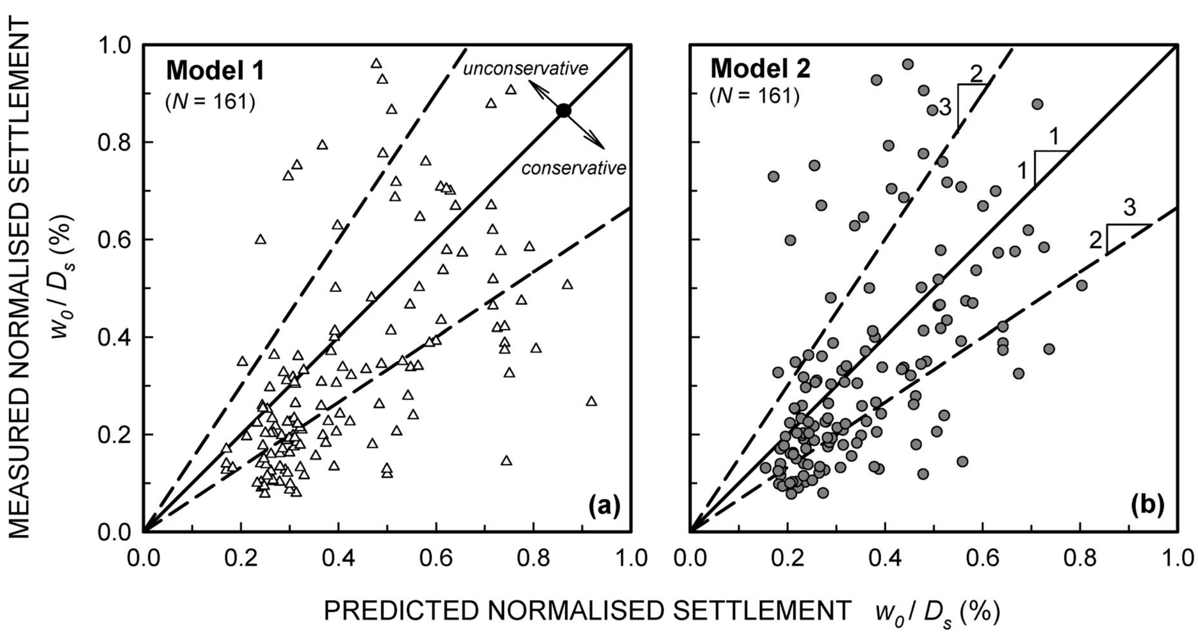

Figure 14. Predicted versus measured pile head settlement comparing Model 1 and Model 2 (parameters from Table 2).

Table 4. Statistics of the model bias factors B (Equation 4) and model error presented in Figures 7–13.

				Statistics of the Bias B = measured/predicted						Statistics of the Absolute Error |predicted – measured|/measured %					
				Model 1			Model 2			Model 1			Model 2		
Case	F_{total}	Figure	N	Mean	σ	COV	Mean	σ	COV	Mean	σ	COV	Mean	σ	COV
Full dataset	5	8(a)	186	1.25	1.9	1.52	1.18	1.71	1.44	100	194	1.92	105	193	1.84
	3	8(b)	174	1.57	3.45	2.19	1.73	3.48	2.01	82	117	1.42	69	80	1.15
	2.5	7	161	1.6	3.9	2.44	1.83	4.16	2.27	77	110	1.44	59	69	1.18
	2	8(c)	147	2	6.71	3.3	2.7	7.88	3.1	78	111	1.41	57	72	1.27
	1.5	8(d)	96	1.5	2.88	1.93	2.18	5.54	2.53	92	135	1.47	60	102	1.71
Category IA	2.5	9(a)	7	0.66	0.24	0.37	0.75	0.26	0.34	80	99	1.23	57	85	1.50
Category IIA		9(b)	24	1.04	0.48	0.46	1.25	0.59	0.47	53	75	1.41	47	62	1.33
Category IIIA		9(c)	41	1.80	3.24	1.80	1.91	3.21	1.69	115	195	1.69	78	110	1.42
Category IIIB		9(d)	92	1.73	4.71	2.73	2.03	5.09	2.5	67	55	0.83	53	41	0.77
London Clay	2.5	10(a)	92	1.63	4.67	2.86	1.86	4.95	2.66	66	55	0.83	52	41	0.78
Mercia Mudstone		10(b)	34	0.87	0.45	0.52	1.07	0.58	0.54	88	200	2.26	63	109	1.73
Gault Clay		10(c)	9	0.65	0.20	0.30	0.82	0.23	0.28	85	126	1.49	45	92	2.03
Other		10(d)	26	2.75	3.89	1.41	3.08	4.29	1.39	96	89	0.93	82	73	0.89
1950–1989	2.5	11(a)	110	1.57	4.32	2.76	1.85	4.67	2.52	72	70	0.98	53	51	0.96
1990-present		11(b)	51	1.67	2.81	1.68	1.78	2.79	1.56	88	167	1.89	70	97	1.38
Bored	2.5	12(a)	133	1.42	3.94	2.78	1.66	4.27	2.57	73	71	0.98	56	55	0.98
CFA		12(b)	12	4.65	4.76	1.03	4.93	4.55	0.92	65	42	0.64	70	34	0.49
Driven Concrete		12(c)	4	1.14	0.87	0.76	1.16	0.77	0.67	316	577	1.83	181	305	1.68
Driven Steel		12(d)	3	0.86	0.13	0.15	0.79	0.12	0.15	18	17	0.91	29	19	0.64
$L/D_s < 25$	2.5	13(a)	84	1.57	2.44	1.56	1.92	2.82	1.47	74	70	0.94	58	52	0.91
$L/D_s \geq 25$		13(b)	77	1.63	5.05	3.09	1.73	5.27	3.04	80	143	1.79	60	84	1.40

B close to 1 and 0, respectively, correspond to exact predictions, whereas, mean values greater than 1 indicate conservative predictions, while mean values below 1 unconservative. A collection of other models and their bias and dispersion factors have been compared using different databases in the works of Phoon and Tang (2019b) and Tang and Phoon (2021). Most of the calculated bias factors are greater than 1, indicating the predictions are generally conservative. This is particularly evident when more data is available. In addition, the difference between the models is minor; however, Model 2 is marginally more conservative than Model 1, with a slightly lower COV in almost all cases.

6. Discussion

After examination of the results shown in Figures 7–14, the following main observations can be made: *First*, there is a tendency towards conservative predictions (i.e. points plotting under the 1:1 line) in most of the graphs. *Second*, this trend probably cannot be attributed to the analysis method employed, as Models 1 and 2 which are based on rather different assumptions (particularly with reference to tip action), exhibit essentially the same trends (Figure 14). Likewise, this trend cannot be attributed on soil material type (Figure 10), epoch of the tests (Figure 11), construction method (Figure 12), or pile slenderness (Figure 13). *Third*, a clear trend towards improved predictions is observed when field data are used (Figure 9, Category IIA) over laboratory data, even when high-quality measurements involving both soil strength and stiffness are employed (Figure 9, Category IIIB). Therefore, the source of the settlement over-prediction should probably be sought in the material properties employed in the analysis, not in the mechanistic models. These properties include: (i) pile stiffness, (ii) soil stiffness (measured directly or inferred from soil strength-to-stiffness correlations), (iii) soil strength.

With reference to pile stiffness (for bored piles which represent about 82% of the dataset), no field or laboratory data are available to allow a proper statistical evaluation of its influence on settlement predictions. Had the dataset included more steel section piles it may have been possible to better ascertain the effect of pile stiffness variability on the prediction quality. The lack of variability of the results depending on epoch of construction may imply that workmanship has not altered significantly during the last 50–70 years, at least in the piling construction industry. However, in light of the results in Figure S1 (Supplemental data), it is unlikely that pile stiffness is responsible for the trends at hand, as these are more or less unaltered for a wide range of E_p values, from 20 GPa to 40 GPa. The minor effect of E_p on pile settlement is backed by theoretical evidence (see Supplemental data).

With reference to soil stiffness (measured directly or inferred via stiffness-to-strength correlations), this naturally controls the behaviour at high factors of safety, yet its influence diminishes at low factors of safety where strength governs pile behaviour. However, had soil stiffness been the main contributor to this effect, this would not justify the trends observed in Figure 8 where the tendency towards more conservative predictions is smaller for high factors of safety ($F_{\text{total}} = 3$ and 5) than for low factors of safety ($F_{\text{total}} = 2$ and 1.5). In other words, settlement predictions improve when soil stiffness governs pile behaviour over soil strength – not the other way around.

The above observations give weight to the postulation that a systematic underpredicted soil strength is present in the dataset. This hypothesis may explain the following key trends: *First*, the increased quality of the predictions in Figure 8 with an increasing factor of safety i.e. when soil stiffness governs the behaviour, not strength. *Second*, the surprisingly good performance of Category IIA data (involving in-situ strength data) over Category IIIA and Category IIB (involving laboratory data). This suggests that soil strength measured in the field (even using correlations with SPT data) may provide better predictions than soil strength measured in the laboratory for use in pile analysis. Undrained shear strength is sensitive to test type and shear mode and rate effects (cf. Chen and Kulhawy 1993; Mayne et al. 2009; Beesley and Vardanega 2020; Kulhawy and Mayne 1990). *Third*, it can explain the insensitivity of the observations in soil material type, epoch of tests, construction method, pile slenderness, and method of analysis. *Fourth*, it can explain the tendency for settlement over-prediction even at high factors of safety, as soil stiffness is often inferred from soil strength (e.g. in Category III data).

The sources for the underprediction in soil strength should be naturally sought in: (1) the disturbance of soil samples collected in the field; (2) the scarcity of high quality in-situ strength measurements (e.g. CPT tests, vane tests) over laboratory measurements. Evidently, conservatism may provide higher safety margins, but does not ensure good predictions of performance.

As a final remark, high-end numerical models such as non-linear FEM, although very powerful in modelling foundation elements such as the one at hand, are not anticipated to alleviate the issue as their performance, like that of the Models 1 and 2, fundamentally depends on the quality of soil material properties they are supplied with. To improve pile settlement predictions, a renewed attention should be made to achieving high quality soil strength and stiffness measurements accompanied by field testing to failure, available in open-source databases.

7. Summary and conclusions

The opensource DINGO Database of over 500 pile load tests in various geological deposits, across the UK has been assembled. The data originated from industrial and literature sources; they are accompanied by site investigation data, have been categorised by geological deposit, pile construction type and epoch; they are user friendly and open to the engineering and scientific community. A subset of 186 pile tests in 57 test sites, in fine-grained soils, in different regions of the country were selected for performing preliminary analyses against two elastoplastic "*t-z*" analytical closed-form solutions. These are: (*i*) a power-law non-linear soil model suitable for hand calculations based on mobilisable strength design (MSD) principles and (*ii*) an analytical closed-form elastoplastic Winkler solution, suitable for spreadsheet or pocket calculator analysis, encompassing depth-dependent elastic-perfectly-plastic "t-z" curves and elastoplastic tip resistance.

The model parameters were calculated based on ground investigation data from the test sites. Where these were not available, values were based on data from similar sites and engineering judgement. The analytical predictions, although based on idealised models, are in good agreement with the test data. Graphs illustrating the predicted vs. measured normalised pile head settlement show a trend towards somewhat conservative predictions of pile settlement. The following conclusions can be drawn:

(1) Comparisons between pile settlement predictions and field measurements from 186 field tests were carried out for five values of the instantaneous factor of safety, ranging from 1.5 to 5. At low loads ($F_{\text{total}} = 5$; $P/P_u = 0.2$), both methods appear to provide somewhat conservative predictions. However, as the load increases, some of the settlement data get significantly underpredicted.

(2) The mean measured normalised settlement is 0.7% and the median is 0.3%. As both the mean and median pile diameter is 0.6 *m*, this gives settlements of about 4 and 2 mm, respectively. For safety factors between 2.5 and 5 a clustering in normalised pile head settlement is observed in the data, near 0.1% of the pile shaft diameter for $F_{\text{total}} = 5$, near 0.15% for $F_{\text{total}} = 3$ and near 0.2% for $F_{\text{total}} = 2.5$. This observation suggests proportionality between load and displacement. Based on this observation, Equation (3) can be used as a rule of thumb predictor of pile settlement in clay under undrained conditions for $P/P_u < 40\%$.

(3) No systematic improvement in the accuracy of the predictions is observed when moving from Category IA to Category IIIB data. In fact, the predictions seem to worsen for Category IIIA relative to IIA and improve again for Category IIIB. Moreover, no clear differences in the quality of predictions is observed between Category IIA

(involving in-situ strength data) and IIIB (involving laboratory strength and deformation data). This observation provides support towards using in-situ strength data in pile settlement predictions in clay.

(4) Comparisons of settlement predictions and field data in London Clay, Gault Clay, Mercia Mudstone and other soils are analogous to those observed in other classifications, with the settlement predictions being somewhat on the conservative side by both analytical methods. An exception is Mercia Mudstone where there is no apparent bias in settlement predictions, especially at high loads, and the Gault Clay where the overprediction in settlement is rather strong.

(5) No clear difference in the quality of the settlement predictions is observed between "old tests" dating from the 1950s to 1980s, and "new tests" dating from the 1990s to 2010s.

(6) The settlement predictions for "slender" piles ($L/D_s \geq 25$) are in general slightly more conservative compared to the predictions for the "less slender" piles ($L/D_s < 25$). There is a similar spread of measured values in both plots.

(7) Comparisons of predictions from Model 1 and Model 2 for a safety factor of 2.5 ($P/P_u = 0.4$) show no major differences. This indicates that the predictions depend mainly on the soil properties employed and to a lesser extent on the method of analysis. Model 2 provides slightly smaller bias towards conservative settlement predictions, but this comes at a price of increased complexity, especially before shaft capacity is exhausted.

To move towards genuine performance-based geotechnical design, high-quality databases of field tests complemented by high-quality lab data, are needed to study the consequences of the key design choices made by geotechnical engineers. Traditional design approaches based on conservative selections of design parameters, although acceptable from a safety viewpoint, do not provide high value for money and should be replaced with genuinely performance-based approaches.

Nomenclature

Latin symbols	definitions
A_p	pile cross sectional area
B	model factor
b	soil non-linearity exponent
c_u	undrained shear strength
c_{ub}	undrained shear strength at pile base
$\overline{c_u}$	average undrained shear strength
D_s	pile shaft diameter
D_b	pile base diameter
E_p	elastic modulus of the pile
E_s	elastic modulus of soil
F_{total}	overall factor of safety
$G_s(z)$	soil shear modulus at depth z
K_{el}	head stiffness of elastic region of pile
K_b	pile base stiffness
K_{b1}, K_{b2}	pile base stiffness in region i, pile base stiffness in region ii
$k(z)$	Winkler spring stiffness at depth z
L	pile length
L_p	length of plastic region of pile
M	soil strength mobilisation factor
N	number of data points
N_c	bearing capacity factor
$P(z)$	pile axial load at depth z
P, P_b, P_s	total applied load, applied base load, applied shaft load
$P_{u,y}$	pile base load when shaft resistance exhausted
P_u, $P_{u,b}$, $P_{u,s}$	total ultimate resistance (bearing capacity), ultimate base resistance, ultimate shaft resistance
r	radial distance from pile axis
$t_u(z)$	ultimate skin friction per unit length at depth z
$w(z)$	pile settlement at depth z
w_0, w_b, Δw	pile head settlement, pile base settlement, pile elastic shortening
w_{by}	pile base yield settlement
$w_y(z)$	soil yield settlement at depth z
z	depth below ground level

Greek symbols	definitions
α	adhesion factor
γ	shear strain
$\gamma_{M=2}$	shear strain when half the undrained shear strength c_u is mobilised
η_1, η_2	fitted coefficients
ν_s	soil Poisson's ratio
ρ	vertical soil inhomogeneity constant
τ, τ_o	shear stress, shear stress at the pile-soil interface
$\tau(r)$	shear stress attenuation function

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Data availability

The underlying data used for the analysis presented in this paper can be sourced from the DINGO database which can be freely downloaded from the data.bris repository via the following weblink: https://doi.org/10.5523/bris.89r3npvewel2ea8ttb67ku4d (Vardanega et al. 2021b).

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Appendix

Table A1. Summary of 57 pile test sites from the DINGO Database analysed in this study (Geology codes are listed in DINGO Database (Vardanega et al. 2021b)).

		Number of				DINGO Categories			
DINGO Project ID	Location	Boreholes	SPTs	Triaxial Tests	Test Piles	Construction Type	Geology	Quality	Year
D04_01	Childsbridge Lane	2	0	0	2	BOR	GLT	IIIA	1978
D04_02	Cockney Wood Accomodation Bridge	1	0	0	1	BOR	GLT	IIIA	1978
D04_03	Darent River	2	0	0	2	BOR	GLT	IIIA	1978
D04_06	Oxenhillshaw	2	0	0	2	BOR	GLT	IIIA	1978
D04_07	Park lane Accomodation Bridge	1	0	0	1	BOR	HEAD,GLT	IIIA	1978
D04_09	St. Clere South Abutment	1	0	0	1	BOR	GLT	IIIA	1978
D07_01	Huntingdon Road, Cambridge, CB24 4AE	1	0	0	1	BOR	AMC	IIIA	2018
D07_02	Mere Way, Huntingdon, PE28	1	0	0	1	BOR	HEAD,OXC	IIIA	2018
D07_04	Buckden Road, Brampton, Huntingdon, PE28	1	0	0	1	BOR	TILL,WHCK, OXC	IIIA	2018
D07_05	B1043	1	0	0	1	BOR	RTD,TILL	IIIA	2018
D07_06	A1, Brampton, Huntingdon, PE28 4NQ	1	0	0	1	BOR	ALV,RTD,OXC	IIIA	2018
D07_07	A1198, Godmanchester, Huntingdon, PE29 2LJ	1	0	0	1	BOR	TILL,OXC	IIIA	2018
D07_08	B1050, Longstanton, Cambridge, CB23 8DS	1	0	0	1	BOR	KC,AMC	IIIA	2018
D09	Stanford Hall, Loughborough	5	0	0	5	CFA	SUPD,BAN, WBY	IIIA	2012
D10	Etihad Campus, Manchester	13	280	0	11	BOR	TILL,PUCM	IIA	2000
R01_01	Wimbledon, London	1	0	14	3	BOR, CFA	ALV,LC	IIIB	2008
R01_02	Chessington	1	0	43	1	BOR	LC	IIIB	2008
R05	Waltham, Grimsby Depot, Grimsby	2	15	51	1	BOR	BOC	IIIA	2001
R09	Hendon, North London	2	0	10	3	DRI_S	LC	IIIB	1979
R11	Manchester Airport	18	110	0	16	BOR, CFA, DRI_S	TILL,MMG	IIA	1996
R12	Birmingham (national exhibition centre)	5	53	0	2	BOR	MMG	IIA	1984
R14	Tanners End Lane, Edmonton, London	13	0	24	7	BOR	RTD,LC,LMBE	IIIB	1978
R15	Leicester	2	7	7	2	BOR	MMG	IIIA	1971
R17	Kensal Green Gas Works	1	0	84	3	BOR	LC	IIIB	1954
R18	Borehamwood, Hertfordshire	1	0	0	8	BOR	LC	IIIB	1955
R21	Burnaston, to the southwest of Derby	26	51	30	14	BOR, DRI_C	TILL,MMG	IIIA	1995
R22	Westbourne Park Station, London	2	0	29	9	CFA	LC	IIIB	1980
R24_02	Grangetown Link (Clarens Bridge), Cardiff	1	0	0	2	BOR	MMG	IA	1989
R24_03	Eastmoors Link, Cardiff	3	128	0	6	BOR	MMG	IIA	1989
R24_04	Grangetown Link & Cogan Spur, Cardiff	1	0	0	4	BOR	MMG	IA	1989
R27	Kilroot, County Antrim, Northern Ireland	1	140	95	3	BOR	GLT,MMG, UEXG	IIIA	1976
R29	Bothkennar, Forth Estuary, Scotland	1	0	90	12	CFA	ITDU	IIA	1992
R31	Aldgate Place, Whitechapel, east London	5	63	82	1	BOR	RTD,LC,LMBE	IIIB	2013
R32	Wembley Stadium in north London	4	72	0	2	BOR	LC	IIA	2014
R33_01	Southall	1	0	29	7	BOR	LC	IIIB	1953
R33_02	Barnet	1	0	29	6	BOR,DRI_C	LC	IIIB	1953
R34	Cardiff (Taff Viaduct)	7	64	0	6	BOR	RTD,MMG, SUPD	IIIA	1992
R35	British Library, London	1	98	0	2	BOR	LC,LMBE,TAB, WHCK	IIIA	1982
R37_01	Lambeth, London	1	0	27	2	BOR	LC	IIIB	1988
R37_02	Lambeth, London	1	0	8	1	BOR	LC	IIIB	1970
R37_03	Oxhey, Watford, London	1	0	22	1	BOR	LC	IIIB	1970
R37_04	Wilson St, London	1	0	18	1	BOR	RTD,LC	IIIB	1970
R37_05	Broadmead Rd, Brickfield, London	2	0	50	2	BOR	RTD,LC	IIIB	1966
R37_06	Kidbrooke, Greenwich, London	1	0	96	12	BOR	LC	IIIB	1968
R37_07	Brentford, London	1	0	63	2	BOR	RTD,LC	IIIB	1968
R37_08	Cambridge Rd, Kingston, London	2	0	67	2	BOR	LC	IIIB	1968
R37_09	Cambridge Rd, Girdlestone Rd, Highgate, London	1	0	76	6	BOR	LC	IIIB	1972
R37_10	Woodgreen Shopping Centre, Wood Green, London	5	0	20	5	BOR	LC,LMBE	IIIB	1972
R37_11	Loughborough Park, Brixton, London	2	0	41	2	BOR	RTD,LC	IIIB	1971

(Continued)

Table A1. Continued.

DINGO Project ID	Location	Number of: Boreholes	SPTs	Triaxial Tests	Test Piles	DINGO Categories: Construction Type	Geology	Quality	Year
R37_12	Pond St, Hampstead, London	2	0	119	2	BOR	LC	IIIB	1968
R37_13	Prince of Wales Rd, Hampstead, London	1	0	43	2	BOR	LC	IIIB	1969
R37_14	Keatley Green, London	7	0	74	7	BOR	ALV,LC	IIIB	1969
R37_15	Canon Street Station, London	3	0	51	3	BOR	LC	IIIB	1969
R38	Chattenden, Kent	1	0	32	28	CFA,BOR,MISC, DRI_C	LC	IIIB	2013
R44	London's Heathrow Airport	1	0	0	1	BOR	RTD,LC	IIIB	2001
R45	Alperton Lane, Wembley, Middlesex	1	0	95	12	BOR	LC	IIIB	1966
R46	Angel Square, Islington, London	2	78	66	2	BOR	LC,LMBE,TAB	IIIB	1993