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Quantifying settlement reduction from two design codes for piles in London clay

Quantification de la réduction du tassement à partir de deux codes de conception pour les pieux dans l'argile de Londres

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ABSTRACT: Piled foundations generally fail due to excessive settlements causing damage to the supported structure. While various methods are available to predict pile settlements, historically codes of practice have relied upon large factors of safety to ensure pile plunging is sufficiently unlikely. This can lead to confusion between serviceability and collapse limit states as both are concerned with limiting settlements. Modern codes of practice do require the serviceability limit state to be considered independently. However, guidance is rarely provided as to the specific method of predicting settlement and in some cases the expressed expectation is that this condition is satisfied for a pile designed according to the collapse limit state. Therefore, quantifying the settlement reduction provided by a design code is of interest to the engineer as this allows the level of design conservatism to be evaluated. However, this can only be done with a sufficiently large database of pile load tests in the soil deposit of interest. In this paper, two design codes are applied to a dataset of pile tests in London clay. Measured settlements are then obtained from load-test results and compared with those obtained from an unfactored 'α-method' analysis to obtain the settlement reduction provided by the design code.

RÉSUMÉ : Les fondations sur pieux échouent généralement en raison de tassements excessifs causant des dommages à la structure supportée. Bien que diverses méthodes soient disponibles pour prédire les tassements des pieux, historiquement, les codes de pratique se sont appuyés sur de grands facteurs de sécurité afin de garantir que le plongement des pieux est suffisamment improbable. Cela peut conduire à une confusion entre l'état de service et les états limites d'effondrement, puisque tous deux sont concernés par la limitation des tassements. Les codes de pratique modernes exigent que l'état limite de service soit considéré indépendamment. Cependant, des indications sont rarement fournies à propos de la méthode spécifique de prévision du tassement et, dans certains cas, on s'attend à ce que cette condition soit remplie pour un pieu conçu selon l'état limite d'effondrement. Par conséquent, la quantification de la réduction de tassement fournie par un code de conception est intéressante pour l'ingénieur puisque cela permet d'évaluer le niveau de conservatisme de la conception. Néanmoins, cela ne peut être réalisé qu'avec une base de données suffisamment large, de tests de charge de pieux dans le dépôt de sol d'intérêt. Dans cet article, deux codes de conception sont appliqués à un ensemble de données de tests de pieux dans de l'argile de Londres. Les tassements mesurés sont ensuite obtenus à partir des résultats des essais de charge et comparés à ceux trouvés à partir d'une analyse de la «méthode α» non pondérée dans le but d'obtenir la réduction de tassement fournie par le code de conception.

KEYWORDS: Piled Foundations, Foundation Performance, Codes of Practice, Partial Factors, Geotechnical Reduction Factors.

1 INTRODUCTION

1.1 *Performance of Piled Foundations*

The performance of piled foundations is arguably one of the most studied subjects in foundation engineering and was the subject of the Rankine lectures of Poulos (1989) and Randolph (2003). Codes of practice frequently specify higher global factors of safety for piled foundations as opposed to other geotechnical constructions e.g., slopes and embankments (cf. Terzaghi & Peck 1948; Vardanega et al. 2012a; Vardanega & Bolton, 2016). By using model factors to ensure that both SLS and ULS failures are sufficiently unlikely (Orr, 2012, Orr & Vardanega, 2013), codes of practice such as Eurocode 7 (BSI, 2004) attempt to reduce settlements to an acceptable level (e.g., Vardanega et al. 2012b). Arguably this leads to a confusion of the serviceability and collapse limit states (Orr & Vardanega, 2013) for piles.

Piled foundations are frequently designed for a 'collapse' limit state that often corresponds to a set displacement e.g., 10% pile diameter (often, incorrectly, attributed to Terzaghi; Fellenius 2013 and Likins et al. 2012). According to Fellenius (2013, p.451):

> "…Terzaghi did not define the capacity as the load generating a movement equal to 10% of the pile diameter, he emphatically stated that whatever definition of capacity or ultimate resistance used, it must not be applied until the *pile toe* has moved at least a distance corresponding to 10% of the pile toe diameter."

Skempton (1959) showed for seven pile tests that at 90% of ultimate load on average the settlement would be 0.04 times pile diameter (D) (range 0.025D to 0.06D). Judgement is needed to determine a consistent criterion for pile 'collapse' failure and therefore design of bored piles in stiff clays is generally concerned with limiting settlements.

1.2 *Codes of Practice*

Skempton (1959, p.157) discussed the use of the factor of safety arguing that:

> "Chiefly it is necessary to keep the settlement within safe limits ..."

Simpson (2000, p.2) explained that in partial factor codes there are three uses of partial factors:

> "a) to allow for uncertainty in material properties, actions or calculation models; b) to ensure that deformations are acceptable. ... c) to achieve compatibility with past practice which has been shown to be safe."

In this paper, two codified approaches AS2159-2009 (Standards Australia, 2009), hereafter referred to as 'AS2159' and Eurocode 7 (BSI, 2004) applied with the UK National Annex (BSI, 2007), hereafter referred to as 'EC7 UK NA', are examined in the context of settlement reduction.

This study follows, in part, the code comparison work presented in Vardanega et al. (2012a), who previously assessed the use of these codes along with other codified approaches for the design of a bored pile in London clay. Further information on the development of the process for assigning the Geotechnical Reduction Factors in AS2159 is given in Poulos (2004) and design of piles to EC7 UK NA is given in Bond & Simpson (2010).

1.3 *Paper Aims*

In this paper, the capacity of piles will be determined by calculation with the aim to evaluate what the subsequent application of partial factors (or geotechnical reduction factors) will have on the pile settlement. A dataset of 108 pile tests in London clay is used to determine settlement values.

2 DATABASES OF PILE LOAD TESTS

There have been various efforts to assemble pile load test databases (e.g., Paikowsky et al. 2004; Lemnitzer & Favaretti, 2013; Galbraith et al. 2014; AbdelSalam et al. 2015; Yang et al. 2015, 2016; Phoon & Tang, 2019; Ong et al. 2021).

A recently concluded Engineering and Physical Sciences Research Council (EPSRC) project 'Databases to INterrogate Geotechnical Observations (DINGO)' has produced an openly accessible database of over 500 pile load tests from sites in the UK (see Vardanega et al. 2021a, 2021b for full details of the database).

In this paper a subset of the database is used. Data from 30 test sites (108 pile tests) represent the piled foundations in London clay (see Figure 1 for the site locations and Table 1 for the site details). Full load-settlement curves can be sourced from Voyagaki et al. (2019, 2021) with the digitized data available from Vardanega et al. (2021b). For most of the sites studied the undrained shear strength values were interpreted by the original authors.

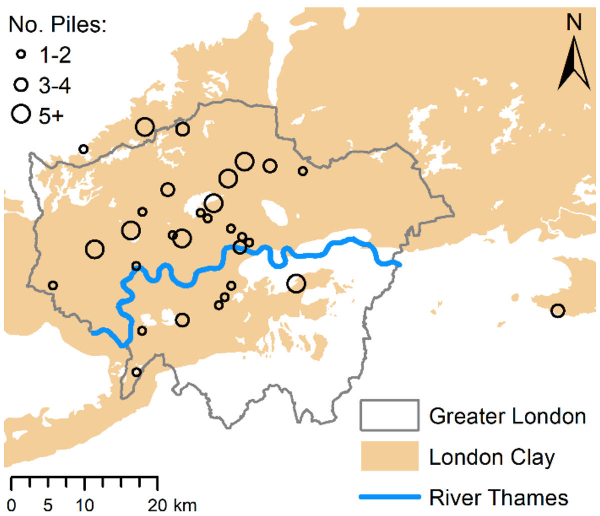

Figure 1. Map of test sites. (Made with Natural Earth. Contains OS data © Crown copyright and database right 2018. Geology Map Data BGS © UKRI 2019).

3 DESIGN BY CALCULATION

3.1 *Design of Piled Foundations in London clay*

The history of pile design in London clay has been recently reviewed by Rutty (2021) indicating in part that while more modern tall building constructions will probably have piles founded in the strata below the London clay (Thanet Sand and Woolwich and Reading beds) most foundation systems in buildings constructed from the 1960s to 1990s would have had piles floating in the London clay strata. This is convenient as the dataset used in this paper is a sub-set of the recently assembled DINGO database (Vardanega et al. 2021a, 2021b) which has a large proportion of pile records from this time interval (approximately 67%).

Pile design in London clay generally uses the adhesion or 'α-method' explained in detail in Skempton (1959). The predicted capacity (P_U) is given by the sum of the calculated shaft resistance (P_s) and the base resistance (P_b), given by Eqs. 1 and 2, respectively.

$$P_s = \pi D \alpha \int_0^L c_u(z)\, dz \qquad (1)$$

where, L is the pile length, α is an adhesion factor and $c_u(z)$ is the undrained shear strength profile of the clay. The response of artificial and/or superficial deposits is often neglected (i.e. $c_u(z)$ assumed to be zero) when low thicknesses are present near the ground surface. The base capacity is given by:

$$P_b = A_b N_c c_u(L) \qquad (2)$$

where, A_b is the area of the pile base, N_c is the bearing capacity factor usually taken as 9 (Skempton, 1959; Meyerhof, 1976).

Skempton (1959) showed for a database of pile tests (not included in DINGO as the original data was not able to be located by the research team) that α generally ranges from 0.3 to 0.6 with an average value of 0.45. Patel (1989, 1992) showed that α could be increased to 0.6 after considering constant rate of penetration tests (CRP) data. Recently, Chantler (2021) presented some new test data giving a range of α of around 0.4 to 0.8. Currently the London District Surveyors (LDSA 2017) guide suggests $\alpha = 0.5$ which was described by Chantler (2021, p.441) as '… a pragmatic lower bound figure'. This value was adopted for the analysis presented in this paper.

3.2 *Design codes*

AS2159 and EC7 UK NA are both limit state design approaches that employ partial factors on applied loads and model and/or material parameters to obtain a design load (Q_D) and a design resistance (R_D), respectively. For a compliant design, the expression $R_D \geq Q_D$ must be satisfied. Vardanega et al. (2012a) expressed this design approach for piles with the generic notation in Eqs. 3 and 4 (using the α method):

$$Q_D = \beta_1 G + \beta_2 V \qquad (3)$$

$$R_D = \frac{\pi D \alpha \int_0^L [c_u(z)/\beta_3] dz}{\beta_5 \beta_7} + \frac{A_b N_c [c_u(L)/\beta_4]}{\beta_6 \beta_7} \qquad (4)$$

where G and V are the unfactored permanent and variable loads, respectively, and β_1 to β_7 are partial factors, given in Table 2 (for more details see Vardanega et al. 2012a).

AS2159 selects the most severe case of a combination of load factors (the governing case for the piles analysed is shown in Table 2) and applies an overall geotechnical reduction factor (ϕ_{gb}) to the design resistance (note that this is the inverse of a partial factor). This is selected from the overall risk category, determined from the average risk rating (ARR), the weighted sum of individual risk ratings (IRRs).

Table 3 shows the individual risk factors assigned for the AS2159 calculation and their weightings, resulting in a geotechnical reduction factor $\phi_{gb} = 0.52$ (equivalent to a β_7 of *1.92*). For more details on the derivation of these values see Vardanega et al. (2012a). Note that the 'Weight' of 'Method of assessment of geotechnical parameters for design' has been increased from 3 to 4 for this study as a range of sites are examined as opposed to the one site investigated in detail in Vardanega et al. (2012a). The EC7 UK NA employs design approach 1, split into the two partial factor sets, DA1-1 and DA1-2. DA1-2 (shown in Table 2) was found to be the governing case for all piles studied herein.

Table 1. Test site details (ML = Maintained load test, CRP = Constant rate of penetration test, CFA = continuous flight auger, see Fig.2 for other symbols)

Site ID	No. Piles	Pile Type	L (m)	D (m)	Max. P_U (MN)	Test type	Max. P_{Max} (MN)	Max. δ_{Max} (mm)	Data source
R01_01	3	Bored, CFA	15-16	0.40	1.57	ML	2.29	37	Baxter & Hadley (2006), Baxter (2009)
R01_02	1	Bored	18	0.40	1.56	ML	2.04	10	Baxter (2009)
R09	3	Driven	5	0.17	0.09	ML	0.13	2	Cooke et al. (1979)
R14	7	Bored	13	0.75	2.08	ML, CRP	1.27	14	Faerenside & Cooke (1978)
R17	2	Bored	7-11	0.61	1.38	ML	1.29	50	Golder & Leanard (1954)
R18	8	Bored	3	0.30-0.36	0.22	ML	0.24	152	Green (1961)
R22	9	Micropile	9-19	0.15	0.76	ML, CRP	0.94	17	Jones & Turner (1980)
R31	1	Bored	38	0.75	7.53	ML	10.0	121	Martin et al. (2016)
R32	2	Bored	27	1.20	8.71	ML	9.00	90	McNamara et al. (2014)
R33_01	6	Bored	6-12	0.30-0.36	1.17	ML	0.69	199	Meyerhof & Murdock (1953)
R33_02	3	Bored, Driven	4-9	0.30-0.36	1.01	ML	0.96	150	Meyerhof & Murdock (1953)
R37_01	2	Bored	7-10	0.36	0.72	ML, CRP	0.89	9	Patel (1989)
R37_02	1	Bored	8	0.46	0.52	ML, CRP	0.66	8	Patel (1989)
R37_03	1	Bored	6	0.37	0.46	ML, CRP	0.68	17	Patel (1989)
R37_04	1	Bored	16	0.44	1.14	ML, CRP	1.73	5	Patel (1989)
R37_05	2	Bored	15-16	0.61-0.76	5.84	ML, CRP	4.18	19	Patel (1989)
R37_06	12	Bored	7-15	0.61-0.76	2.91	ML, CRP	2.39	38	Patel (1989)
R37_07	2	Bored	15	0.76	6.41	ML, CRP	5.15	69	Patel (1989)
R37_08	2	Bored	19-20	0.61	2.66	ML, CRP	2.83	31	Patel (1989)
R37_09	6	Bored	9-14	0.45	1.13	ML, CRP	1.13	40	Patel (1989)
R37_10	5	Bored	18-24	0.61	4.27	ML, CRP	4.47	22	Patel (1989)
R37_11	2	Bored	17-19	0.46	1.61	ML, CRP	1.70	18	Patel (1989)
R37_12	2	Bored	13-15	0.91	5.43	ML, CRP	4.97	74	Patel (1989)
R37_13	2	Bored	9-15	0.61	1.8	ML, CRP	2.51	16	Patel (1989)
R37_14	3	Bored	15-20	0.46-0.61	3.25	ML, CRP	2.75	19	Patel (1989)
R37_15	3	Bored	15-20	0.61-0.76	3.35	ML, CRP	4.43	21	Patel (1989)
R38	4	Bored	6-10	0.30	0.49	ML	0.45	10	Powel & Skinner (2013)
R44	1	Bored	39	1.05	9.52	CRP	12.8	150	Unwin & Jessep (2004)
R45	11	Bored	9-16	0.62-0.94	7.22	ML, CRP	6.97	306	Whitaker & Cooke (1966)
R46	1	Bored	12	0.75	1.87	ML, CRP	1.31	16	Whitworth et al. (1993)

Table 2. Partial factors (notation as in Eqs. 3 and 4)

Partial Factor	AS2159	EC7 UK NA DA1-2
β_1	1.2*	1
β_2	1.5*	1.3
β_3	1	1
β_4	1	1
β_5	1	1.6
β_6	1	2.0
β_7	$1/\phi_{gb}$	1.4

*the governing case for the load combination considered is shown

Table 3. AS2159 IRR values used (based on Vardanega et al. 2012a)

Risk factor		Weight	IRR
Site	Geological complexity of site	2	2
	Extent of ground investigation	2	2
	Amount and quality of geotechnical data	2	3
Design	Experience with similar foundations in similar geological conditions	1	2
	Method of assessment of geotechnical parameters for design	2	4
	Design method adopted	1	3
	Methods of utilizing results of in situ test data and installation data	2	2
Installation	Level of construction control	2	3
	Level of monitoring	0.5	3

4 RESULTS

Both the EC7 UK NA and AS2159 calculations were performed for each of the piles in the dataset and the maximum design loads (Q_D) obtained. A variable load of 20% of the total was assumed for all calculations (i.e. $V = 0.25G$). A design working load (P_W) was then obtained by summing the unfactored permanent and variable loads. This is the load that a pile designed according to each code is expected to routinely resist throughout its design life. An unfactored analysis (all partial factors set to 1) was also performed to obtain the predicted capacity of the α method (P_U).

The ratio of these two loads (P_U / P_W) is the effective global factor of safety, the bulk factor of safety that would be required to reduce P_U to P_W. AS2159 resulted in an effective global factor of safety of 2.42 for all the piles in the database, while EC7 UK NA resulted in values ranging from 2.37 to 2.77 (depending on the ratio of shaft to base resistance), with a mean of 2.52. These results indicate that for the sites considered, both codes would result in very similar piles being designed. A similar result is obtained in Vardanega et al. (2012a).

For each pile considered, the measured settlement at P_W (δ_W) and P_U (δ_U) was recorded along with the maximum test load (P_{Max}) and the maximum test settlement (δ_{Max}), as illustrated in Figure 2. If P_{Max} was less than the predicted value, P_U, the settlement generated by an unfactored design load was interpreted as δ_{Max}. Otherwise it was interpreted as δ_U directly, the measured value at P_U. Of the 108 records, 97 have settlement readings at working load and 52 of these have settlement readings at unfactored ultimate load. Figure 3 shows the load-settlement values for the dataset with and without the code factors applied. A wide range of settlements can be observed, although most factored working loads result in settlements less than 10mm.

As expected, reducing the working load by applying the relevant design code factors results in lower settlements. This is quantified in Fig. 4, where the percentage reduction in settlement (S_R, defined in Fig. 2) due to reducing the applied load from the predicted capacity (i.e., unfactored design load) to the design working load is shown. The tests in the database were carried out for different purposes; therefore, it is not known if they were carried out to failure. As the definition of failure is open to interpretation, where a load-test was not carried out to the predicted capacity (45 tests), the maximum test load and settlement were substituted instead.

Were the pile to behave elastically, the settlement reduction would equal the reduction in load, which is easily calculated from the effective global factor of safety and shown on Fig. 4 for each design code. As the expected behaviour is non-linear (and soil stiffness generally decreases with strain), S_R should be larger than this value. This is the case for all piles where $P_{Max} \geq P_U$ and a δ_U value was available. However, for some of the piles where δ_{Max} was employed this is not the case; therefore, it is likely that these tests were not carried out to failure. Neglecting these results, for the majority of the remaining piles applying a design code results in a reduction in settlement of over 75% from the observed settlement generated by an unfactored design load (although there is a large variation in values).

Randolph (2003, p.848) stated that engineers "… may never be able to estimate axial pile capacity in many soil types more accurately than about ±30%". These bounds are shown on Fig. 4. Assuming the most of the remaining P_{Max} values are at failure, the majority of the predicted capacities (using the α method) are within this range; however, evidently the aforementioned statement is correct for this dataset and a more appropriate prediction range is around ±50%.

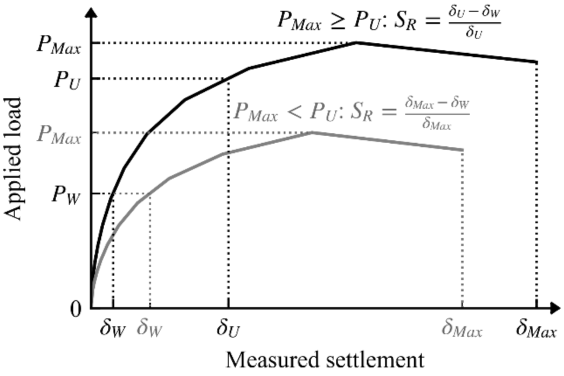

Figure 2. Idealised pile test result with S_R calculation

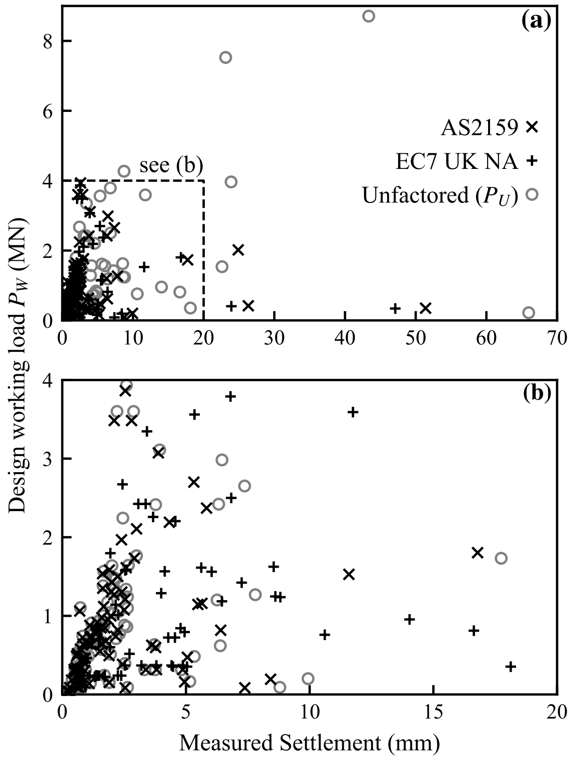

Figure 3. Measured settlement at predicted load

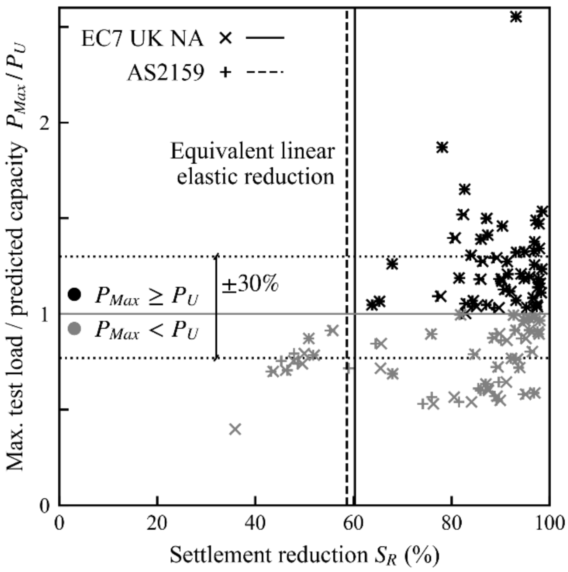

Figure 4. Measured settlement reduction due to applied factor set

5 DISCUSSION

Evidently from Fig. 4, implementing the chosen design codes significantly reduces the settlement of piled foundations. It is expected that for routine problems this settlement will be 'acceptable' and 'within safe limits' according to the uses of design codes stated by Simpson (2000, p.2) and Skempton (1959, p.157), respectively (see Section 1.2). In fact, EC7 does state (BSI 2004, cl. 7.6.4.1)

> "… For piles bearing in medium-to-dense soils and for tension piles, the safety requirements for the ultimate limit state design are normally sufficient to prevent a serviceability limit state in the supported structure."

However, as evidenced by Fig. 3, a wide range of settlement values are obtained. Therefore, when rigorous settlement criteria are required for a sensitive structure, a settlement prediction method is required (e.g., Crispin et al. 2018, 2019; Vardanega et al. 2012b). EC7 (BSI 2004) for example does specify that this limit state should be considered; however, no specific guidance as to how this should be conducted is provided. In addition, if (as stated) the ultimate limit state requirements normally ensure the serviceability limit state requirements are met, and serviceability is a key goal of the chosen partial factors (Simpson 2000 and Skempton 1959), these design approaches are likely quite conservative.

It is noted that partial factors and geotechnical reduction factors have other uses in practice other than settlement reduction. Simpson et al. (1981, p.21) notes

> "No margin of safety which is economically acceptable can be expected to cover situations in which the geology or material properties have been completely misunderstood, a major source of load has been ignored, an inappropriate calculation has been performed, or a decimal point has been misplaced."

Indeed, the 'social' sources of risk (cf. McMahon 1985, Vardanega & Bolton 2016) are unlikely to be consistently dealt with merely by applying codified design rules.

It should also be noted that knowledge of the site investigation quality and extent is often limited when using the DINGO database and therefore it is acknowledged that on individual sites different outcomes may be obtained by 'design by calculation' if more statistically robust site investigation procedures are used (see e.g., Jaksa et al. 2005; Goldsworthy et al. 2007 and Crisp et al. 2019). In addition, it is unknown exactly how conservative the site c_u values are (see Voyagaki et al. 2021 for more details). Therefore, it is not known if the c_u values are the average, the characteristic values (as required in EC7; BSI 2004) or the 'worst-credible' values (cf. Bolton 1989). If this information were available, the analysis results presented here may change.

6 CONCLUSIONS

For a dataset of pile load tests in London clay (sourced from the DINGO database) the effect of two codified design approaches on settlement is investigated. It is shown that both approaches (which imply a similar global factor of safety) do reduce settlements as expected, generally by over 75% from an unfactored approach. It is argued that excessive deformations do not usually occur for well-designed piled foundations in London clay. Studies such as that presented in this paper provide code drafters with valuable information to better evaluate the intrinsic levels of conservatism, and therefore sustainability, built into codified design processes.

7 DATA AVAILABILITY STATEMENT

The DINGO database 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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