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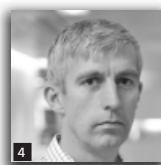
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**Bored pile design in stiff clay I: codes of practice**

Vardanega, Kolody, Pennington, Morrison and Simpson

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# Bored pile design in stiff clay I: codes of practice

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The assessment of the allowable bearing load of bored piles ‘floating’ in stiff clay is a standard engineering task. Although the soil mechanics is universal, engineers designing structures in different parts of the world will need to take into account the pertinent codes of practice. It will be helpful to compare such codes, especially in relation to their treatment of uncertainty in the design of bored piles. This paper presents a series of design calculations for a real set of geotechnical data using four international codes of practice: the Australian, American, European and Russian codes. The National Annexes of Ireland, the Netherlands and the UK are used in conjunction with the European code. This selection of countries covers the three Eurocode 7 design approaches (DA1, DA2 and DA3). A non-codified design method is used to provide a base case for comparative purposes with the six codified calculations. A companion paper investigates the issues of soil mechanics in pile design methods, uncertainty in soil parameters and settlement criteria.

## Notation

$A_b$	area of pile base
$c_u$	undrained shear strength (kPa)
$D$	pile diameter (m)
$E_d$	design value of effect of actions
$f_s$	unit skin friction (kPa)
$G$	unfactored permanent load
$I_L$	liquidity index
$I_P$	plasticity index
$K$	testing benefit factor
$L$	length of pile in clay stratum (m)
$N_c$	bearing capacity factor
$Q_b$	base resistance of the pile (kN)
$Q_d$	design load of a bored pile (kN)
$Q_s$	shaft resistance of the pile (kN)
$Q_T$	geotechnical design resistance of a pile
$Q_{work}$	$G + V =$ working load (kN)
$R_d$	design value of resistance
$R_{d,g}$	design geotechnical strength
$R_{d,ug}$	design ultimate geotechnical strength
$V$	unfactored variable load
$z$	depth below top of clay stratum (m)
$\alpha$	correlation factor between unit skin friction ( $f_s$ ) and undrained shear strength ( $c_u$ )
$\beta_1$	partial factor on permanent load

$\beta_2$	partial factor on variable load
$\beta_3$	partial factor on $c_u$ along pile shaft
$\beta_4$	partial factor on $c_u$ at pile base
$\beta_5$	partial factor on pile shaft resistance
$\beta_6$	partial factor on pile base resistance
$\beta_7$	partial factor on design resistance
$\gamma_{RD}$	model factor used in the UK National Annex to Eurocode 7
$\phi_g$	geotechnical reduction factor
$\phi_{gb}$	basic geotechnical reduction factor
$\phi_{tf}$	intrinsic test factor

## 1. Introduction

The design of piles in stiff over-consolidated clay is common in geotechnical engineering. The engineer uses judgement, experience, available site data and knowledge of soil mechanics to complete the design task and ensure designs are compliant with the code of practice in force in the relevant jurisdiction. In this paper the requirements of AS2159-2009 in Australia (Standards Australia, 2009), Eurocode 7 (BSI, 2010) in the European Union, the American Association of State Highway and Transportation Officials (AASHTO) load and resistance factor design (LRFD) bridge design specifications (AASHTO, 2007) in the USA and SNiP 2.02.03-85 (SNiP, 1985a) in Russia, are considered together with a simple lump factor of safety design method. The Eurocode

7 calculations are performed using three national annexes to show the effect of using each of the three design approaches in Eurocode 7. A key aim of the paper is to explore the different approaches to uncertainty and safety intrinsic in these codes, so that engineers may be better informed on how to achieve their customary safety standards when working with an unfamiliar code.

The design example in this paper is a single pile in stiff over-consolidated clay. The data are taken from a site in London. However, this review could equally be applied to other stiff over-consolidated clays such as the Keswick-Hindmarsh Clay of Adelaide, Old Bay Clay of San Francisco, Boom Clay of the Netherlands, Palaeogene Clay of Denmark or Voskrensky Clay of Moscow and so on. This problem can be tackled with varying degrees of rigour depending on the nature of the design project being completed and the design assumptions required. Only the collapse/ultimate limit state will be considered in this paper. Settlement/serviceability limit state considerations are examined in a companion paper.

## 2. Method of calculation

The design for ‘collapse’ or ‘ultimate’ limit states is based on undrained shear strength. Calculations based on effective stress parameters are considered in the companion paper. The ‘ $\alpha$  method’ for pile design is used to calculate unit skin friction ( $f_s$ )

$$1. \quad f_s = \alpha c_u$$

where  $f_s$  is the unit skin friction on the pile shaft and  $\alpha$  is an empirical adhesion co-efficient linking undrained shear strength to  $f_s$ . A common assumption of  $\alpha = 0.5$  was adopted for the calculations (e.g. Meyerhof (1976) and Tomlinson (1986) suggested 0.45 after Skempton (1959)). Patel (1992) suggested that in London Clay  $\alpha = 0.6$  (for constant rate of penetration (CRP) tests) was reasonable. Some codes of practice mandate values of  $\alpha$  but in London large amounts of available test data mean that  $\alpha = 0.5$  is commonly used and not unduly optimistic.  $c_u$  is the undrained shear strength of the clay (kPa).

For a clay deposit with a  $c_u$  value dependent on depth ( $z$ ) the pile shaft resistance is calculated using Equation 2

$$2. \quad Q_s = \pi D \alpha \int_0^L c_u dz$$

where  $D$  is pile diameter (m);  $c_u$  is undrained shear strength (kPa);  $\alpha$  is an empirical adhesion co-efficient;  $L$  is the length of pile in the clay stratum (m); and  $z$  is depth of clay stratum (m).

The base capacity in clays is generally determined using

$$3. \quad Q_b = A_b N_c c_u$$

where  $A_b$  is the area of the base ( $m^2$ );  $N_c$  is the bearing capacity factor, which varies depending on the sensitivity and deformation characteristics of the clay, but is generally taken as 9 (e.g. Meyerhof, 1976); and  $c_u$  is the undrained shear strength (kPa) at the base.

The geotechnical resistance ( $Q_T$ ) of a pile is determined using the following equation

$$4. \quad Q_T = Q_s + Q_b$$

## 3. General design formula

Partial factors can be applied at various stages in the calculation process. In limit state design these reflect the different sources of uncertainty. Equation 5 shows the general pile design formula with partial factors, denoted as  $\beta_1$  to  $\beta_7$ . In the codes of practice reviewed, various combinations of partial factors are used. No one approach utilises all the possible partial factors shown below. Therefore, some will be given a value of unity when the design approach does not specify a value for them. The ‘ $\beta$ ’ factors shown in Equation 5 all take a value greater than or equal to unity. Equation 5 could be re-written to make the factors less than unity by changing them from multipliers to divisors or vice-versa. As an example, using only one factor ( $\beta_7$ ) with a non-unity value would represent a design approach with a single overall factor of safety.

Since different codes use different terminologies and symbols for various quantities the ‘generic’ notation defined in Equation 5 will be used so that the different approaches can be easily compared. In this paper, the terminology of most recent codes will be adopted, in which the ‘design value’ of a parameter is one that incorporates margins or factors of safety. For an economic design the design load  $Q_d$  equals the design resistance. That is

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

where  $G$  is the unfactored permanent load;  $V$  is the unfactored variable load;  $\beta_1$  is the partial factor on permanent load ( $G$ );  $\beta_2$  is the partial factor on variable load ( $V$ );  $\beta_3$  is the partial factor on  $c_u$  along pile shaft;  $\beta_4$  is the partial factor on  $c_u$  at pile base;  $\beta_5$  is the partial factor on pile shaft resistance;  $\beta_6$  is the partial factor on pile base resistance; and  $\beta_7$  is the partial factor on combined shaft and base resistance

The term ‘partial factor’ is used for the ‘ $\beta$ ’ terms to include all types of factors used in the various codes (factor of safety, partial factor, model factor and so on).

In order to compare the different codes fairly a quantity  $Q_{work}$ , termed the working load, is defined

$$6. \quad Q_{work} = G + V$$

The value of  $Q_{work}$  includes no partial factors and  $G$  and  $V$  are unfactored loads.

#### 4. Design problem and site data

To illustrate how independently developed codes of practice affect the design of a single pile in clay, as well as the influence that different methods of analysis have on the resulting design, the following example is presented.

An engineer has been asked to determine the allowable working load ( $Q_{work}$ , defined as the combined unfactored permanent plus variable load) of the piles shown in Figure 1. In this paper, for simplicity, eccentricity of loading is not considered. The pile to be designed is a bored, straight-shafted, cast-in-place concrete pile, with no load testing carried out on the site. The variable load ( $V$ ) is assumed to be 0.25 times the permanent load ( $G$ ). This is a generic permanent to variable load ratio that has been taken to simulate a standard structure. Information based on Simpson *et al.* (1980) has been used to provide ground investigation data for the London Clay deposit. Data were collected from six boreholes with locations as shown on Figure 2. The Atterberg

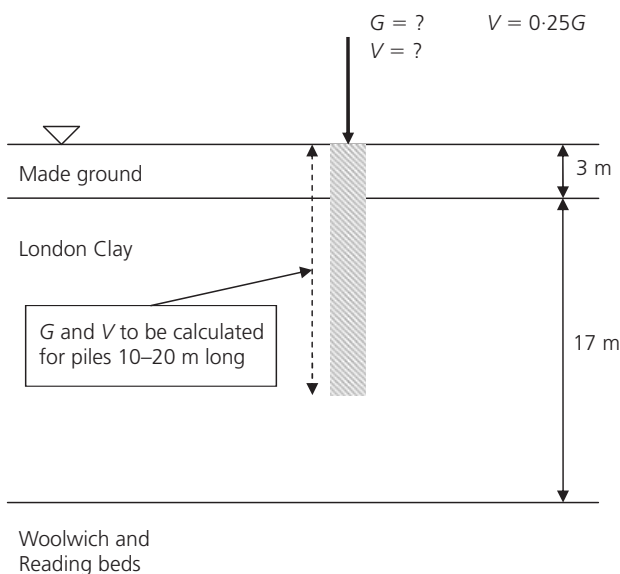


Figure 1. Idealised soil profile and pile to be designed

limits are summarised in Figure 3. Data from 102 mm unconsolidated, undrained (UU) triaxial tests (Figure 4) and correlated Standard Penetration Test (SPT) data (Figure 5) show the variation of undrained shear strength ( $c_u$ ) with depth in the clay.

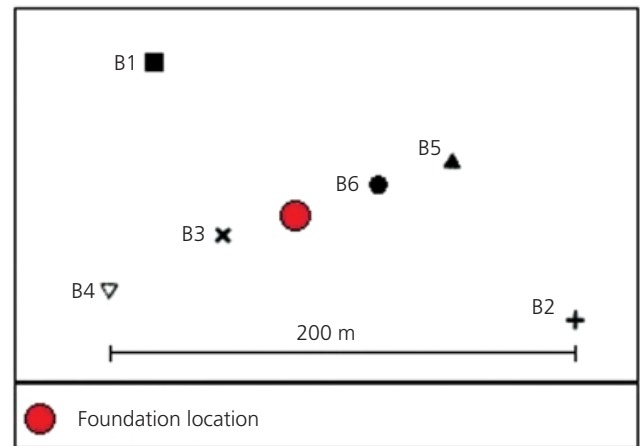


Figure 2. Location of foundation and boreholes (1 cm = 30 m)

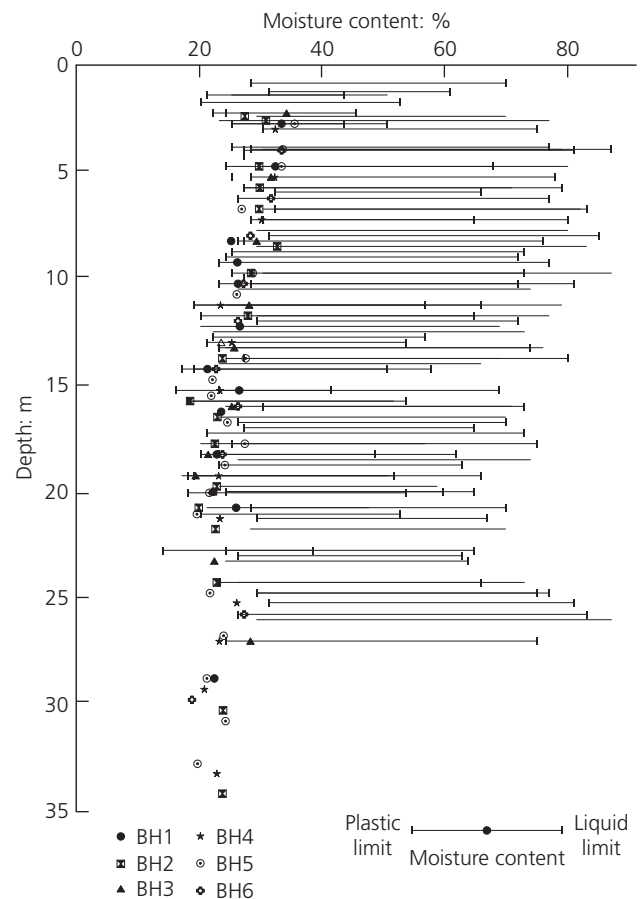


Figure 3. Site Atterberg limits

To convert the SPT  $N_{60}$  values to  $c_u$ , Equation 7 was used (see Figure 6)

$$7. \quad c_u = 4.4(N_{60})$$

Plasticity index ( $I_p$ ) varies on site from about 30% to 50% (Figure 3). Using the correlation from Figure 6 for  $I_p = 30\%$  gives a multiplier on  $N_{60}$  for  $c_u$  of about 4.7 and for  $I_p = 50\%$  a multiplier of 4.2. For an  $N_{60}$  equal to 40 the range in  $I_p$  values would correspond to a range of  $c_u$  from 168 kPa to 188 kPa as  $I_p$  decreases. An average  $I_p$  of 40% was adopted for the following analysis. Comments on Stroud (1974) with respect to the lack of statistical treatment have been made (Reid and Taylor, 2010). Vardanega and Bolton (2011) showed that a power curve, drawn

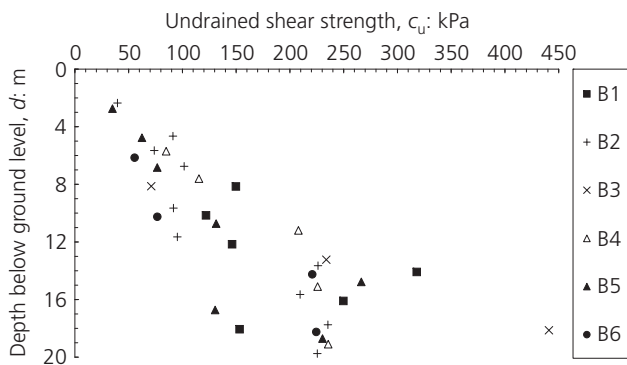


Figure 4. Results from triaxial tests

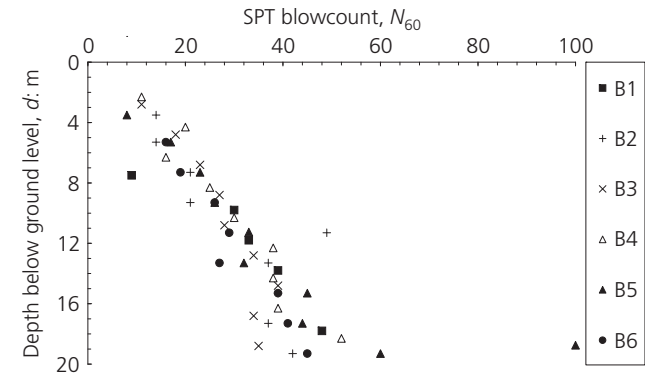


Figure 5. Results from SPT measurements

through Stroud's data (Figure 6) is a good statistical relationship that could be fitted to the data. The coefficient of determination ( $R^2$ ) of the regression line is 0.37 ( $R^2 = 0.37$ ). The regression curve is similar to Stroud's original line and the use of either curve results in Equation 7 for a plasticity index of 40%. There is a divergence between the two curves at low and high plasticity indices.

## 5. Undrained shear strength ( $c_u$ ) relationship with depth

Figure 7 shows the combined data from Figures 4 and 5 (converted SPTs and data from 102 mm UU triaxial tests), with linear regression lines through the undrained strength data of individual boreholes. The slope does not vary considerably for the six boreholes. The data points are not highly scattered with

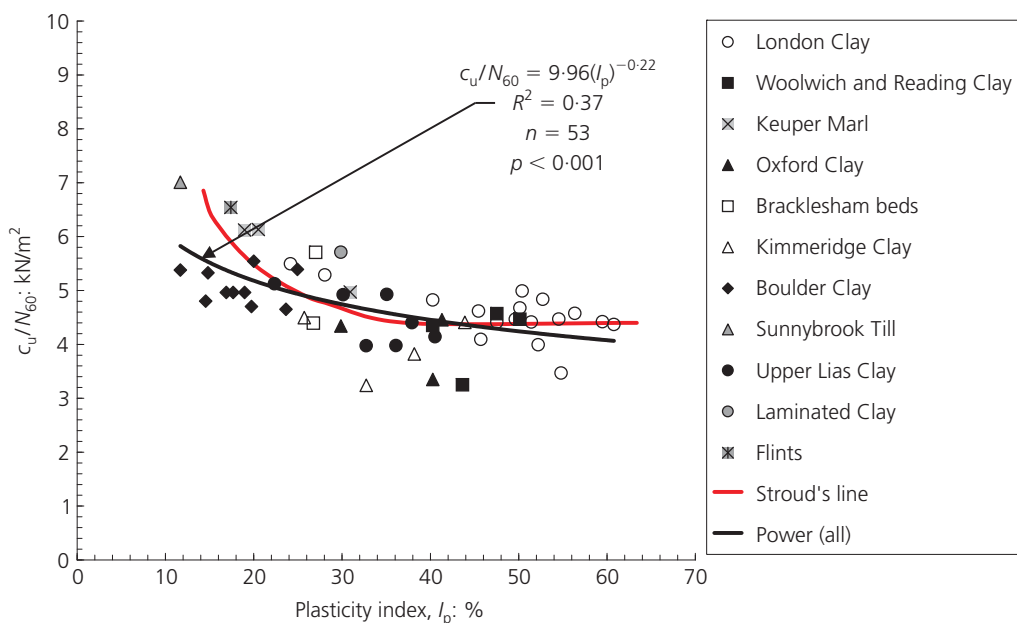


Figure 6. Relationship between  $c_u/N_{60}$  and  $I_p$  for a variety of clays (replotted from Stroud (1974) and Vardanega and Bolton (2011))

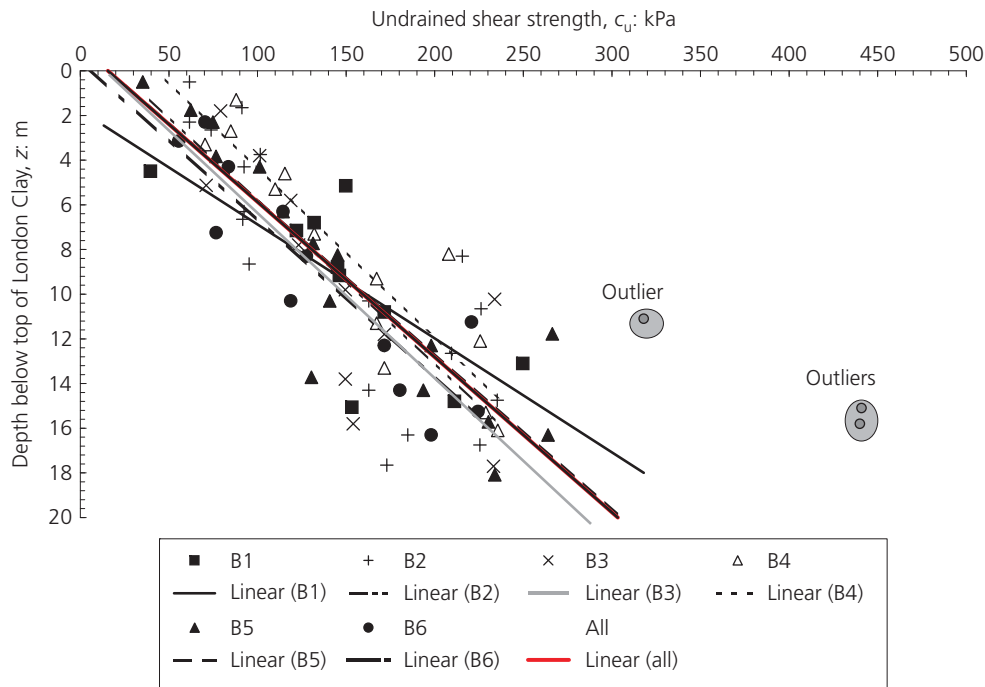


Figure 7. Linear regression lines to characterise  $c_u$  increase with depth (individual boreholes)

the  $R^2$  for the individual lines varying from about 0.55 to 0.83. This is not an unexpected characteristic of the London Clay which was deposited in quiet, deep water conditions and has locally had the same geological history of overburden and erosion. Vertical variation is much more likely on this scale, as original deposition conditions change with depth. For instance, locations 50 m apart horizontally (deposited at the same time) may be much more similar than locations 1 m apart vertically (deposited many years apart). Of course, there could be some slight rotation of the bedding, but not very much, and there are occasional anomalies such as faults and pingos.

Regression lines in Table 1 are unsuitable as a design line if they imply negative or unreasonable shear strength at the top of the clay. In a stiff, overconsolidated deposit the mere ability for people to ‘stand on the soil’ implies some shear strength is present. This geological fact means that a blind regression is not advised for the determination of the design  $c_u$  profile. Indeed there is no geological reason for a straight line to be used. The reason for adopting a straight line is that a single gradient can be easily defined, thus simplifying the computation of skin friction.

Many engineers could offer a variety of possible design lines/relationships to characterise the data shown in Figure 7. In this paper, it is assumed that the characteristic value or ‘cautious estimate’ described by the Eurocode is given by Equation 8a. The ‘representative value’ used in conventional design is given by

Borehole	$c_u$ relationship	$R^2$	$n$
B1	$c_u = 19.61z - 34.84$	0.545	10
B2	$c_u = 13.52z + 22.12$	0.679	19
B3	$c_u = 13.73z + 15.01$	0.603	11
B4	$c_u = 13.09z + 43.35$	0.831	13
B5	$c_u = 14.43z + 16.94$	0.809	16
B6	$c_u = 14.03z + 6.74$	0.815	12
All	$c_u = 14.39z + 15.50$	0.711	81

Table 1. Undrained shear strength relationships for each borehole (coefficient of determination,  $R^2$ , and number of data points,  $n$ , used in each regression shown)

Equation 8b and was derived by an eye fit to the data. Both lines are plotted on Figure 8. Equation 8a is drawn at the 25th percentile (of the total number of data points) parallel to the lower bound trace of the data (also shown of Figure 8). The lower bound trace is used to define the gradient of Equation 8a. This methodology for defining the gradient of the shear strength with depth works because there are no obvious outliers to the lower bound of the data set. The AASHTO and SNiP calculations make use of ‘average value’ soil parameters. In the AASHTO guide, clause 10.4.6.2.2 states ‘correlations for  $c_u$  based on SPT tests should be avoided’. Therefore, for the AASHTO calculations only

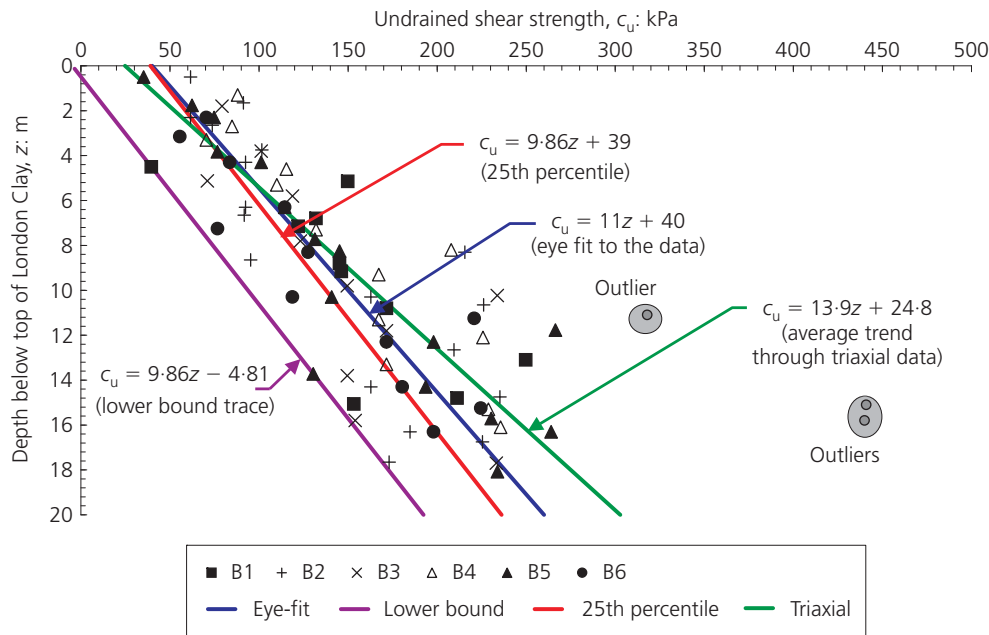


Figure 8. Relationships between  $c_u$  and depth used in this paper

the triaxial data were used to characterise the strength increase with depth; this is given by Equation 8c and the line shown on Figure 8. For SNiP, calculations are based on liquidity index and not  $c_u$ .

$$8a. \quad c_u = 39 + 9.86z \quad (\text{kPa})$$

$$8b. \quad c_u = 40 + 11z \quad (\text{kPa})$$

$$8c. \quad c_u = 24.8 + 13.9z \quad (\text{kPa})$$

The rationale for a line similar to Equation 8a is that it lies below the mean of the data and is a cautious estimate of strength and therefore a good choice for the ‘characteristic value’ that is required for determination of soil properties in Eurocode 7. It is acknowledged that a 5th percentile line could be used, but that this is an extremely conservative view of what is essentially a large amount of data (Simpson *et al.*, 2009).

## 6. Conventional design

For comparison with the codes considered in this paper a simple design method is presented as the base case. The design is based on a global factor of safety.

BS 8004 (clause 7.3.8) (BSI, 1986) states

in general, an appropriate factor of safety for a single pile would be between two and three. Low values within this range may be applied where the ultimate bearing capacity has been determined by a sufficient number of loading tests or where they may be justified by local experience; higher values should be used when there is less certainty of the value of the ultimate bearing capacity.

(BS 8004 has now been superseded by BS EN 1997-1:2004).

For the purpose of this example a value of 3.0 is adopted herein assuming that no pile load testing is carried out. For ‘conventional design calculations’ Equation 5 reduces to Equation 9

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

$$9. \quad Q_d = G + V = \left( \pi D \alpha \int_0^L c_u dz + A_b N_c c_u \right) / \beta_7$$

where  $\beta_7 = 3.0$ .

Calculations for a conventional design, for a 15 m long (12 m into the clay), 0.45 m diameter pile follows

$$10. \quad Q_d = G + V = \left( \pi D \alpha \int_0^L c_u dz + A_b N_c c_u \right) / 3.0$$

$$11. \quad Q_d = G + V = \left( \pi D (0.5) \int_0^{12} (40 + 11z) dz + A_b N_c c_u \right) / 3.0$$

( $\alpha = 0.5$  for bored piles in London).

$$12. \quad Q_d = G + V = (899.1 + 246.2) / 3.0$$

$$13. \quad Q_d = G + V = 381.8 \text{ kN}$$

In this case  $Q_d = Q_{work}$  as no factors are applied to the loads.

The split between  $G$  and  $V$  based on  $V$  being 25% of  $G$  returns values of

$$G = 305.4 \text{ kN}$$

$$V = 76.4 \text{ kN}$$

$$Q_{work} = 381.8 \text{ kN.}$$

## 7. AS2159-2009 (Australia)

The Australian approach to designing piles makes use of partial factors with loads being factored separately from the capacities. A single factor is applied to the calculated geotechnical resistance, termed the 'geotechnical reduction factor', applied to the calculated resistances, not the soil parameters.

AS2159-2009 (Standards Australia, 2009) directs the engineer to AS/NZS 1170.0 (Standards Australia, 2002) (structural design actions) for the load factors. The two relevant combinations for a pile are most likely to be the greater of:  $1.2G + 1.5V$  or  $1.35G$ . Since, for this design,  $V/G = 0.25$ , the critical case is  $1.2G + 1.5V$ . Since this paper is only considering collapse limit states, serviceability and actions induced by ground movements are not considered. Earthquake loading is also not considered.

Clause 4.3.1 of AS2159-2009 states that the design geotechnical strength ( $R_{d,g}$ ) must not be less than the design action effect ( $E_d$ ).

For a single pile, not a group,  $E_d$  will be taken as the load imparted from the pile cap to the individual pile. The code defines  $R_{d,g}$

$$14. \quad R_{d,g} = \phi_g R_{d,ug}$$

$R_{d,ug}$  is the design ultimate geotechnical strength, determined from site data and calculation methods;  $\phi_g$  is the geotechnical reduction factor (not to be confused with friction angle)

$$15. \quad \phi_g = \phi_{gb} + (\phi_{tf} - \phi_{gb})K \geq \phi_{gb}$$

$\phi_{tf}$  is the intrinsic test factor;  $K$  is the testing benefit factor; and  $\phi_{gb}$  is the basic geotechnical reduction factor.

In this example no load testing is being considered so  $\phi_g = \phi_{gb}$  as calculated in the next section. There is a testing benefit factor ( $K$ ) in the Australian code which allows  $\phi_g$  to be reduced if load testing is performed.  $K$  is determined using the percentage of piles statically or dynamically tested (see clause 4.3.1 of AS2159-2009).

### 7.1 Determination of basic geotechnical reduction factor

To determine the basic geotechnical reduction factors the individual risk ratings (IRRs) (Table 2) are assigned to each of the risk factors listed in Table 3. This approach to determine geotechnical reduction factors was explained in Poulos (2004).

$\phi_{gb}$  is determined from the average risk rating (ARR), calculated using Equation 16, and then using Table 4. Design of a single pile, not in a large group, is treated as a design with *low redundancy*.

$$16. \quad ARR = \sum(w_i IRR_i) / \sum w_i$$

$$16a. \quad ARR = 36.5 / 14.5 = 2.52$$

$$16b. \quad \phi_{gb} = 0.52 \quad (\text{low to moderate risk})$$

The Australian method gives more responsibility to the engineer

Risk level	Very low	Low	Moderate	High	Very high
Individual risk rating	1	2	3	4	5

Table 2. Individual risk rating (after T4.3.2(B) AS2159)



Risk factor	Weighting factor, $w_i$	Individual risk rating (IRR)	Comments
Site			
Geological complexity of site	2	2	Well-understood soil strata, London Clay is widely studied and lots of testing done on this site Low risk
Extent of ground investigation	2	2	Relatively deep boreholes with lots of test data down to pile depth proposed Low risk
Amount and quality of geotechnical data	2	3	Undrained triaxial data and SPTs taken Moderate risk
Design			
Experience with similar foundations in similar geological conditions	1	2	Bored piles in London Clay, very common Low risk
Method of assessment of geotechnical parameters for design	2	3	Combination on conventional laboratory triaxial testing and well-established site correlations on SPT data Moderate risk
Design method adopted	1	3	Simple empirical methods are being employed here but both are well calibrated for London Clay Moderate risk
Method of utilising results of in situ test data and installation data	2	2	Using the 25th percentile of the data to determine $c_u$ against depth relationship Low risk
Installation			
Level of construction control	2	3	Since only performing a desktop study, conventional construction processes will be used, limited degree of professional involvement. Moderate risk
Level of monitoring	0.5	3	Assume little long-term monitoring as this is a simple project Moderate risk

Note: The pile design shall include the risk circumstances for each individual risk category and consideration of all of the relevant site and construction factors (AS2159 T4.3.2(A)).

**Table 3.** Geotechnical risk factors, weightings and ratings

to determine the reduction factor on the geotechnical calculations. It bounds the value of  $\phi_{gb}$  between 0.67 and 0.4 for low-redundancy systems and between 0.76 and 0.47 for high-redundancy systems. For low-redundancy problems, this is akin to dividing the calculated resistances by 1.50 for very low risk and 2.5 for very high risk, as shown in the 'Equivalent  $\beta_7$ ' column in Table 4; that is, the 'partial factor' on the geotechnical resistance is between 1.5 and 2.5 with the loading being considered separately.

## 7.2 Design calculations

For design to AS2159-2009 Equation 5 reduces to Equation 17

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

$$17. \quad Q_d = \beta_1 G + \beta_2 V = \left( \pi D \alpha \int_0^L c_u dz + A_b N_c c_u \right) / \beta_7$$

Range of average risk rating (ARR)	Overall risk category	Low-redundancy systems		High-redundancy systems	
		$\phi_{gb}$	Equivalent $\beta_7$	$\phi_{gb}$	Equivalent $\beta_7$
ARR $\leq$ 1.5	Very low	0.67	1.50	0.76	1.32
1.5 < ARR $\leq$ 2.0	Very low to low	0.61	1.64	0.70	1.43
2.0 < ARR $\leq$ 2.5	Low	0.56	1.79	0.64	1.56
2.5 < ARR $\leq$ 3.0	Low to moderate	0.52	1.92	0.60	1.67
3.0 < ARR $\leq$ 3.5	Moderate	0.48	2.08	0.56	1.79
3.5 < ARR $\leq$ 4.0	Moderate to high	0.45	2.22	0.53	1.89
4.0 < ARR $\leq$ 4.5	High	0.42	2.38	0.50	2.00
ARR > 4.5	Very high	0.40	2.50	0.47	2.13

**Table 4.** Basic geotechnical strength reduction factor for average risk rating

where

$\beta_1$ , partial factor on permanent load = 1.2 (AS1170)

$\beta_2$ , partial factor on variable load = 1.5 (AS1170)

$\beta_7 = 1/\phi_g = 1.92$ .

For a 15 m long (12 m into the clay), 0.45 m diameter pile

$$Q_d = \beta_1 G + \beta_2 V$$

$$= \left[ \pi D \alpha \int_0^{12} (9.86z + 39) dz + A_b N_c (9.86 \times 12 + 39) \right] / \beta_7$$

18.

$$19. \quad Q_d = 1.2G + 1.5V = (832.6 + 225.2)/1.92$$

Taking

$$V = 0.25G$$

$$G = 349.8 \text{ kN}$$

$$V = 87.5 \text{ kN}$$

$$Q_{work} = 437.3 \text{ kN.}$$

$$\text{The equivalent overall FOS} = (832.6 + 225.2)/437.3 = 2.42$$

## 8. Introduction to Eurocode 7

Eurocode 7 (EN 1997-1:2004 (BSI, 2010)) is a limit state code which employs partial factors. After checking the relevant limit states, the designer must ensure that the design value of the effect of actions,  $E_d$ , (the design loads) is less than or equal to the design value of the resistance,  $R_d$  (the design capacity)

$$20. \quad E_d \leq R_d$$

approaches: DA1, DA2 and DA3. Partial factors can be applied to the actions 'A' (i.e. the loads), the material properties 'M' (e.g. soil strengths) and the resistances 'R' (e.g. skin friction). Different design approaches use different combinations of partial factors. In order for the code to be used within a particular country, the national standards body of that country is required to produce a national annex (NA). The NA will specify which design approach(es) is/are permitted for construction in that country, and specifies the values of the partial factors to be used. In order to demonstrate the use of each design approach, three countries have been selected on the basis of their NA choice: the UK for DA1, Ireland for DA2 and the Netherlands for DA3.

## 9. EC 7 – design approach 1 (UK national approach)

### 9.1 Partial factors

This design approach is the one adopted by the UK NA to Eurocode 7 (BSI, 2007). In this design approach two sets of calculations are performed (DA1-1 and DA1-2), with the partial factors shown in Tables 5 and 6.

### 9.2 Model factor

Paragraph 2.4.1(8) of Eurocode 7 states: 'If needed, a modification of the results from the model may be used to ensure that the design calculation is either accurate or errs on the side of safety.' Paragraph 2.4.1 (9) states

Description	Partial factor	$\beta$ term
Variable load	1.5	$\beta_2$
Permanent load	1.35	$\beta_1$
Skin friction	1.0	$\beta_5$
Base resistance	1.0	$\beta_6$

**Table 5.** DA1-1 partial factors used

For design by calculation, Eurocode 7 presents three design

Description	Partial factor	$\beta$ term
Variable load	1.3	$\beta_2$
Permanent load	1.0	$\beta_1$
Skin friction	1.5 (driven piles)	$\beta_5$
	1.6 (bored piles)	
Base resistance	1.7 (driven piles)	$\beta_6$
	2.0 (bored piles)	

Note: Partial factors on resistances can be reduced with explicit verification of serviceability limit state (not applicable for this example).

Table 6. DA1-2 partial factors used

if the modification of the results makes use of a model factor, it should take account of: the range of uncertainty in the results of the method of analysis; any systematic errors known to be associated with the method of analysis.

The UK NA introduces a model factor termed  $\gamma_{Rd}$ . In this example it is applied to the calculated shaft and base resistances to account for the fact that the analysis model is empirically based. The UK NA requires a value of 1.4 (which would be reduced to 1.2 if there were load testing). This term is represented in Equation 6 at the  $\beta_7$  term; for more information on pile design to Eurocode 7 see Bond and Simpson (2010).

### 9.3 Design calculations

For a DA1-1 calculation Equation 5 reduces to Equation 25 and for a DA1-2 calculation Equation 5 reduces to Equation 22, assuming that no load testing is carried out

$$Q_d = \beta_1 G + \beta_2 V$$

$$5. \quad = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

DA1-1; terms  $\beta_3$ ,  $\beta_4$ ,  $\beta_5$  and  $\beta_6$  are equal to unity and have been omitted

$$21. \quad Q_d = 1.35G + 1.5V = \left[ \pi D \alpha \int_0^L c_u dz + A_b N_c c_u \right] / 1.4$$

DA1-2; terms  $\beta_1$ ,  $\beta_3$  and  $\beta_4$  are equal to unity and have been omitted

$$22. \quad Q_d = G + 1.3V = \left[ \frac{\pi D \alpha \int_0^L c_u dz}{1.6} + \frac{A_b N_c c_u}{2.0} \right] / 1.4$$

For the 15 m pile (12 m into the clay) of 0.45 m diameter DA1-1

$$23. \quad Q_d = 1.35G + 1.5(0.25G) = \left[ \frac{832.6}{1.0} + \frac{225.2}{1.0} \right] / 1.4$$

$$24. \quad 1.725G = [832.6 + 225.2] / 1.4$$

$$G = 438.0 \text{ kN}$$

$$V = 109.5 \text{ kN}$$

$$Q_{work} = 547.5 \text{ kN}$$

The equivalent factor of safety is  $1057.8/547.5 = 1.93$ .

DA1-2 (governs)

$$25. \quad Q_d = 1.0G + 1.3(0.25G) = \left[ \frac{832.6}{1.6} + \frac{225.2}{2.0} \right] / 1.4$$

$$26. \quad 1.325G = (832.6/1.6 + 225.2/2.0) / 1.4$$

$$G = 341.2 \text{ kN}$$

$$V = 85.3 \text{ kN}$$

$$Q_{work} = 426.5 \text{ kN}$$

The equivalent factor of safety is  $1057.8/426.5 = 2.48$ .

## 10. EC 7 – design approach 2 (Irish national annex)

To demonstrate the use of DA2 for the calculation of pile load carrying capacity, the Irish NA (NSAI, 2005) has been selected. The Irish NA is unique in that it allows for any of the three design approaches to be used for geotechnical works.

### 10.1 Design parameters

Table 7 presents the parameters to be used for the Irish adoption of DA2.

### 10.2 Design calculation

Therefore, for DA2 design to the Irish NA Equation 5 reduces to Equation 27

Description	Partial factor	$\beta$ term
Variable load	1.35	$\beta_2$
Permanent load	1.5	$\beta_1$
Skin friction	1.1	$\beta_5$
Base resistance	1.1	$\beta_6$

Table 7. DA2 parameters

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

$$27. \quad Q_d = \beta_1 G + \beta_2 V = \left( \frac{\pi D \alpha \int_0^L c_u dz}{\beta_5} + \frac{A_b N_c c_u}{\beta_6} \right) / \beta_7$$

NA.2.19 in the Irish NA requires a model factor of 1.75 to be applied to the shaft and base resistance factors for pile design.

$$28. \quad Q_d = 1.35G + 1.5V = \left( \frac{\pi D \times 0.5 \int_0^L c_u dz}{1.1} + \frac{A_b N_c c_u}{1.1} \right) / 1.75$$

By way of example, for a 15 m long (12 m into the stiff clay), 0.45 m diameter pile with  $V$  assumed to be 0.25 $G$  this reduces to

$$29. \quad Q_d = 1.35G + 1.5(0.25G) = \left( \frac{832.6}{1.1} + \frac{225.2}{1.1} \right) / 1.75$$

$$G = 318.6 \text{ kN}$$

$$V = 79.6 \text{ kN}$$

$$Q_{\text{work}} = 398.2 \text{ kN}$$

$$\text{The equivalent FOS} = 1057.8 / 398.2 = 2.66$$

### 11. EC 7 – design approach 3 (Netherlands national annex)

The Dutch use design approach 3 (NEN, 2007). The partial factors are applied to the soil strength and the base and shaft resistances ( $\beta_5$  and  $\beta_6$ ).

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

$$30. \quad Q_d = 1.0G + 1.0V = \left[ \frac{\pi D \alpha \int_0^L (c_u / 1.35) dz}{1.8} + \frac{A_b N_c (c_u / 1.35)}{1.8} \right]$$

#### 11.1 Design calculation

For a 15 m long pile (12 m into the clay) with  $\alpha = 0.5$  and 0.45 m diameter, where  $V = 0.25G$

$$31. \quad Q_d = G + (0.25G) = \left( \frac{832.6 / 1.35}{1.8} + \frac{225.2 / 1.35}{1.8} \right)$$

$$G = 348.2 \text{ kN}$$

$$V = 87.1 \text{ kN}$$

$$Q_{\text{work}} = 435.3 \text{ kN}$$

$$\text{The equivalent FOS} = (1057.8 / 435.3) = 2.43$$

### 12. American Association of State Highway and Transportation Officials (AASHTO)

The AASHTO bridge design specification (4th edition, AASHTO, 2007) specification adopts a limit state approach known in the USA as LRFD. This can be represented as follows

$$32. \quad \sum \eta_i \gamma_i Q_i \leq \phi_{qp} R_p + \phi_{qs} R_s$$

The  $c_u$  relation with depth used in the AASHTO method is a mean value of triaxial data only, Equation 8c. Table 8 defines the parameters used in the AASHTO method and compares the notation used in AASHTO with that in the present paper.

For the design calculation according to AASHTO, Equation 5 reduces to Equation 33

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u / \beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u / \beta_4)}{\beta_6} \right] / \beta_7$$

AASHTO notation	Notation in current paper	Description	Value
$Q_{\text{live}}$	$V$	Variable load	To be calculated
$Q_{\text{permanent}}$	$G$	Permanent load	To be calculated
$\eta_i$	$\eta_i$	Reliability factor	1.0
$\gamma_i$	$\beta_1$	Factor on permanent load	1.25
$\gamma_i$	$\beta_2$	Factor on variable load	1.75
$\phi_{\text{qp}}$	$1/\beta_6$	Reduction factor on base resistance	0.4
$\phi_{\text{qs}}$	$1/\beta_5$	Reduction factor on shaft resistance	0.45
$c_u$	$c_u$	Undrained shear strength	$13.9z + 24.8$ (see Figure 8)
$Z$	$z$	Depth	Varies
$D$	$D$	Pile diameter	Varies
—	$1/\beta_7$	Reduction factor on resistance	0.8
$\alpha$	$\alpha$	Adhesion factor	0.55 for $c_u/p_a \leq 1.5$ $0.55 - 0.1(c_u/p_a - 1.5)$ for $1.5 \leq c_u/p_a \leq 2.5$
$N_c$	$N_c$	Bearing capacity factor	9
$R_s$	$Q_s$	Shaft resistance	To be calculated
$R_p$	$Q_b$	Base resistance	To be calculated
$A_s$	$A_s$	Shaft area	To be calculated
$A_p$	$A_b$	Base area	To be calculated

Note 1: The value of  $\eta_i$  represents a conventional design, a conventional level of redundancy and a typical structure.

Note 2: The values for  $\gamma_i$  are for the Strength I load combination.

Note 3: The value of  $\alpha$  is zero for the top 1.52 m (5 ft) and bottom one diameter.

Note 4: The value of  $1/\beta_7$  applies to isolated piles.

Note 5: The value of  $N_c = 6(1 + 0.2(z/D)) \leq 9$

Table 8. AASHTO parameters

$$33. \quad Q_d = \beta_1 G + \beta_2 V = \left( \frac{\pi D \alpha \int_0^L c_u dz}{\beta_5} + \frac{A_b N_c c_u}{\beta_6} \right) / \beta_7$$

$\beta_1$ , partial factor on permanent load; for AASHTO = 1.25

$\beta_2$ , partial factor on variable load; for AASHTO = 1.75

$\beta_5$ , partial factor on pile shaft resistance; for AASHTO

$\beta_5 = 1/0.45 = 2.22$

$\beta_6$ , partial factor on pile base resistance; for AASHTO

$\beta_6 = 1/0.40 = 2.5$

$\beta_7$ , partial factor on design resistance; for AASHTO

$\beta_7 = 1/0.8 = 1.25$

$$34. \quad Q_d = 1.25G + 1.75V = \left[ \frac{\pi D \alpha \int_0^L (24.8 + 13.9z) dz}{2.22} + \frac{A_b N_c c_u}{2.5} \right] / 1.25$$

### 12.1 Design calculation

In the example calculation  $\alpha = 0.5$ . AASHTO suggest a value of  $\alpha$  that decreases with  $c_u/p_a$  which can be interpreted as an increase with depth. The value of  $\alpha$  as suggested by AASHTO is used to compute the AASHTO capacities in the summary in Section 14, Figures 10 and 11.

For a 15 m long (12 m into the stiff clay), 0.45 m diameter pile

$$35. \quad Q_d = 1.6875G = \left( \frac{917.8}{2.22} + \frac{274.3}{2.5} \right) / 1.25$$

$G = 248.0$  kN

$V = 62.0$  kN

$Q_{\text{work}} = 310.0$  kN

The equivalent FOS =  $(1192.1/310) = 3.85$ .

If the shaft and base resistances calculated using the 25th percentile of soil data are compared with the factored capacities here then the equivalent FOS =  $(1057.8/310) = 3.41$ .

### 13. SNiP (Russian approach)

The Russian design method for pile capacity is outlined in SNiP 2.02.03-85 (SNiP, 1985a). The method of determining bearing capacity is based on relating pile capacity (shaft and end bearing) to liquidity index ( $I_L$ ) for fine-grained soils and to density and grain size for coarse-grained soils. The minimum liquidity index allowed in the SNiP is 0.2 for the skin resistance and 0.0 for the base resistance; these are higher than the site data would suggest (Figure 9), so use of these values will provide a lower bound result. Values for shaft adhesion as a function of liquidity index,

taken from Table 2 of SNiP and values for base resistance as a function of liquidity index, taken from Table 7 of SNiP are shown as charts in Figures 13 and 14 in the Appendix.

Bearing capacity of a bored pile can be calculated using Equation 36, for which Table 9 gives a full explanation of the terminology

$$36. F_d = \gamma_c \left( \gamma_{cR} RA + u \gamma_{cf} \sum f_i h_i \right)$$

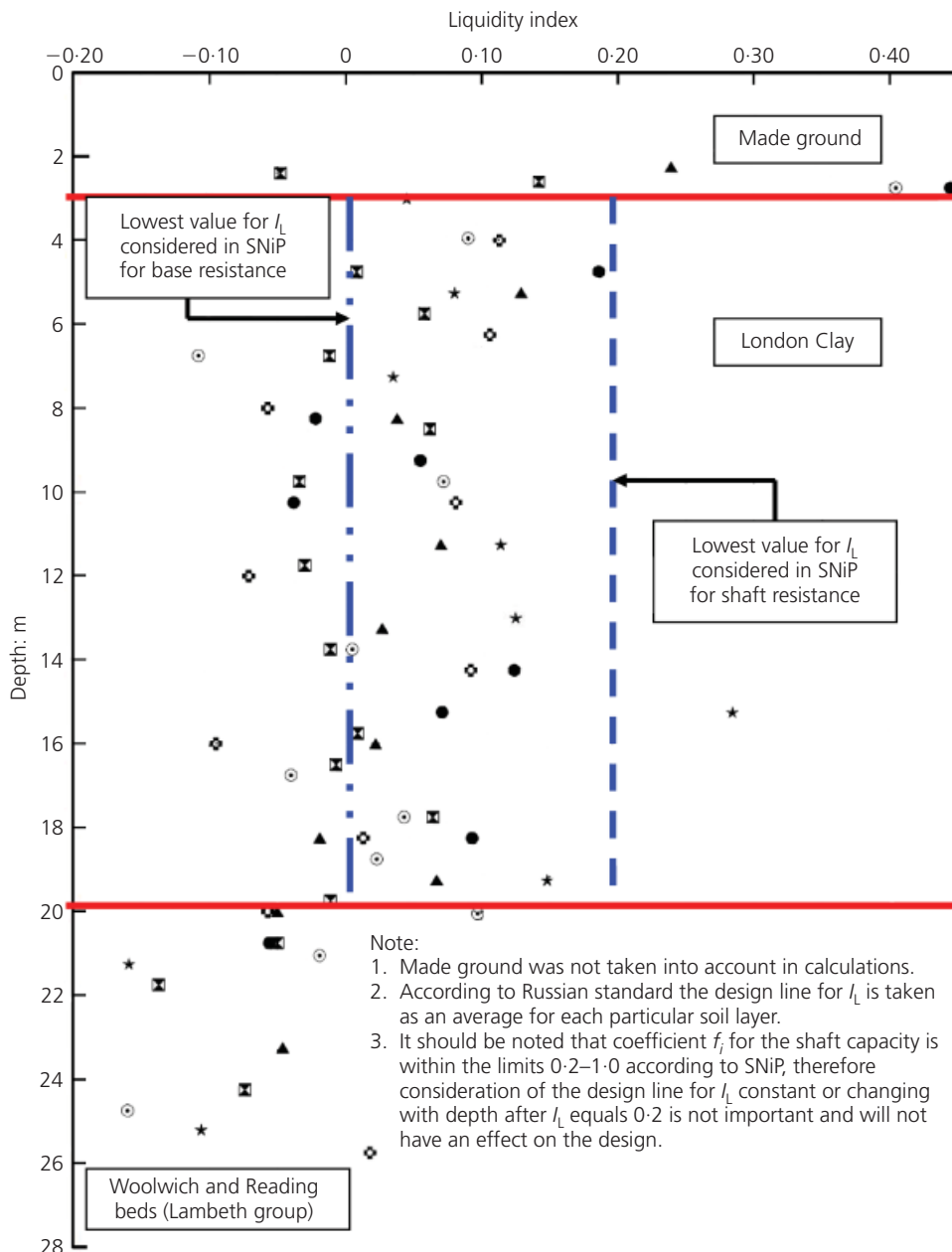


Figure 9. Design line through liquidity index data (SNiP calculation) (see Figure 3 for key)

SniP 2.02.03-85 notation	Equivalent notation in current paper	Notes
$F_d$	$Q_b + Q_s$	Bearing capacity, $F_d = \gamma_c(\gamma_{cR}RA + u\gamma_{cf}\sum f_i h_i)$
$\gamma_c$		Service factor for pile work in soil. If pile toe is in a soil with saturation degree of $< 0.90$ or in loose soils, then $\gamma_c = 0.80$ . In all other cases $\gamma_c = 1$ . Taken as $\gamma_c = 1$ for the current paper
$\gamma_{cR}$		Service factor for soil beneath the bottom end of the pile; taken as for bored pile without under-reaming $\gamma_{cR} = 1$
$R$		Design resistance of soil at the pile tip level, kPa, accepted according to Table 7 SNiP 2.02.03-85, depending on liquidity index ( $I_L$ ) and embedment depth of the base of the pile (design chart is shown in the Appendix, Figure 14).
$A$		Cross-sectional area of pile base: $m^2$
$u$		Perimeter of the pile shaft: m
$\gamma_{cf}$	Similar to $\alpha$	The service factor for soil on the surface of the shaft, depending on pile installation method $\gamma_{cf} = 0.6$ (Table 5, SNiP) – this is similar to the adhesion factor $\alpha$
$f_i$		The design resistance of $i$ th layer of soil on the surface of the shaft on driven and rotary bored piles, kPa, taken from Table 2 (SNiP 2.02.03-85). Depends on type of soils, liquidity index $I_L$ and average depth of soil stratum location (design chart is shown in the Appendix, Figure 13)
$h_i$		The thickness of $i$ th layer of soil, contacting the pile shaft: m

**Table 9.** Description of terms in SNiP 2.02.03-85 ‘Pile foundations’ design equation

The factored pile resistance should be taken based on the condition

$$37. \quad N = F_d/\gamma_k$$

$\gamma_k$  – factor of safety = 1.4 (see SNiP 2.02.03-85, item 3.10)

For standard buildings the typical partial factors on variable ( $V$ ) and permanent loads ( $G$ ) are 1.2. (SNiP 2.01.07-85\* ‘Loads and effects’, SNiP (1985b)). For SNiP calculations Equation 5 needs to be completely re-written as Equation 38

$$5. \quad Q_d = \beta_1 G + \beta_2 V = \left[ \frac{\pi D \alpha \int_0^L (c_u/\beta_3) dz}{\beta_5} + \frac{A_b N_c (c_u/\beta_4)}{\beta_6} \right] / \beta_7$$

$$38. \quad N = \beta_1 G + \beta_2 V = \left[ \frac{\gamma_{cR} R A}{(1.0/\gamma_c)} + \frac{u \gamma_{cf} \sum f_i h_i}{(1.0/\gamma_c)} \right] / \gamma_k$$

$$39. \quad N = 1.2G + 1.2V = \left[ \frac{\gamma_{cR} R A}{(1.0/1.0)} + \frac{u \gamma_{cf} \sum f_i h_i}{(1.0/1.0)} \right] / 1.4$$

### 13.1 Design calculation

For a 15 m long pile (12 m into the clay) and 0.45 m diameter and taking an  $I_L = 0.2$  (limit of SNiP) the skin friction calculation is summarised as Table 10.

Using Table 7 from SNiP 2.02.03-85 (Figure 14, in the Appendix of the current paper) and a representative  $I_L$  of 0.1 at the pile toe depth of 12 m below top of bearing stratum (SNiP is not clear if depth is below ground level or top of bearing stratum) the base resistance is  $R = 1400$  kPa.

$$40. \quad N = 1.2G + 1.2V = \left[ \frac{1.0 \times 1400 \times 0.45^2 (\pi/4)}{(1.0/1.0)} + \frac{\pi \times 0.45 \times 456.7}{(1.0/1.0)} \right] / 1.4$$

Taking  $V = 0.25G$

$$41. \quad N = 1.2G + 1.2(0.25G) = (222.7 + 645.6)/1.4 = 1.5G = (222.7 + 645.6)/1.4$$

$G = 413.5$  kN  
 $V = 103.3$  kN  
 $Q_{work} = 516.8$  kN

The equivalent FOS =  $(868.3/516.8) = 1.68$ . The calculated shaft

Material	Layer	Depth to mid-point: m	$h_i$ : m	$l_L$	$\gamma_{cf}$	$f_i$ : kPa	$\gamma_{cf} \times f_i \times h_i$ : kPa m
Made ground	1	0.5	1	0.0	0.0	0.0	0.0
Made ground	2	1.5	1	0.0	0.0	0.0	0.0
Made ground	3	2.5	1	0.0	0.0	0.0	0.0
Stiff clay	4	3.5	1	0.2	0.6	49.5	29.7
Stiff clay	5	4.5	1	0.2	0.6	53.1	31.9
Stiff clay	6	5.5	1	0.2	0.6	56.2	33.7
Stiff clay	7	6.5	1	0.2	0.6	58.9	35.3
Stiff clay	8	7.5	1	0.2	0.6	61.3	36.8
Stiff clay	9	8.5	1	0.2	0.6	63.5	38.1
Stiff clay	10	9.5	1	0.2	0.6	65.5	39.3
Stiff clay	11	10.5	1	0.2	0.6	67.4	40.4
Stiff clay	12	11.5	1	0.2	0.6	69.1	41.5
Stiff clay	13	12.5	1	0.2	0.6	70.7	42.4
Stiff clay	14	13.5	1	0.2	0.6	72.3	43.4
Stiff clay	15	14.5	1	0.2	0.6	73.7	44.2
					$\Sigma$		456.7

Table 10. Example of SNIp 2.02.03-85 calculation

and base resistances from the ‘ $\alpha c_u$ ’ method (832.6 kN and 225.2 kN) gives a combined resistance of 1057.8 kN, which is not too dissimilar to the 868.3 kN from the SNIp calculation. The correlations implicit in SNIp seem to give capacities very similar to UK practice.

#### 14. Summary of results

Table 11 summarises calculations for the 0.45 m diameter, 15 m long pile analysed throughout the paper, using the seven design methods. Figure 10 shows the calculated combined unfactored allowable loads for a 0.45 m diameter pile ( $Q_{work}$ ) for the various design approaches with respect to pile lengths from 10 to 20 m.

Figure 11 shows the same for a 0.9 m diameter pile. Figure 12 shows the global factor of safety for the 0.9 m pile. Most codes have a consistent factor of safety; the UK value drops slightly as the pile lengthens, as the base resistance is less significant and it is the base that has the higher partial factor. The DA2 approach (with the reduction for a bored pile) has an increasing factor of safety as the pile lengthens, as only the skin friction is reduced. In all other cases a single FOS value is used over the range of pile lengths studied. Coincidentally, the DA3 calculations and the AS2159 (Australian) calculations basically give the same results in terms of pile length and overall FOS. Therefore the lines on Figures 10–12 are virtually indistinguishable.

Code	G: kN	V: kN	$Q_{work}$ : kN	Equivalent FOS	$\beta$ factors used by the code for this design (non-unity)
Conventional design	305.4	76.4	381.8	3.0	$\beta_7$
AS2159-2009	349.8	87.5	437.3	2.42	$\beta_1, \beta_2$ and $\beta_7$
EC7-UK DA1-2	341.2	85.3	426.5	2.48	$\beta_2, \beta_5, \beta_6$ and $\beta_7$
EC7-Ireland DA2	318.6	79.6	398.2	2.66	$\beta_1, \beta_2, \beta_5, \beta_6$ and $\beta_7$
EC7-The Netherlands DA3	348.2	87.1	435.3	2.43	$\beta_3, \beta_4, \beta_5$ and $\beta_6$
AASHTO (USA)	248.0	62.0	310.0	3.85 <sup>a</sup>	$\beta_1, \beta_2, \beta_5, \beta_6$ and $\beta_7$
SNIp (Russia)	413.5	103.3	516.8	1.68	$\beta_1, \beta_2$ and $\beta_7$

<sup>a</sup> 3.41 if compare capacity with shaft and base resistances calculated using 25th percentile through the undrained shear strength data. (see Figure 8)

Table 11. Summary calculations, 0.45 m diameter; 15 m long pile



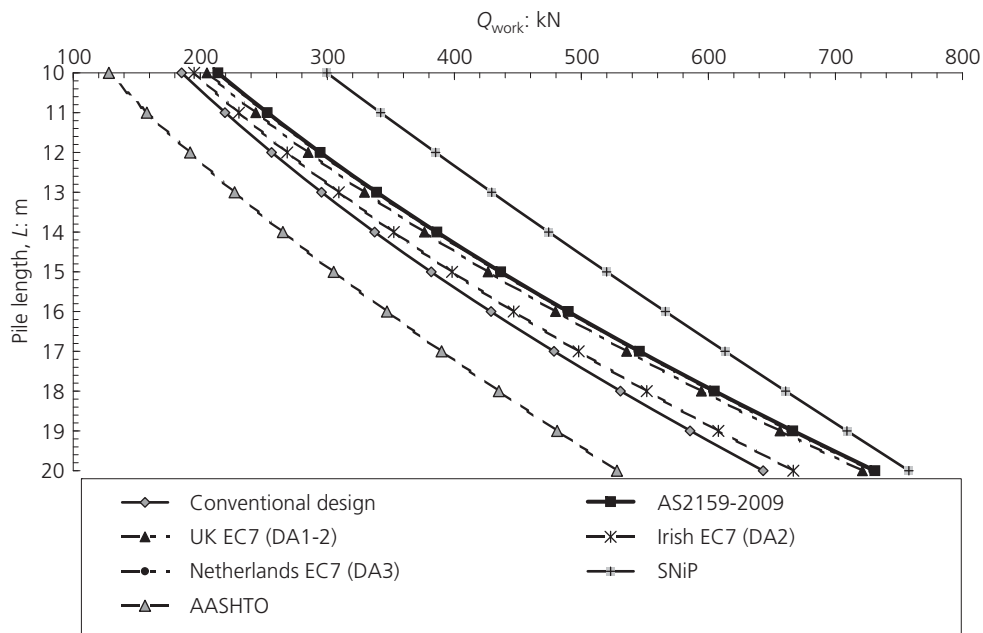


Figure 10. Unfactored working load plotted against pile length (0.45 m diameter pile)

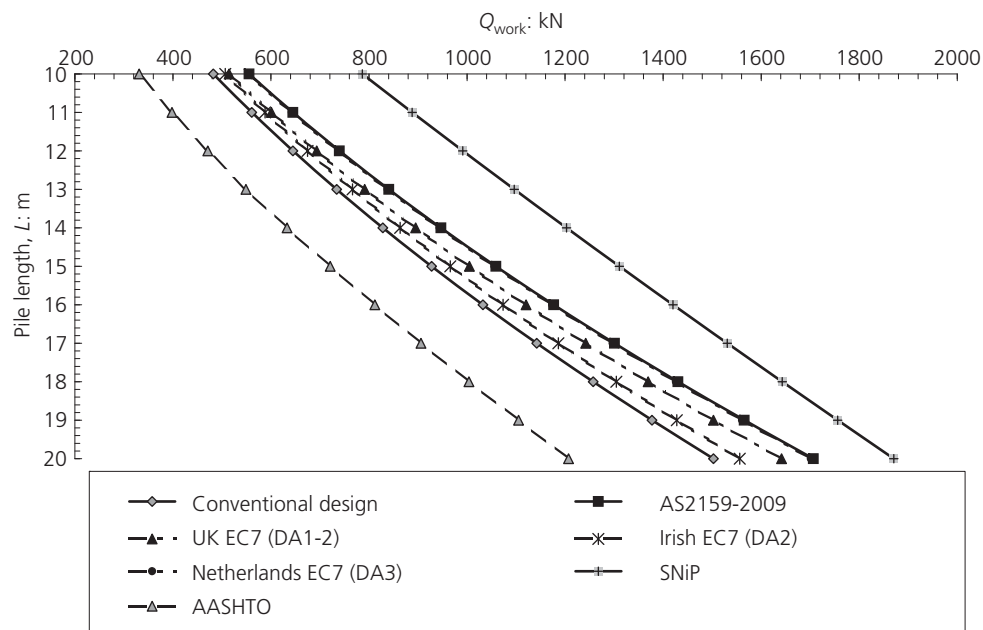


Figure 11. Unfactored working load plotted against pile length (0.9 m diameter pile)

## 15. Discussion and conclusion

The following observations are made based on the study described in the current paper.

(a) The UK (DA1), Netherlands (DA3) and AS2159 calculations give closely similar results (for this design example, using the

$\alpha$ -method of calculation) with a global FOS of just under 2.5. The Irish DA2 approach gives a slightly higher FOS value. The difference occurs when AASHTO and SNiP are considered. AASHTO is a very conservative code as the factors and the loading and resistance are very high. AASHTO would be even more conservative if the design line

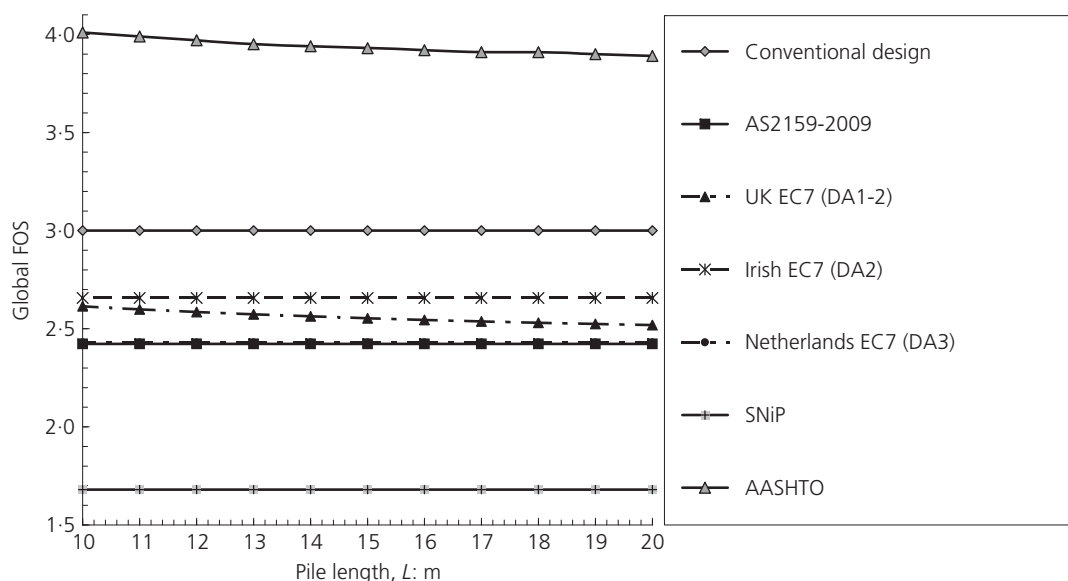


Figure 12. Global FOS plotted against pile length (0.9 m diameter pile)

(Equation 8a) was used instead of Equation 8c. The fact that AASHTO is mainly a bridge code could be why the variable loading factor of 1.75 is very high and why designs are very conservative. The SNiP calculations are significantly less conservative.

- (b) Most codes have the flexibility of applying different factors to the shaft and base resistance. The base is generally factored higher as more uncertainty exists in the determination of what the pile is founded in and how much the base is disturbed by construction. The Australian and Russian codes use a single reduction factor applied to the combined resistances.
- (c) The Australian code is unique in that the engineer has input into the factor of safety chosen by means of a simple risk analysis approach. This recognises that site conditions dictate the amount of uncertainty in the design to a certain extent and that a 'one-size-fits-all' approach can constrain engineering judgement which is crucial for good design. This could also be seen in the Eurocode context as embodied in the use of a 'cautious estimate', which is perhaps a more abstract concept that achieves a similar result.
- (d) Direct comparison of the allowable working pile resistance,  $Q_{work}$ , for each code is obscured by the fact that different estimates of shear strength were used, especially in AASHTO and SNiP where triaxial data only and liquidity index correlations respectively are used to derive a  $c_u$  profile and  $f_i$  profile respectively.
- (e) There is little guidance in any of the design codes on how to construct a design line for the shear strength profile. Some codes specify (or imply) the use of average soil parameters while Eurocode 7 (design by calculation) requires the use of a 'characteristic design line' which is a 'cautious estimate'. Code drafters could adopt a statistical approach (e.g. mean or 5th percentile); however, it is considered that this ignores the

causes of ground variability. The use of a 'cautious estimate' or similar concept does allow the engineer a degree of flexibility in this respect. If the engineer accepts each data point as equally valid then a design line could be derived statistically. It does seem curious that partial factors can be assigned without knowledge of how conservatively engineers treat their soil data. If average soil values are to be used in design then higher partial factors are needed than if 5th percentile values are used. This is investigated further in the companion paper (Vardanega *et al.*, 2012).

- (f) A complication when comparing different codes of practice is that permanent and variable loads are factored differently from code to code. For a fair comparison of codes, the factors on loads (actions) and resistances need to be brought together. The key to success is that there is a clear understanding between structural and geotechnical engineers as to who applies the partial factors on actions.
- (g) For bored piles in London Clay ' $\alpha$ ' of 0.5 is generally recommended. The AASHTO approach and the SNiP approach use similar values of shaft resistance. AASHTO has ' $\alpha$ ' of 0.55 dropping gradually as estimates of  $c_u$  increase. In other words this code penalises a high  $c_u$  value.
- (h) In SNiP the factor  $\gamma_{cf}$  can be interpreted as similar to the ' $\alpha$ ' concept as it reduces the shear strength of the clay around a bored pile and relates to the method of installation. The use of liquidity index ( $I_L$ ) is not without basis as relationships between  $I_L$  and  $c_u$  have been discussed (e.g. Muir Wood, 1983). Possibly, the use of  $I_L$  in SNiP 'works' because it indirectly measures values of  $c_u$ , which relate to shaft friction.
- (i) An interesting feature of the AASHTO approach is that the SPT is not favoured for design; triaxial data are favoured. This is despite SPT data sometimes displaying less scatter than triaxial data (see Figures 4 and 5 and LDSA (2000)).

(j) The major reason SNiP appears unconservative is that the partial factor on resistance (1.4) and the partial factor on actions (1.2) are both relatively low. It is not known if the estimates of skin friction are conservative or not as the source of the data in SNiP Tables 2 and 7 (Figures 11 and 12 in this paper) is unclear. A comparison with  $\alpha c_u$  values derived suggests that they are high at shallow depth and low at greater depth. Overall for the 12 m pile, there is little difference between the SNiP representative resistance and that derived from the ‘ $\alpha$ ’ method. It would be interesting to know performance statistics for piled foundation systems constructed under the SNiP framework.

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### Appendix – SNiP design charts

For the shaft resistance of piles in clay, cubic equations of the form in Equation 42 were fitted to the data tables from SNiP. The regression coefficients are shown in Table 12 and the plotted functions in Figure 13.

$$42. \quad f_i = a(I_L)^3 + b(I_L)^2 + c(I_L) + z$$

For the base resistance of piles in clay, linear equations of the form shown below were fitted to the data tables from SNiP. The regression coefficients are shown in Table 13 and the plotted functions in Figure 14.

$$43. \quad R = A(I_L) + K$$

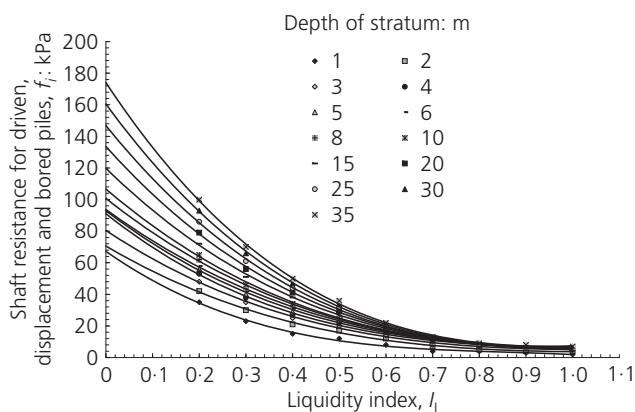


Figure 13. Graphical representation of Table 2 in SNiP 2.02.03-85

Stratum depth, d: m	d = 1	d = 2	d = 3	d = 4	d = 5	d = 6	d = 8	d = 10	d = 15	d = 20	d = 25	d = 30	d = 35
a	-90.1	-43.8	-48.0	-76.6	-58.9	-41.2	-47.1	-52.2	-61.4	-74.1	-79.1	-89.2	-107.7
b	230.7	151.3	168.5	225.8	198.2	168.2	186.7	203.7	236.0	275.6	303.5	341.1	388.4
c	-206.0	-174.5	-195.9	-235.1	-226.4	-214.7	-234.4	-252.2	-288.5	-329.2	-365.0	-405.1	-447.5
z	67.2	70.6	80.4	91.1	93.0	93.7	100.7	106.6	119.8	133.6	146.8	160.5	174.0

Table 12. Fitted coefficients (Table 2, SNiP)

Design resistance, $R$ : kPa below the pile tip	$d = 3$	$d = 5$	$d = 7$	$d = 10$	$d = 12$	$d = 15$	$d = 18$	$d = 20$	$d = 30$	$d = 40$
A	-1054	-1107	-1196	-1250	-1446	-1679	-1911	-2107	-3300	-5000
K	845	875	1116	1325	1541	1811	2088	2304	3300	4500

Table 13. Fitted coefficients (Table 7, SNiP)

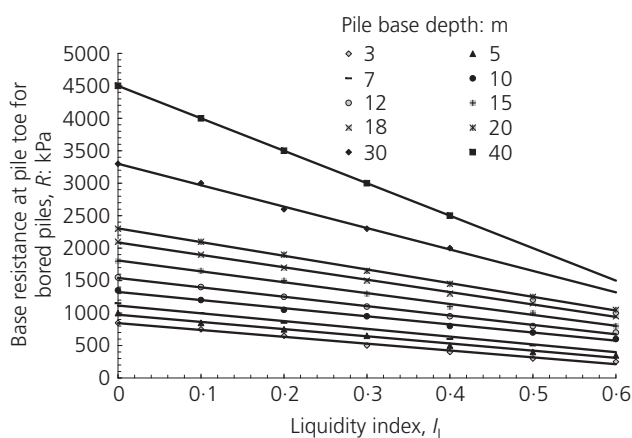


Figure 14. Graphical representation of Table 7 in SNiP 2.2.03-85

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