BS 8004:2015

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Code of practice for foundations

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Contents

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 106, an inside back cover and a back cover.

Foreword

Publishing information

This British Standard is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 30 June 2015. It was prepared by Technical Committee B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

Together with BS EN 1997-1:2004+A1:2013, this British Standard supersedes BS 8004:1986, which is withdrawn.

Relationship with other publications

BS 8004 gives non-contradictory, complementary information for use with BS EN 1997 and its National Annexes.

Information about this document

This is a full revision of the standard, which introduces the following principal changes:

- the revised text is fully compatible with the current version of Eurocode 7 (BS EN 1997);
- guidance is given on designing foundations according to limit state principles using partial factors;
- guidance is given on the selection of design parameters for soils;
- guidance is given on the calculation of ultimate bearing resistance of shallow foundations;
- guidance is given on the design of pile foundations by calculation and by testing;
- the revised text reflects advances in foundation technology over the past 30 years.

Use of this document

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Any user claiming compliance with this British Standard is expected to be able to justify any course of action that deviates from its recommendations.

Presentational conventions

The provisions of this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

The word "should" is used to express recommendations of this standard. The word "may" is used in the text to express permissibility, e.g. as an alternative to the primary recommendation of the clause. The word "can" is used to express possibility, e.g. a consequence of an action or an event.

Notes and commentaries are provided throughout the text of this standard. Notes give references and additional information that are important but do not form part of the recommendations. Commentaries give background information.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

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1 Scope

This British Standard gives recommendations for the design and construction of foundations for the normal range of buildings and engineering structures. It provides non-contradictory, complementary information for use in conjunction with BS EN 1997 and its UK National Annex.

Clause **4** gives general recommendations for the design and construction of all types of foundations; Clause **5** and Clause **6** give specific recommendations for the design and construction of spread foundations and pile foundations (respectively).

Annex A gives specific recommendations for the design and construction of helical steel piles.

Annex B gives specific recommendations for the design and construction of underpinning.

Annex C gives information about specific geological formations encountered in the UK.

Annex D gives information about the UK Government's policy regarding archaeological finds.

NOTE 1 This standard does not cover the design and construction of earthworks, for which see BS 6031.

NOTE 2 This standard does not cover the design and construction of earth retaining structures, for which see BS 8002.

NOTE 3 This standard does not cover the design and construction of maritime works, for which see BS 6349.

NOTE 4 For non-industrial structures of not more than four storeys, see BS 8103-1.

NOTE 5 This standard does not cover the design and construction of foundations for reciprocating machinery.

NOTE 6 This standard does not cover the design and construction of offshore foundations, for which see BS EN ISO 19901-4.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 65, *Specification for vitrified clay pipes, fittings and ducts, also flexible mechanical joints for use solely with surface water pipes and fittings*

BS 437, *Specification for cast iron drain pipes, fittings and their joints for socketed and socketless systems*

BS 1852-1, *Plastics piping systems for non-pressure underground drainage and sewerage – Polypropylene (PP) – Part 1: Specifications for pipes, fittings and the system*

BS 4449, *Steel for the reinforcement of concrete – Weldable reinforcing steel – Bar, coil and decoiled product – Specification*

BS 4660, *Thermoplastics ancillary fittings of nominal sizes 110 and 160 for below ground gravity drainage and sewerage*

BS 4729, *Clay and calcium silicate bricks of special shapes and sizes – Recommendations*

BS 4962, *Specification for plastics pipes and fittings for use as subsoil field drains*

BS 5480, *Specification for glass reinforced plastics (GRP) pipes, joints and fittings for use for water supply or sewerage*

BS 5481, *Specification for unplasticized PVC pipe and fittings for gravity sewers*

BS 5837, *Trees in relation to design, demolition and construction – Recommendations*

BS 5911 (all parts), *Concrete pipes and ancillary concrete products*

BS 5930, *Code of practice for site investigation*

BS 5975, *Code of practice for temporary works procedures and the permissible stress design of falsework*

BS 6031, *Code of practice for earthworks*

BS 6349 (all parts), *Maritime works*

BS 8006-1:2010, *Code of practice for strengthened/reinforced soils and other fills*

BS 8002, *Code of practice for earth retaining structures*

BS 8081, *Code of practice for ground anchors*

BS 8215, *Code of practice for design and installation of damp-proof courses in masonry construction*

BS 8417, *Preservation of wood – Code of practice*

BS 8500-1:2015, *Concrete – Complementary British Standard to BS EN 206-1 – Part 1: Method of specifying and guidance for the specifier*

BS 8500-2, *Concrete – Complementary British Standard to BS EN 206-1 – Part 2: Specification for constituent materials and concrete* [1)]

BS 10175, *Code of practice on investigation of potentially contaminated sites*

BS EN 206:2013, *Concrete – Specification, performance, production and conformity*

BS EN 295, *Vitrified clay pipe systems for drains and sewers*

BS EN 335:2013, *Durability of wood and wood-based products – Use classes: definitions, application to solid wood and wood-based products*

BS EN 350-2, *Durability of wood and wood-based products – Natural durability of solid wood – Part 2: Guide to the natural durability of and treatability of selected wood species of importance in Europe*

BS EN 351-1, *Durability of wood and wood-based products – Preservative-treated solid wood – Part 1: Classification of preservative penetration and retention*

BS EN 460, *Durability of wood and wood-based products – Natural durability of solid wood – Guide to the durability requirements for wood to be used in hazard classes*

BS EN 598, *Ductile iron pipes, fittings, accessories and their joints for sewerage applications – Requirements and test methods*

BS EN 771-1, *Specification for masonry units – Part 1: Clay masonry units*

BS EN 771-2, *Specification for masonry units – Part 2: Calcium silicate masonry units*

BS EN 771-3, *Specification for masonry units – Part 3: Aggregate concrete masonry units*

[1)] Informative reference is made to BS 8500-2:2015.

BS EN 771-4, *Specification for masonry units – Part 4: Autoclaved aerated concrete masonry units*

BS EN 771-5, *Specification for masonry units – Part 5: Manufactured stone masonry units*

BS EN 771-6, *Specification for masonry units – Part 6: Natural stone masonry units*

BS EN 1090 (all parts), *Execution of steel structures and aluminium structures*

BS EN 1401-1, *Plastic piping systems for non-pressure underground drainage and sewerage – Unplasticized poly(vinyl chloride) (PVC-U) – Part 1: Specifications for pipes, fittings and the system*

BS EN 1536, *Execution of special geotechnical works – Bored piles* [2)]

BS EN 1852-1, *Plastics piping systems for non-pressure underground drainage and sewerage – Polypropylene (PP) – Part 1: Specifications for pipes, fittings and the system*

BS EN 1916, *Concrete pipes and fittings, unreinforced, steel fibre and reinforced*

BS EN 1990:2002+A1:2005, *Eurocode: Basis of structural design*

BS EN 1991 (all parts), *Eurocode 1: Actions on structures* [3)]

BS EN 1992 (all parts), *Eurocode 2: Design of concrete structures* [4)]

BS EN 1993 (all parts), *Eurocode 3: Design of steel structures* [5)]

BS EN 1995 (all parts), *Eurocode 5: Design of timber structures* [6)]

BS EN 1996-1-1, *Eurocode 6: Design of masonry structures – Part 1-1: General rules for reinforced and unreinforced masonry structures*

BS EN 1996-2, *Eurocode 6: Design of masonry structures – Part 2: Design considerations, selection of materials and execution of masonry*

BS EN 1997-1:2004+A1:2013, *Eurocode 7: Geotechnical design – Part 1: General rules*

BS EN 1997-2:2007, *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing (incorporating corrigendum 2010)*

BS EN 10025, *Hot rolled products of structural steels*

BS EN 10080, *Steel for the reinforcement of concrete – Weldable reinforcing steel – General*

BS EN 10210, *Hot finished structural hollow sections of non-alloy and fine grain steels*

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2) Informative reference is made to BS EN 1536:2010+A1:2015. At the time of publication, amendment A1:2015 is in preparation.

3) Specific references are made to the following part: BS EN 1991-2:2003, *Eurocode 1: Actions on structures – Part 2: Traffic loads on bridges*.

4) Specific references are made to the following part: BS EN 1992-1-1:2004+A1:2014, *Eurocode 2: Design of concrete structures – Part 1-1: General rules and rules for buildings.*

5) Specific references are made to the following parts:
- BS EN 1993-1-1:2005, *Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*;
- BS EN 1993-5, *Eurocode 3: Design of steel structures – Part 5: Piling*.

6) Specific references are made to the following part: BS EN 1995-1-1:2004, *Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings.*

BS EN 10219, *Cold formed welded structural hollow sections of non-alloy and fine grain steels*

BS EN 10248, *Hot rolled sheet piling of non alloy steels*

BS EN 10249, *Cold formed sheet piling of non alloy steels*

BS EN 12666-1, *Plastics piping systems for non-pressure underground drainage and sewerage – Polyethylene (PE) – Part 1: Specifications for pipes, fittings and the system*

BS EN 12699, *Execution of special geotechnical work – Displacement piles* [7)]

BS EN 12715, *Execution of special geotechnical works – Grouting*

BS EN 12716, *Execution of special geotechnical works – Jet grouting*

BS EN 12794, *Precast concrete products – Foundation piles*

BS EN 13670, *Execution of concrete structures*

BS EN 14199:2015, *Execution of special geotechnical works – Micropiles*

BS EN 14227 (all parts), *Hydraulically bound mixtures – Specifications*

BS EN 14679:2005, *Execution of special geotechnical works – Deep mixing*

BS EN 14731:2005, *Execution of special geotechnical works – Ground treatment by deep vibration*

BS EN 15237:2007, *Execution of special geotechnical works – Vertical drainage*

BS EN 16228, *Drilling and foundation equipment – Safety*

BS EN ISO 13793, *Thermal performance of buildings – Thermal design of foundations to avoid frost heave*

BS EN ISO 14688-1, *Geotechnical investigation and testing – Identification and classification of soil – Part 1: Identification and description*

BS EN ISO 14688-2, *Geotechnical investigation and testing – Identification and classification of soil – Part 2: Principles for a classification* [8)]

BS EN ISO 14689-1, *Geotechnical investigation and testing – Identification and classification of rock – Part 1: Identification and classification*

prEN ISO 22476-13, *Geotechnical investigation and testing – Field testing – Part 13: Plate loading test* [9)]

prEN ISO 22477 (all parts), *Geotechnical investigation and testing – Testing of geotechnical structures* [9)]

BS ISO 5667-11, BS 6068-6.11: *Water quality – Sampling – Part 11: Guidance on sampling of groundwaters*

NA to BS EN 1991-2, *UK National Annex to Eurocode 1 – Actions on structures – Part 2: Traffic loads on bridges*

NA to BS EN 1992-1-1, *UK National Annex to Eurocode 2 - Design of concrete structures – Part 1-1: General rules and rules for buildings* [10)]

NA to BS EN 1993-1-1, *UK National Annex to Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings*

NA to BS EN 1993-5, *UK National Annex to Eurocode 3: Design of steel structures – Part 5: Piling*

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7) Informative reference is made to BS EN 12699:2001.
8) Informative reference is made to BS EN ISO 14688-2:2004+A1:2003.
9) In preparation.
10) Informative reference is made to NA to BS EN 1992-1-1:2004.

NA+A1:2014 to BS EN 1997-1:2004+A1:2013, *UK National Annex to Eurocode 7 – Geotechnical design – Part 1: General rules*

PAS 8811, *Code of practice for temporary works – Client procedures* [11)]

PAS 8812, *Guide to the application of European Standards in temporary works design* [11)]

PD 6694-1, *Recommendations for the design of structures subject to traffic loading to BS EN 1997-1:2004*

PD 6697, *Recommendations for the design of masonry structures to BS EN 1996-1-1 and BS EN 1996-2*

ASTM D7949-14, *Standard test methods for thermal integrity profiling of concrete deep foundations* [12)]

Other publications

[N1] BURLAND, J.B., BROMS, B.B., and de MELLO, V.F.B. *Behaviour of foundations and structures*, State-of-the-Art Report, Session 2, Proc. 9th International Conference on Soil Mechanics and Foundation Engineering, 1977, Tokyo, Vol. 2, pp 495–546.

[N2] LORD, J.A., CLAYTON, C.R.I., and MORTIMORE, R.N. *Engineering in chalk (CIRIA Report C574)*. London: CIRIA, 2002. ISBN 0-86017-574-X.

[N3] INSTITUTION OF CIVIL ENGINEERS. *ICE Specification for piling and embedded retaining walls* (2nd edition, 2007), London: Thomas Telford Publishing, ISBN 978-0-7277-3358-0.

3 Terms and definitions

For the purposes of this British Standard, the terms and definitions given in BS EN 1990, BS EN 1997, and the following apply.

NOTE Symbols are defined locally to the equations where they are used.

3.1 low-rise building/housing

building of not more than three storeys above ground intended for domestic occupation and of traditional masonry construction

NOTE For example, detached, semi-detached, and terraced housing or flat. Also includes certain single-storey, non-residental buildings.

3.2 micropiles

drilled piles which have a diameter smaller than 300 mm

[SOURCE: BS EN 14199:2015, **3.1**]

3.3 undrained shear strength of fine soils (c_u)

[SOURCE: BS EN ISO 14688-2:2004+A1:2013, Table 5]

3.3.1 extremely low

$c_u < 10$ kPa

3.3.2 very low

$c_u = 10–20$ kPa

3.3.3 low

$c_u = 20–40$ kPa

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11) In preparation. It is anticipated that PAS 8811 and 8812 will be published in 2015.
12) Available from ASTM International, www.astm.org (last viewed 25/6/15).

3.3.4 medium
c_u = 40–75 kPa

3.3.5 high
c_u = 75–150 kPa

3.3.6 very high
c_u = 150-300 kPa

3.3.7 extremely high
c_u > 300 kPa

3.4 working platform
temporary structure that provides a foundation for construction plant

4 General rules

4.1 Choice and design of foundation

4.1.1 General

4.1.1.1 The design of foundations should conform to BS EN 1997-1 and this clause (**4**).

NOTE 1 Guidance on geotechnical design, construction, and verification can be found in the ICE manual of geotechnical engineering (2012), Volume II *[1].*

NOTE 2 Guidance on the selection of a suitable foundation type can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 9 *[2] and* Volume II, Chapter 52 *[1].*

4.1.1.2 The design of maritime works should conform to BS 6349.

NOTE Guidance on geotechnical engineering principles, problematic soils, and site investigation can be found in the ICE manual of geotechnical engineering (2012), Volume I *[2].*

4.1.2 Spread foundations

4.1.2.1 The design of spread foundations should conform to **4.1.1.1** and Clause **5**.

NOTE Spread foundations are generally suitable in, but not limited to, the following design situations:

- *where an adequate bearing stratum occurs at shallow depth (typically less than 2 m deep);*
- *on dense (or denser) coarse soils above the water table;*
- *on medium strength (or stronger) fine soils.*

4.1.2.2 Spread foundations should not be placed on non-engineered fill unless such use can be justified on the basis of a thorough ground investigation and detailed design. Spread foundations should generally not be placed on engineered fill unless that fill is designed for the use of spread foundations.

4.1.3 Pile foundations

4.1.3.1 The design of pile foundations should conform to **4.1.1.1** and Clause **6**.

NOTE Pile foundations are generally suitable in, but not limited to, the following design situations:

- *on compressible strata overlying bedrock;*

- *on compressible strata overlying coarse soils of adequate density, there being no other beds of greater compressibility below the coarse soils;*
- *where the strata consist of fine soils of great thickness capable of supporting the piles by friction;*
- *where there is a need to minimize movements of a structure owing to swelling or shrinkage of surface soils; and*
- *where the addition of a foundation load would cause instability of the existing ground (e.g. at the crest of an existing slope).*

4.1.3.2 Deep foundations, such as piles, should be considered when no adequate bearing stratum exists at shallow depths to permit economic construction of spread foundations.

4.1.3.3 The design of helical steel pile foundations should conform to Annex A.

4.1.4 Underpinning

The design of underpinning should conform to **4.1.1.1** and Annex B.

NOTE 1 Underpinning is typically used for, but not limited to, the following design situations:

- *to rectify distress that a structure has already suffered;*
- *to extend foundations downwards to enable building and civil engineering works to be carried out; and*
- *to provide adequate resistance to increased loading on an existing structure.*

NOTE 2 Guidance on the use of underpinning can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 83 *[1].*

4.1.5 Piled rafts

*COMMENTARY ON **4.1.5***

There are two different types of piled raft in common practice: raft-enhanced pile groups and pile-enhanced rafts.

4.1.5.1 The design of piled rafts should conform to **4.1.1.1** and Clause **5** and Clause **6**.

NOTE 1 Piled rafts are typically used for, but not limited to, the following design situations:

- *to minimize total, and particularly differential, foundation settlement;*
- *to reduce bending moments and shear forces in unpiled rafts; and*
- *to reduce the thickness of a raft foundation.*

NOTE 2 Guidance on the use of piled rafts can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 56 *[1].*

4.1.5.2 The design of raft-enhanced pile groups should ensure that stresses within both the piles and the raft do not exceed the elastic range of behaviour.

4.1.5.3 The design of pile-enhanced rafts should ensure that all of the piles mobilise their ultimate resistance before the raft reaches its ultimate resistance.

4.1.6 Piled embankments

4.1.6.1 The design of piles used in piled embankments should conform to **4.1.1.1** and Clause **6**.

NOTE Piled embankments are typically used for, but not limited to, the following design situations:

- *to support infrastructure over low strength ground (e.g. alluvium);*
- *to support infrastructure over compressible ground (e.g. loose fill);*
- *to support infrastructure over collapsing ground (e.g. loess and karst geology); and*
- *to support infrastructure over unstable areas (e.g. land prone to mining subsidence).*

4.1.6.2 The design of embankments should conform to BS 6031.

4.1.6.3 The design of basal reinforcement to piled embankments should conform to BS 8006-1:2010, **8.3**.

4.2 Basis of geotechnical design

NOTE Attention is drawn to The Construction (Design and Management) Regulations 2015 *[3], with regards to health and safety requirements for construction works, in particular:*

- *Regulation 9, duties of designers;*
- *Regulation 11, duties of the principal designer in relation to pre-construction phase; and*
- *Regulation 22, excavations.*

4.2.1 Design requirements

4.2.1.1 Geotechnical data and investigation

4.2.1.1.1 Geotechnical data should be obtained in accordance with BS EN 1997-1:2004+A1:2013, Clause **3**.

4.2.1.1.2 Particular attention should be given to the requirements of BS EN 1997-1:2004+A1:2013, **3.1**.

4.2.1.1.3 Geotechnical investigations should conform to BS EN 1997-1:2004+A1:2013, **3.2**.

4.2.1.1.4 Ground investigations should conform to BS EN 1997-2 and BS 5930. Information about the site from the desk study should be used to supplement that obtained in any new ground investigation.

4.2.1.1.5 Sampling and testing of contaminated ground and groundwater should conform to BS 10175 and BS ISO 5667-11.

4.2.1.1.6 The spacing and depth of site investigations for foundations should conform to BS EN 1997-2:2007, **B.3**, and this clause.

4.2.1.1.7 The number of investigation points should be sufficient to establish ground conditions, and any variability in those conditions, in the vicinity of the foundation.

4.2.1.1.8 For low-rise buildings, the number of investigation points undertaken (n) should satisfy the following, unless the ground conditions can be shown to be sufficiently uniform to justify a greater spacing:

$$n \geq \begin{cases} 3 & \text{for } A \leq 300\ \text{m}^2 \\ A/100\ \text{m}^2 & \text{for } A > 300\ \text{m}^2 \end{cases} \tag{1}$$

where:

A is the plan area of each building's footprint.

4.2.1.1.9 Where a development involves construction of several buildings, equation (1) may be applied to the aggregate plan area of the buildings' footprints, provided the ground conditions can be shown to be sufficiently uniform.

4.2.1.1.10 For high-rise buildings and industrial structures, the number of investigation points undertaken should conform to BS EN 1997-2:2007, **B.3**.

4.2.1.1.11 For low-rise buildings, the depth of investigation below the planned base of the foundation (z_a) should satisfy the following minimum requirements:

$$z_a \geq \begin{cases} 2b_F \\ 3\text{ m} \end{cases} \tag{2}$$

where:

b_F is the smaller side length of the wall's foundation (on plan).

4.2.1.1.12 For high-rise buildings and industrial structures, the depth of investigation should conform to the minimum requirements of BS EN 1997-2:2007, **B.3**, namely:

$$z_a \geq \begin{cases} 3b_F \\ 6\text{ m} \end{cases} \tag{3}$$

4.2.1.1.13 Where the base of the foundation is located on or near bedrock, the depth of investigation may be reduced.

4.2.1.1.14 The presence of trees and large shrubs should be noted during the course of the ground investigation, so that decisions can be taken concerning their retention or subsequent removal.

NOTE Guidance on site investigation can be found in the ICE manual of geotechnical engineering (2012), Volume I, Section 4: Site investigation *[2] and in the* NHBC Design Guide NF21, Efficient design of piled foundations for low rise housing (2010), Section 4: Site investigation [4].

4.2.2 Design situations

4.2.2.1 Design situations should be specified in accordance with BS EN 1997-1:2004+A1:2013, **2.2**.

4.2.2.2 Design situations for foundations should include:

- the conditions given in BS EN 1997-1:2004+A1:2013, **6.3**;
- collapse settlement;
- building within the zone of influence of buried highwalls (at the edge of opencast mining);
- building over landfill;
- the influence of global ground movements on foundation performance; and
- all other conditions specific to the site that can reasonably be forseen.

4.2.2.3 To conform to BS EN 1990, exceptional conditions involving local failure (such as a burst water pipe that continues to leak after the burst) should be classified as accidental design situations.

4.2.2.4 Design water levels should be compatible with the drainage provisions.

NOTE Information about exclusion zones around buried highwalls can be found in BRE Report 424 *[5].*

4.2.3 Design considerations

*COMMENTARY ON **4.2.3***

Design and construction considerations for foundations are given throughout BS EN 1997-1. The design considerations given in this clause are more specific examples of the issues that can affect the performance of a foundation.

4.2.3.1 General

The design of foundations should consider:

- the design considerations given in BS EN 1997-1:2004+A1:2013, **6.4**;
- the possibility of weak layers below the proposed foundation level leading to greater settlement of the foundation than would occur in their absence;
- the possibility of shear failure of the ground supporting the foundation owing to the presence of cuttings, excavations, or sloping ground;
- the possibility of inclined or jointed strata in which the bedding or joint planes dip towards an excavation or natural depression causing stability problems due to failure along bedding planes or joints;
- the possibility of settlement of adjacent structures caused during or after construction of the foundations;
- the effect on the groundwater regime during and after construction;
- the execution (i.e. construction or installation) of the foundation; and
- global ground movements (e.g. due to bulk excavation or filling, water table changes, etc.) that can cause foundation deformations, additional to those induced by superstructure loads, and/or additional forces in the foundation or superstructure.

NOTE Global ground movements can be horizontal or vertical depending on the design situation.

4.2.3.2 Drainage

*COMMENTARY ON **4.2.3.2***

Provision of suitable drainage is vital to ensure the acceptable performance of a foundation. Groundwater control is separate from the control of surface water, such as rainfall run-off.

4.2.3.2.1 Drainage should be provided to prevent:

- surface water from entering and eroding the face of any excavations;
- build-up of water pressures during construction in case they have harmful effects upon the foundations; and
- instability of slopes.

4.2.3.2.2 Surface water drains should be constructed using one of the types of pipe listed in **4.3.10**.

4.2.3.2.3 Drainage systems should be designed for ease of maintenance and renewal during the design working life of the structure.

4.2.3.2.4 Drainage systems should be designed with positive outfalls to prevent ponding and include provisions of suitable discharge outfalls or soakaways.

4.2.3.2.5 Where the safety and serviceability of the works depend on the successful performance of the drainage system, the consequences of failure should be considered and one of the following conditions (or a combination of them) should be applied:

- a maintenance programme specified;
- a drainage system specified that will perform adequately without maintenance; or
- a secondary ("backup") system specified – for example, a pipe or channel that encloses the primary system – that will prevent any potential leakage from entering the ground beneath or next to the structure.

NOTE 1 Guidance on surface water control can be found in Groundwater lowering in construction *[6],* Construction dewatering and groundwater control *[7], and* Groundwater control: design and practice *[8].*

NOTE 2 Guidance on groundwater control can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 80 *[1].*

4.2.3.3 Cyclic loading

The design of foundations subject to cyclic loading should consider the following:

- degradation of ground strength (leading to ultimate limit states being exceeded at loads below those expected from verifications based on static strength);
- degradation of ground stiffness, leading to an accumulation of permanent foundation displacement ("ratcheting" effects); and
- amplification of loads or movements owing to resonance.

NOTE Guidance on foundations subjected to cyclic and dynamic loads can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 60 *[1].*

4.2.3.4 Trees

*COMMENTARY ON **4.2.3.4***

Foundations built adjacent to existing trees might suffer deleterious effects from the penetration of root-systems. These effects include increased loading on the structure and penetration of roots into joints or drainage systems.

4.2.3.4.1 The principles and procedures to be applied to achieve a harmonious and sustainable relationship between trees (including shrubs and hedges) and structures should conform to BS 5837.

4.2.3.4.2 Foundations should be designed to accommodate any volumetric changes in clay soils that might be caused by the presence of nearby trees.

NOTE Guidance on building near trees can be found in NHBC Standards, Chapter 4.2 *[9].*

4.2.3.5 Environmental considerations

The design of foundations should consider:

- their effect on sensitive species;
- generation and control of noise and dust during construction;
- generation, reuse and disposal of waste materials;
- minimizing the amount of material to be disposed of;
- contaminants entering watercourses (for example, as a result of excavation);
- the use of ground improvement; and
- the carbon footprint of the construction and the use of the structure.

4.2.3.6 Ground improvement

*COMMENTARY ON **4.2.3.6***

Ground improvement is the modification of ground properties obtained by binding or densifying the ground or creating inclusions in the ground.

Ground improvement involves the enhancement of ground properties, principally by strengthening of stiffening processes and compaction or densification mechanisms, to achieve a specific geotechnical performance (see ICE manual of geotechnical engineering, Volume I *[2]).*

Ground improvement includes any method by which natural or disturbed ground has its geotechnical performance altered.

Ground improvement covers many methods of ground treatment, including: modification, chemical alteration, reinforcement with steel or geosynthetics, strengthening by drainage, densification by vibration or consolidation, void filling, settlement or compensation grouting (see Contributions to Géotechnique 1948–2008 *[10]).*

Ground improvement techniques can be used to alter ground strength, stiffness, or permeability. Common applications include methods to enhance bearing resistance, control post-construction settlements, increase or decrease ground permeability, improve ground stability, generate controlled ground displacement, and mitigate the risk of liquefaction.

4.2.3.6.1 The design of ground improvement by deep mixing should conform to BS EN 14679:2005, Clause **7**.

4.2.3.6.2 The design of ground improvement by deep vibration should conform to BS EN 14731:2005, Clause **7**.

4.2.3.6.3 The design of ground improvement by vertical drainage should conform to BS EN 15237:2007, Clause **7**.

NOTE 1 Comprehensive guidance on the design and execution of the most common forms of ground improvement can be found in Ground Improvement *[11].*

NOTE 2 Information about the role of ground improvement can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 25 *[2].*

NOTE 3 Guidance on vibrocompaction and vibro stone columns, vibro concrete columns, and dynamic compaction can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapters 59 *and* 84 *[1].*

NOTE 4 Guidance on a range of ground improvement techniques can be found in CIRIA Reports C572 *[12] and* C573 *[13].*

NOTE 5 Guidance on grouting for ground engineering (including permeation, rock grouting, hydrofracture, ground compaction, jet grouting, and compensation grouting) can be found in CIRIA Report C514 *[14].*

NOTE 6 Guidance on soil mixing can be found in the BRE Design Guide to soft soil stabilisation *[15].*

NOTE 7 Information about vibro stone columns can be found in BRE Report BR 391 *[16].*

NOTE 8 Information about the design of vibrocompaction/vibroflotation and vertical band drains can be found in the Institution of Civil Engineers' Specification for ground treatment *[17].*

NOTE 9 Guidance on the design, construction and control of rigid inclusion ground improvements can be found in the recommendations of the ASIRI National Project *[18].*

4.3 Materials

NOTE For guidance on specific formations (e.g. London Clay, Lambeth group, glacial soils and tills, problematic soils, chalk and Mercia mudstone group), see Annex C.

4.3.1 Soils

4.3.1.1 General

*COMMENTARY ON **4.3.1.1***

The design of foundations usually involves effective stress analysis, although, in some circumstances, total stress analysis might be appropriate or necessary for the design of foundations in fine soils. Soil properties are determined as part of the site investigation process but might be supplemented by data from back analysis of comparable foundations in similar ground conditions.

4.3.1.1.1 The identification and description of soil should conform to BS EN ISO 14688-1.

4.3.1.1.2 The classification of soil should conform to BS EN ISO 14688-2.

4.3.1.1.3 Soil properties should be determined in accordance with BS EN 1997-2 and BS 5930.

4.3.1.1.4 Characteristic soil parameters should be selected in accordance with BS EN 1997-1, based on the results of field and laboratory tests, complemented by well-established experience.

NOTE 1 Guidance on soil description can be found in Soil and rock description in engineering practice [19].

NOTE 2 Information about the behaviour of soils as particulate materials can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 14 *[2].*

NOTE 3 Information about the strength and deformation behaviour of soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 17 *[2].*

4.3.1.2 Very coarse soils (cobbles and boulders)

*COMMENTARY ON **4.3.1.2***

Very coarse soils contain a majority (by weight) of particles >63 mm in size. Cobbles are between 63 mm and 200 mm in size; boulders are greater than 200 mm.

Very coarse soils should be identified and classified in accordance with BS EN ISO 14688 and BS 5930.

4.3.1.3 Coarse soils (sands and gravels)

*COMMENTARY ON **4.3.1.3***

Coarse soils contain a majority (by weight) of particles ≤63 mm in size and do not stick together when wet. Sand particles are between 0.063 mm and 2 mm in size; gravels are between 2 mm and 63 mm. Coarse soils cannot be remoulded.

4.3.1.3.1 When establishing the values of parameters for coarse soils, the following should be considered:

- the items listed in BS EN 1997-1:2004+A1:2013, **2.4.3(5)**;
- the weakening of collapsible soils above the groundwater table, owing to percolation or a rise in groundwater levels;
- disturbance of dense deposits owing to unsuitable construction methods; and

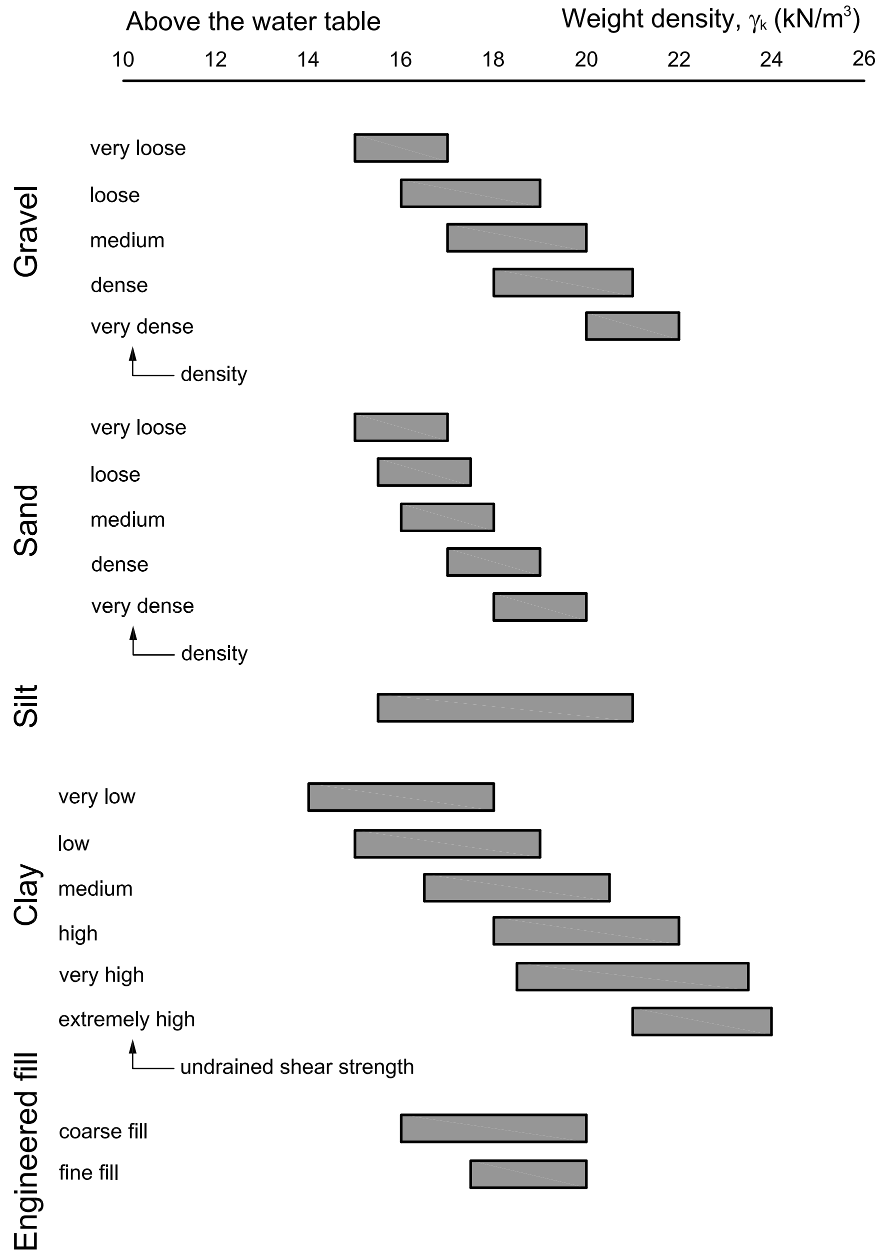
- the presence of weaker material.

4.3.1.3.2 For coarse soils above the groundwater table, the suggested values for characteristic weight density given in Figure 1 may be used in the absence of reliable test results.

4.3.1.3.3 For coarse soils below the groundwater table, the suggested values for characteristic weight density given in Figure 2 may be used in the absence of reliable test results.

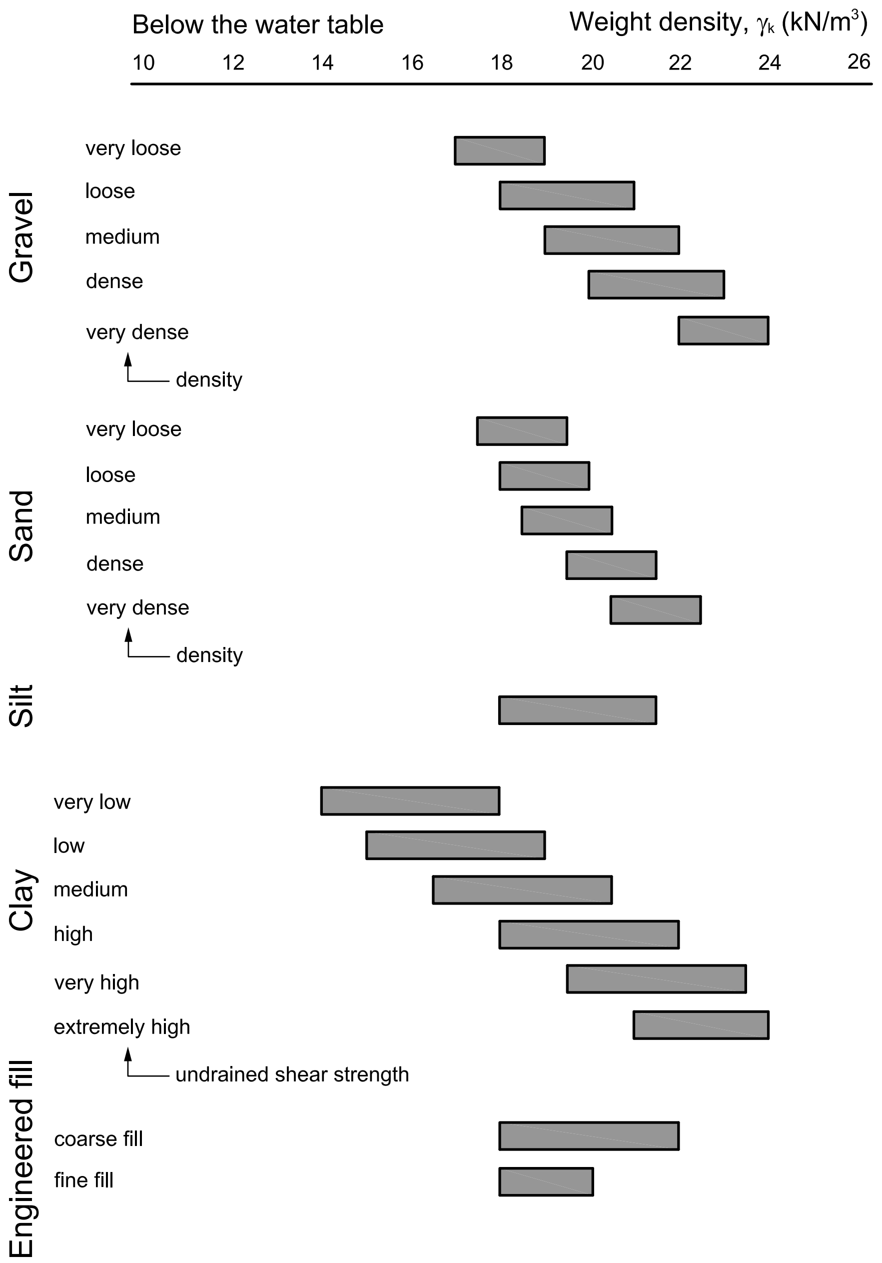
4.3.1.3.4 A superior value of weight density should be selected when a high value is unfavourable; an inferior value should be selected when a low value is unfavourable.

Figure 1 **Suggested values for characteristic weight density of soils above the groundwater table**



Source: Data from Bond [20].

Figure 2 **Suggested values for characteristic weight density of soils below the groundwater table**



Source: Data from Bond [20].

4.3.1.3.5 For siliceous sands and gravels, the characteristic constant volume (also known as critical state) effective angle of shearing resistance ($\varphi'_{cv,k}$) may, in the absence of reliable test results, be estimated from:

$$\varphi'_{cv,k} = 30° + \varphi'_{ang} + \varphi'_{PSD} \tag{4}$$

where:

φ'_{ang} is contribution to $\varphi'_{cv,k}$ from the angularity of the particles; and

φ'_{PSD} is contribution to $\varphi'_{cv,k}$ from the soil's particle size distribution.

Values of φ'_{ang} and φ'_{PSD} are given in Table 1.

4.3.1.3.6 For siliceous sands and gravels with a fines content less than 15%, the characteristic peak effective angle of shearing resistance ($\varphi'_{pk,k}$) may be estimated from:

$$\varphi'_{pk,k} = \varphi'_{cv,k} + \varphi'_{dil} \quad (5)$$

where:

φ'_{dil} is contribution to $\varphi'_{pk,k}$ from soil dilatancy.

Values of φ'_{dil} are given in Table 1.

4.3.1.3.7 The value of φ'_{dil} may alternatively be estimated from:

$$\varphi'_{dil} = nI_R = n\left[I_D \times \ln\left(\sigma_c / \sigma'_f\right) - 1\right] \quad (6)$$

where:

n is 3 for triaxial strain or 5 for plane strain;

I_R is the soil's relative dilatancy index;

I_D is the soil's density index (defined in BS EN ISO 14688-2);

σ_C is the aggregate crushing stress; and

σ'_f is mean effective stress in the soil at peak strength.

NOTE 1 Bolton [21] defines the relative dilatancy index as $I_R = I_D(Q - l_n\ p') - 1$, where $Q = l_n\ \sigma_c$ and $p' = \sigma'_f$.

NOTE 2 Many geotechnical problems can be simplified into a two-dimensional form where the foundation or structure is significantly long in one direction in comparison with other dimensions. Hence, a large number of stability problems involving embankments and cuttings, retaining walls and strip footings are commonly analysed by assuming a plane strain condition in which no deformation occurs in the direction of the long dimension of the foundation or structure.

4.3.1.3.8 The value of σ_C may be taken as 20 MPa for quartz sands, but can be substantially larger for quartz silts, and substantially smaller for carbonate sands (see *The strength and dilatancy of sands* [21]).

4.3.1.3.9 If the fines content of the coarse soil exceeds 25%, then φ'_{dil} should be assumed to be zero, unless testing demonstrates otherwise. For coarse soils with fines content between 15% and 25%, φ'_{dil} may be determined by linear interpolation.

4.3.1.3.10 If shearing is matrix (i.e. fines) controlled, then φ'_{ang} should be assumed to be zero, unless testing demonstrates otherwise. For coarse soils with fines content between 15% and 25%, φ'_{ang} may be determined by linear interpolation.

4.3.1.3.11 The characteristic angle of shearing resistance (φ'_k) for coarse soils with fines content exceeding 25% should be determined as for fine soils (see **4.3.1.4**).

Table 1 **Values of φ'_{ang}, φ'_{PSD} and φ'_{dil} to obtain values of $\varphi'_{pk,k}$ and $\varphi'_{cv,k}$ for siliceous sands and gravels with fines content not exceeding 15%**

Soil property	Determined from	Classification	Parameter D)
Angularity of particles A)	Visual description of soil	Rounded to well-rounded	$\varphi'_{ang} = 0°$
		Sub-angular to sub-rounded	$\varphi'_{ang} = 2°$
		Very angular to angular	$\varphi'_{ang} = 4°$
Uniformity coefficient, C_U B)	Soil grading	$C_U < 2$ (evenly graded)	$\varphi'_{PSD} = 0°$
		$2 \le C_U < 6$ (evenly graded)	$\varphi'_{PSD} = 2°$
		$C_U \ge 6$ (medium to multi graded)	$\varphi'_{PSD} = 4°$
		High C_U (gap graded), with C_U of fines < 2 E)	$\varphi'_{PSD} = 0°$
		High C_U (gap graded), with $2 \le C_U$ of fines < 6 E)	$\varphi'_{PSD} = 2°$
Density index, I_D C)	Standard penetration test blow count, corrected for energy rating and overburden pressure $(N_1)_{60}$	$I_D = 0\%$	$\varphi'_{dil} = 0°$
		$I_D = 25\%$	$\varphi'_{dil} = 0°$
		$I_D = 50\%$	$\varphi'_{dil} = 3°$
		$I_D = 75\%$	$\varphi'_{dil} = 6°$
		$I_D = 100\%$	$\varphi'_{dil} = 9°$

A) Terms for defining particle shape can be found in BS EN ISO 14688-1.
B) The uniformity coefficient C_U is defined in BS EN ISO 14688-2.
C) The density index I_D is defined in BS EN ISO 14688-2. Density terms may be estimated from the results of field tests (e.g. Standard Penetration Test, Cone Penetration Test) using correlations given in BS EN 1997-2.
D) Values of φ'_{dil} are appropriate for siliceous sands and gravels reaching failure at a mean effective stress up to 400 kPa. For non-siliceous sands, see *The strength and dilatancy of sands* [21].
E) "Fines" refers to that fraction of the soil whose particle size is less than 0.063 mm.

4.3.1.4 Fine soils (silts and clays)

*COMMENTARY ON **4.3.1.4***

Fine soils contain a majority (by weight) of particles ≤63 mm in size and stick together when wet. Silt particles are between 0.002 mm and 0.063 mm in size; clay particles are smaller than 0.002 mm. Fine soils can be remoulded.

Clay soils with plasticity indices greater than about 20% might exhibit considerably lower angles of shearing resistance than observed at the critical state, if their particles become fully aligned with one another. This phenomenon is termed "sliding shear" to distinguish it from "rolling shear" observed in other soils (including coarse soils and fine soils with plasticity indices less than 20%). The angle of shearing resistance exhibited during sliding shear is called the "residual angle of shearing resistance".

4.3.1.4.1 When establishing the values of parameters for fine soils, the following should be considered, as a minimum:

- the items listed in BS EN 1997-1:2004+A1:2013, **2.4.3**(5);
- pre-existing slip surfaces;
- dessication; and
- any changes in stress state either induced by construction or resulting from the final design condition.

4.3.1.4.2 For fine soils above the groundwater table, the suggested values for characteristic weight density given in Figure 1 may be used in the absence of reliable test results**.**

4.3.1.4.3 For fine soils below the groundwater table, the suggested values for characteristic weight density given in Figure 2 may be used in the absence of reliable test results.

4.3.1.4.4 A superior value of weight density should be selected when a high value is unfavourable; an inferior value should be selected when a low value is unfavourable.

4.3.1.4.5 In the absence of reliable test data, the characteristic undrained shear strength of a fine soil ($c_{u,k}$) may be estimated from:

$$\frac{c_{u,k}}{p'_v} = k_1 R_O^{k_2} = k_1 \left(\frac{p'_{v,max}}{p'_v} \right)^{k_2} \quad (7)$$

where:

p'_v is the effective overburden pressure;

$p'_{v,max}$ is the maximum effective overburden pressure that the soil has previously been subjected to;

R_O is the soil's overconsolidation ratio; and

k_1 and k_2 are constants.

NOTE The ratio c_u/p'_v normally varies with depth (i.e. it is not a constant).

4.3.1.4.6 In the absence of reliable test data, the values of k_1 and k_2 in equation (7) may be taken as 0.23 ±0.04 and 0.8 respectively, following *New developments in field and laboratory testing of soils* [22].

4.3.1.4.7 When determining the characteristic undrained strength of high strength fine soils, due allowance should be made for:

- the detrimental effect of any sand or silt partings containing free groundwater;
- the influence of sampling;
- the influence of the method of testing; and
- likely softening on excavation.

4.3.1.4.8 For fine soils, the characteristic constant volume (also known as critical state) effective angle of shearing resistance ($\varphi'_{cv,k}$) may, in the absence of reliable test results, be estimated from:

$$\varphi'_{cv,k} = (42° - 12.5 \log_{10} I_P) \quad \text{for} \quad 5\% \le I_P \le 100\% \quad (8)$$

where:

I_P is the soil's plasticity index (entered as a %).

NOTE 1 Equation (8) is based on an expression proposed by Santamarina and Díaz-Rodriguez [23], which fits data presented by Terzaghi, Peck, and Mesri [24].

NOTE 2 Values of $\varphi'_{cv,k}$ based on this expression are given in Table 2.

Table 2 **Values of $\varphi'_{cv,k}$ for fine soils from plasticity index**

Plasticity index, I_P	Characteristic constant volume angle of shearing resistance, $\varphi'_{cv,k}$
%	Degrees (°)
15	27
30	24
50	21
80	18

NOTE Values of φ'_{cv} in excess of 40° have been observed for clays that classify as highly plastic but show signs of bioturbation or the presence of microfossils.

4.3.1.4.9 The characteristic constant volume effective cohesion ($c'_{cv,k}$) should be taken as zero.

4.3.1.4.10 The peak effective angle of shearing resistance (φ'_{pk}) may be related to the constant volume effective angle of shearing resistance (φ'_{cv}) by:

$$\varphi'_{pk} = \varphi'_{cv} + \varphi'_{dil} \tag{9}$$

where:

φ'_{cv} is the soil's constant-volume angle of shearing resistance; and

φ'_{dil} is the contribution to φ'_{pk} from soil dilatancy.

NOTE 1 The value of φ'_{dil} for fine soils is not the same as that for coarse soils. For fine soils, it is typically in the range of 0°–4°. No specific guidance is given in this standard for values of φ'_{dil} for fine soils.

NOTE 2 Values of φ'_{dil} are known to increase with a fine soil's overconsolidation ratio and are greater than or equal to zero.

4.3.1.4.11 When a clay soil is able to undergo "sliding shear", normally only where pre-existing slip surfaces exist in the ground, then the operational angle of shearing resistance is the clay's residual value (φ'_{res}):

$$\varphi'_{res} \le \varphi'_{cv} \le \varphi'_{pk} \tag{10}$$

where:

φ'_{cv} is the soil's constant-volume angle of shearing resistance; and

φ'_{pk} is the soil's peak angle of shearing resistance.

NOTE Guidance on the undrained strength and the residual shear strength of clay soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 17 *[2].*

4.3.1.5 Mixed soils

*COMMENTARY ON **4.3.1.5***

Some deposits (especially glacial deposits) can comprise a mixture of fine and coarse soils, e.g. sandy clays and clayey sands. The behaviour of mixed soils is typically intermediate between that of coarse and fine soils.

For mixed soils with clay fraction less than 50% (plasticity index less than 30% or liquid limit less than 60%), $\varphi'_{cv,k}$ may, in the absence of reliable test data, be estimated from *Drained shear strength parameters for analysis of landslides* [25].

4.3.1.6 Soil stiffness

*COMMENTARY ON **4.3.1.6***

Young's modulus of elasticity of an isotropic soil (E) is related to its shear modulus (G) by:

$$E = 2G(1 + v) \quad (11)$$

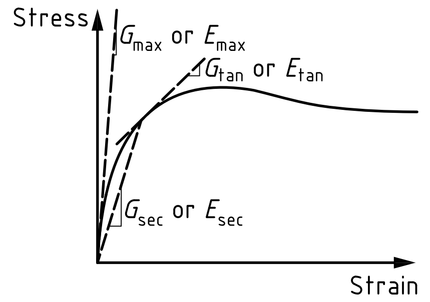
where:

v *is the soil's Poisson's ratio.*

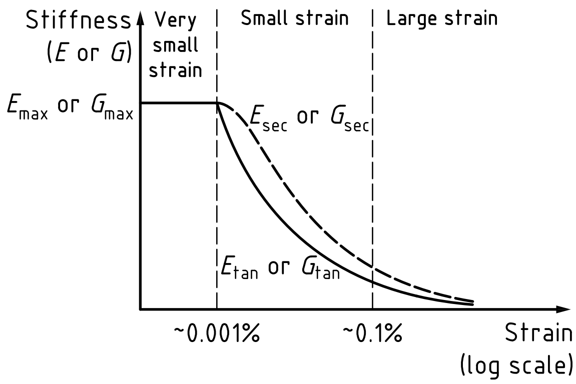
Stiffness parameters for a soil depend on the level of strain (axial ε or engineering shear strain γ) applied to the soil, as illustrated in Figure 3:

- *at very small strains (ε or $\gamma \leq \sim 0.001\%$), the soil's moduli of elasticity (E and G) reach their maximum values (E_{max} and G_{max}), also known as "very-small-strain" values; E_{max} and G_{max} are normally measured using dynamic methods, such as laboratory tests using bender-elements or field tests using seismic methods;*
- *at small strains ($\sim 0.001\% \leq \varepsilon$ or $\gamma \leq \sim 0.1\%$), E and G decrease rapidly with increasing strain, as shown in Figure 3; their values are measured using advanced methods, such as laboratory tests with local gauges;*
- *tangent values of small-strain stiffness (E_{tan} and G_{tan}) are commonly used in numerical methods of design;*
- *at large strains (ε or $\gamma > \sim 0.1\%$), E and G decrease less rapidly with increasing strain and are commonly taken to be constant in value; those values are measured using conventional laboratory testing;*
- *secant values of large-strain stiffness (E_{sec} and G_{sec}) are commonly used in routine methods of design;*
- *soil stiffness parameters are not normally isotropic. Simple isotropic models are to be used with caution, especially if their adequacy has not been demonstrated by back analysis. Depending on the soil's deposition history, different values of stiffness might be appropriate for vertical strains (E_v) compared to horizontal strains (E_h) and for shearing in different directions (G_{hh}, G_{hv}).*

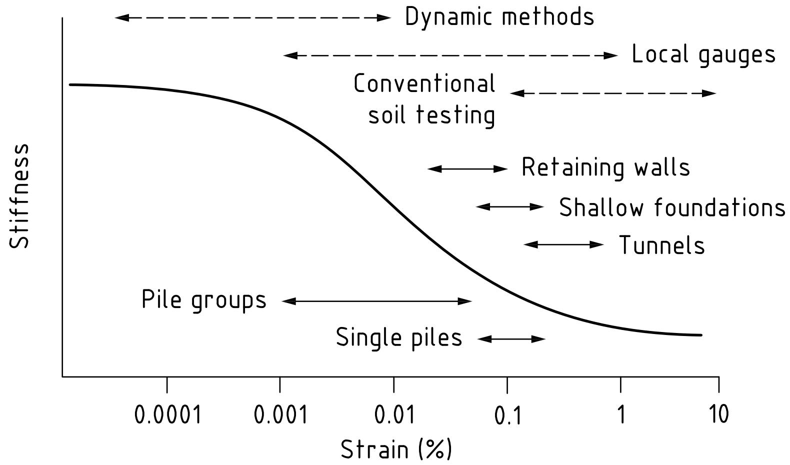
Figure 3 **Stiffness parameters for non-linear soil**



a) Definition of tangent and secant moduli



b) Definition of very small, small and large strain stiffnesses



c) Typical strain ranges for common geotechnical constructions (solid arrows) and laboratory tests (dashed arrows)

NOTE Based on Non-linear soil stiffness in routine design *[26] and the* ICE manual of geotechnical engineering (2012), Volume II, Chapter 52 *[1].*

4.3.1.6.1 The difference between the direction of loading and the direction of measurement of soil stiffness should be taken into account in the assessment of soil stiffness.

4.3.1.6.2 In the absence of reliable test results, the secant shear modulus of a soil, G_{sec}, may be estimated from (see also *Stiffness of sands through a laboratory database* [27]):

$$\frac{G_{\mathrm{sec}}}{G_{\mathrm{max}}} = \left[1 + \left(\frac{\gamma - \gamma_{\mathrm{e}}}{\gamma_{\mathrm{ref}}}\right)^{m}\right]^{-1} \leq 1 \tag{12}$$

where:

G_{max}	is the soil's very-small-strain shear modulus;
γ	is the engineering shear strain in the soil;
γ_e	is the elastic threshold strain beyond which shear modulus falls below its maximum value;
γ_{ref}	is a reference value of engineering shear strain (at which $G_{sec}/G_{max} = 0.5$); and
m	is a coefficient that depends on soil type.

4.3.1.6.3 In the absence of reliable test results, the values of the parameters in equation (12) may be taken from Table 3.

Table 3 **Values of parameters for use with equation (12)**

Soil type	**Parameter**			**Reference**
	γ_{ref} %	m	γ_e %	
Sand	0.02–0.1 (0.044[A)])	0.88	0.02% + 0.012 γ_{ref}	Oztoprak and Bolton [27]
Clays and silts	0.0022 I_P [B)]	0.736 ±0.122 [C)]	0 (assumed)	Vardanega and Bolton [28]

A) Mean value.
B) I_P is the soil's plasticity index.
C) ± value indicates standard error.

4.3.1.6.4 In the absence of reliable test results, the very-small-strain shear modulus of a soil, G_{max}, may be estimated from (see *Non-linear soil stillness in routine design* [26] and *Stiffness at small strain: research and practice* [29]):

$$\frac{G_{max}}{p_{\mathrm{ref}}} = \frac{k_1}{(1+e)^{k_2}}\left(\frac{p'}{p_{\mathrm{ref}}}\right)^{k_3} \tag{13}$$

where:

e	is the soil's voids ratio;
p'	is the mean effective stress in the soil;
p_{ref}	is 100 kPa; and

k_1, k_2, and k_3 are coefficients that depend on soil type.

4.3.1.6.5 In the absence of reliable test results, the values of the parameters in equation (13) may be taken from Table 4.

Table 4 **Values of parameters for use with equation (13)**

Soil type	Parameter				Reference
	k_1	k_2	k_3	p_{ref}	
Fine soil	2 100 A)	0	0.6–0.8 A)	1 kPa	Viggianni and Atkinson [30]
Sand	370–5 760 B)	3	0.49–0.86 C)	100 kPa	Oztoprak and Bolton [27]
Clays and silts	20 000 ±5 000	2.4	0.5	1 kPa	Vardanega and Bolton [28]

A) Depends on the soil's plasticity index, I_P.
B) Decreasing with strain.
C) Increasing with strain.

4.3.2 Rocks and rock masses

*COMMENTARY ON **4.3.2***

The engineering properties of rock relevant in design are controlled by the extent and orientation of the bedding planes and joints within the rock mass, together with the water pressures on the discontinuity planes. The site investigation needs to establish the strength and orientation of the discontinuity planes.

Weak rocks, particularly weakly cemented sandstones, fissured shales and chalk, are often difficult materials to sample and test.

4.3.2.1 The identification and classification of rock should conform to BS EN ISO 14689-1.

4.3.2.2 Rock properties should be determined in accordance with BS EN 1997-2 and BS 5930, as well as BS EN 1997-1:2004+A1:2013, **3.3.2**, **3.3.8** and **3.3.9**.

4.3.2.3 Characteristic rock parameters for intact rock should be selected in accordance with BS EN 1997-1, based on the results of field and laboratory tests, complemented by well-established experience.

4.3.2.4 Design parameters for the rock mass should take into account the properties of the intact rock and any discontinuities. See BS EN 1997-1:2004+A1:2013, **3.3.8**.

4.3.2.5 The following non-destructive tests may be used to determine rock mineralogy and composition in order to predict the rock's performance during excavation (in particular, if it will slurry or smear rather than be broken up into smaller pieces):

- microscope;
- X-ray computerized tomography; and
- spectroscopy.

NOTE 1 Guidance on rock description in engineering practice can be found in Soil and rock description in engineering practice *[19].*

NOTE 2 Information about the behaviour of rocks can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 18 *[2].*

NOTE 3 Information about mudrocks, clay, and pyrite (and their issues) can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 36 *[2].*

4.3.3 Fill

*COMMENTARY ON **4.3.3***

The term "fill" refers to artificially deposited material, for example, in an excavation or as ground made by human activity. These materials are also termed "artificial ground".

"Non-engineered" fill is material that is dumped with little control and in deep lifts. It is often poorly compacted, and thus in a loose state, and has varying geotechnical properties, both horizontally and vertically. Non-engineered fill is commonly referred to as "made ground".

"Engineered" fill is material that is placed with some degree of control to ensure that its geotechnical properties conform to a predetermined specification. Engineered fill is commonly referred to as just "fill".

On geological maps, "made ground" refers to material placed above and "fill" to material placed below original ground level. On recent British Geological Survey (BGS) maps, these terms have been altered to "made up ground" and "infilled ground".

4.3.3.1 All materials intended to be placed behind foundations should be properly investigated and classified.

4.3.3.2 Non-engineered fill, such as industrial, chemical, and domestic wastes, should not be placed beneath foundations.

4.3.3.3 Engineered fill comprising selected coarse granular soils, such as well-graded small rockfills, gravels, and sands, may be placed beneath foundations.

4.3.3.4 Engineered fill should be appropriate to the intended application. The fill should be classified and an earthworks specification provided detailing acceptability criteria, compliance testing, and compaction requirements. The general specification of earthworks fill should conform to BS 6031.

NOTE 1 Guidance on the description of made ground can be found in Soil and rock description in engineering practice, Chapter 14 *[19].*

NOTE 2 Information about non-engineered fills can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 34 *[2].*

NOTE 3 Information about fill formation and deposits (covering opencast mining backfill, colliery spoil, pulverized fly ash, industrial and chemical wastes, urban fill, domestic refuse, infilled docks, pits, and quarries, and hydraulic fill) can be found in BRE Report 424 (2nd edition), Chapter 2 *[31].*

4.3.4 Earthworks

The properties of earthworks should be determined in accordance with BS 6031.

4.3.5 Groundwater

4.3.5.1 Groundwater pressures should be determined by considering hydrological, hydrogeological, and environmental information.

4.3.5.2 To conform to BS EN 1997-1:2004+A1:2013, **2.4.6.1**(6)P, design values of groundwater pressure at the serviceability limit state should be the most unfavourable values that could occur during normal circumstances.

4.3.5.3 To conform to BS EN 1997-1:2004+A1:2013, **2.4.6.1**(6)P, design values of groundwater pressure at the ultimate limit state should be the most unfavourable values that could occur during the design lifetime of the structure.

NOTE The relationship between characteristic and design water pressures can be determined by applying a geometrical margin.

4.3.5.4 If suitable statistical data is available, then design values of groundwater pressure at the serviceability limit state should be chosen with a return period at least equal to the duration of the design situation; and design values at the ultimate limit state should be chosen such that there is a 1% probability that they are exceeded during the design situation.

4.3.5.5 Where groundwater pressures are not hydrostatic, the design should take into account:

- the worst credible combination of heterogeneity and anisotropy of permeability;
- effects of layering, fissuring and other heterogeneity;
- any geometrical features that could cause pressures to concentrate (such as in corners of excavations).

4.3.5.6 Long-term changes in groundwater that are likely to occur during the design working life of the structure (including those due to climate change and rising groundwater) should be taken into account.

4.3.5.7 Increased groundwater pressures owing to burst pipes and other failures of engineered systems should be classified as accidental actions if the event that causes the increase in groundwater pressure is unlikely to occur during the design working life of the structure.

4.3.5.8 The design of a foundation should be based on the most adverse water pressure conditions that can be anticipated.

NOTE Guidance on the selection of water tables and seepage forces can be found in CIRIA Report C580 *[32].*

4.3.5.9 Design water pressures should take into account the effect of tides on water levels in the ground.

NOTE Guidance on tides and water level variations can be found in BS 6349-1-3.

4.3.5.10 If the equilibrium level of the water table is well defined and measures are taken to prevent it changing during heavy rain or flood, the design water pressures can be calculated from the position of the equilibrium water table, making due allowance for possible seasonal variations. Otherwise, the most adverse water pressure conditions that can be anticipated should be used in design.

4.3.5.11 Equilibrium water levels in fine soils should be determined from piezometric readings taken over an adequate length of time.

4.3.5.12 Allowance should be made in undrained (i.e. total stress) analyses for water pressures due to the temporary filling of cracks in fine soils.

4.3.5.13 Water pressures used in drained (i.e. effective stress) analyses should be determined for the groundwater regime in the vicinity of the structure.

4.3.5.14 Where a difference in water pressures exists on opposite sides of a foundation, allowance should be made for seepage around the wall. Where layers of markedly different permeability exist, the water levels relevant to each permeable stratum should be taken into account.

4.3.5.15 The distribution of pore water pressures may be determined from a flow net, provided it adequately represents the hydraulic and permeability conditions in the vicinity of the structure.

4.3.6 Concrete

4.3.6.1 Concrete incorporated into foundations should conform to BS EN 1992-1-1, BS EN 206, and BS 8500-2.

4.3.6.2 Concrete incorporated into foundations should be specified in accordance with BS EN 206 and BS 8500-1.

4.3.6.3 Steel reinforcement for concrete foundations should conform to BS EN 10080 and BS 4449.

4.3.7 Steel

4.3.7.1 Steel incorporated into foundations should conform to BS EN 1993-1-1, BS EN 1993-5, and BS 8081, as appropriate.

4.3.7.2 The values of steel parameters should be determined in accordance with BS EN 1993-1-1 and BS EN 1993-5, and their UK National Annexes.

4.3.7.3 Hot rolled steel products should conform to BS EN 10025.

4.3.7.4 Hot rolled steel products manufactured to a different standard than BS EN 10025 may be used if it can be demonstrated by appropriate additional testing that the products meet the requirements of BS EN 10025 that are relevant to the foundation.

4.3.7.5 Cold formed hollow steel sections should conform to BS EN 10219.

4.3.7.6 Cold formed hollow steel sections manufactured to a different standard than BS EN 10219 may be used if it can be demonstrated by appropriate additional testing that the sections meet the requirements of BS EN 10219 that are relevant to the foundation.

4.3.8 Timber

4.3.8.1 Timber incorporated into foundations should conform to BS EN 1995-1-1.

4.3.8.2 The values of timber parameters should be determined in accordance with BS EN 1995-1-1.

4.3.9 Masonry

4.3.9.1 Masonry incorporated into foundations should conform to BS EN 1996-1-1.

4.3.9.2 The values of masonry parameters should be determined in accordance with BS EN 1996-1-1.

4.3.9.3 The following masonry units should conform to the relevant part of BS EN 771:

- clay masonry units (Part 1);
- calcium silicate masonry units (Part 2);
- aggregate concrete masonry units (Part 3);
- autoclaved aerated concrete masonry units (Part 4);
- manufactured stone masonry units (Part 5); and
- natural stone masonry units (Part 6).

4.3.9.4 The dimensions of clay and calcium silicate brick of special shapes and sizes should conform to BS 4729.

4.3.9.5 Mortars used in masonry foundations should conform to PD 6697.

4.3.9.6 Damp proof courses used in masonry foundations should conform to BS 8215.

4.3.9.7 Wall ties should conform to PD 6697.

4.3.10 Pipes

Pipes used in drainage systems for foundations should conform to one of the following standards, as appropriate:

- vitrified clay pipes – BS 65 and BS EN 295;
- concrete pipes – BS 5911 and BS EN 1916;
- glass reinforced plastics (GRP) pipes – BS 5480;
- cast iron – BS 437;
- ductile iron – BS EN 598;
- unplasticised polyvinyl-chloride (PVC-U) – BS 4660 or BS 5481 or BS EN 1401-1;
- polypropylene (PP) – BS EN 1852-1;
- polyethylene (PE) – BS EN 12666-1;
- thermoplastics structure wall pipes – BS 4962;
- geotextile wrapped land drains – BS 4962.

4.4 Durability

4.4.1 General

The durability of foundations should conform to BS EN 1990.

4.4.2 Concrete

4.4.2.1 The durability of concrete should conform to BS EN 1992-1-1.

4.4.2.2 Exposure classes for concrete should be determined in accordance with BS EN 206 and BS 8500-1.

4.4.2.3 For the purpose of specifying concrete to be used in foundations, ground conditions should be classified in accordance with BS 8500-1:2015, Table A.2.

NOTE 1 BS 8500-1:2015 and BS 8500-2:2015 are complementary British Standards to BS EN 206:2013.

NOTE 2 Guidance on concrete in aggressive ground can be found in BRE Special Digest 1 *[33].*

4.4.3 Steel

4.4.3.1 The durability of steel should conform to BS EN 1993-1-1.

4.4.3.2 The durability of steel reinforcement in reinforced concrete should conform to BS EN 1992-1-1.

NOTE Guidance on corrosion at bi-metallic contacts and its remediation can be found in PD 6484.

4.4.4 Timber

4.4.4.1 General

NOTE 1 Information on the biological agents that can attack wood can be found in BS EN 335:2013, Annex C.

NOTE 2 Guidance on shipworm (various species of the genera Teredo and Banksia), Martesia, and gribble (various species of the genus Limnoria) can be found in BRE Technical Note 59 *[34].*

4.4.4.1.1 The preservative treatment of timber should conform to BS 8417.

4.4.4.1.2 Components should be machined so that they contain a high proportion of permeable sapwood.

NOTE Wood species can be selected for permeability and sapwood content from the information given in BS EN 350-2.

4.4.4.1.3 The durability of timber should conform to BS EN 1995-1-1, which requires timber and wood-based materials to have either:

- adequate natural durability conforming to BS EN 350-2 for the particular hazard class defined in BS EN 335-1:1992, BS EN 335-2:1992, and BS EN 335-3:1992, or
- be given a preservative treatment selected conforming to BS EN 351-1 and BS EN 460.

NOTE BS EN 335-1:1992, BS EN 335-2:1992, and BS EN 335-3:1992 have been superseded by BS EN 335:2013.

4.4.4.1.4 All machining of timber, including notching, should be undertaken before applying preservative treatment.

4.4.4.2 Service classes

Service classes for timber should be determined in accordance with BS EN 1995-1-1.

4.4.4.3 Use classes

Use classes for timber should be determined in accordance with BS EN 335.

NOTE 1 Use classes relevant to timber in earth retaining structures are summarized in Table 5.

NOTE 2 BS EN 1995-1-1:2004 requires timber structures to be assigned to one of three "service classes". BS EN 335:2013 defines five "use classes" for wood and wood-based products. BS EN 335:2013, Annex A provides a possible mapping of service classes to use classes.

Table 5 **Use classes relevant to timber in foundations**

Use class	Situation	Attack is possible by:
UC 4	Wood is in direct contact with the ground or fresh water	Fungi and wood-destroying fungi Wood-boring insects Termites (in countries where these present a hazard) Bacterial decay
UC 5	Wood is permanently or regularly submerged in salt water (i.e. sea water and brackish water)	Invertebrate marine organisms Wood-destroying fungi Growth of surface moulds and staining fungi Wood-boring insects (above water)

4.4.5 Masonry

The durability of masonry foundations should conform to BS EN 1996-1-1, BS EN 1996-2, and PD 6697.

4.5 Geotechnical analysis

4.5.1 The actions assumed in the geotechnical analysis of foundations should conform to BS EN 1991.

4.5.2 The actions assumed in the geotechnical analysis of foundations subject to traffic loading should additionally conform to the UK National Annex to BS EN 1991-2 and to PD 6694-1.

4.5.3 To conform to BS EN 1990:2002+A1:2005, **6.4.3.3**(4), combinations of actions for accidental design situations should include either the accidental action itself or actions that occur after the accidental event.

4.5.4 The effects of dynamic and cyclic loads on the performance of a foundation should be considered.

4.5.5 For the verification of limit states GEO, EQU, and STR, the value of load classification factor defined in BS EN 1991-2:2003 should be taken as α = 1.1. This factor should be applied to the equivalent vertical loading for earthworks and the earth pressure effects due to rail traffic actions, according to the requirements of BS EN 1991-2:2003, **6.3.2**(3)P and **6.3.6.4**.

4.6 Ultimate limit states

4.6.1 General

The ultimate limit state design of a foundation should conform to BS EN 1997-1:2004+A1:2013, **2.4.7**.

4.6.2 Design values of geotechnical parameters

*COMMENTARY ON **4.6.2***

The UK National Annex to BS EN 1997-1:2004+A1:2013 states that "it might be more appropriate to determine the design value of φ'_{cv} directly, rather than apply the partial factor γ_φ (= 1.25 for limit state GEO, Set M2) to its characteristic value".

4.6.2.1 In accordance with BS EN 1997-1:2004+A1:2013, **2.4.5.2**, the characteristic value of a geotechnical parameter should be selected as a cautious estimate of the value "affecting the occurrence of the limit state". The value of φ'_k may therefore be selected as a peak value, a constant volume value, a residual value, or an intermediate value (as appropriate).

4.6.2.2 The design values of geotechnical parameters should conform to BS EN 1997-1:2004+A1:2013, 2.4.6.2.

4.6.2.3 When the peak angle of shearing resistance is the value that affects the occurrence of the limit state, the design value of shearing resistance (φ'_d) should either be assessed directly or obtained from:

$$\tan\varphi'_d = \frac{\tan\varphi'_{pk,k}}{\gamma_\varphi} \qquad (14)$$

where:

$\varphi'_{pk,k}$ is the characteristic value of the soil's peak angle of shearing resistance; and

γ_φ is the partial factor specified in the UK National Annex to BS EN 1997-1:2004+A1:2013.

4.6.2.4 If it is anticipated that there can be significant post-peak softening of the soil's shearing resistance, together with significant straining of the soil, then the peak angle of shearing resistance should not be selected as the value that affects the occurrence of the limit state.

4.6.2.5 When the constant volume angle of shearing resistance is the value that affects the occurrence of the limit state, the design value of shearing resistance (φ'_{d}) should either be assessed directly or obtained from:

$$\tan\varphi'_{d} = \min\begin{cases} \dfrac{\tan\varphi'_{pkk}}{\gamma_{\varphi}} \\ \dfrac{\tan\varphi'_{cv,k}}{\gamma_{\varphi,cv}} \end{cases} \qquad (15)$$

where:

$\varphi'_{pk,k}$ and γ_{φ}	are as defined for equation (14);
$\varphi'_{cv,k}$	is the characteristic value of the soil's constant volume angle of shearing resistance; and
$\gamma_{\varphi,cv}$	is a partial factor whose value is 1.0.

4.6.2.6 When the residual angle of shearing resistance is the value that affects the occurrence of the limit state, the design value of shearing resistance (φ'_{d}) should either be assessed directly or obtained from:

$$\tan\varphi'_{d} = \min\begin{cases} \dfrac{\tan\varphi'_{pk,k}}{\gamma_{\varphi}} \\ \dfrac{\tan\varphi'_{res,k}}{\gamma_{\varphi,res}} \end{cases} \qquad (16)$$

where:

$\varphi'_{pk,k}$ and γ_{φ}	are as defined for equation (14);
$\varphi'_{res,k}$	is the characteristic value of the soil's residual angle of shearing resistance; and
$\gamma_{\varphi,res}$	is a partial factor whose value is 1.0.

4.7 Serviceability limit states

*COMMENTARY ON **4.7***

A serviceability limit state is a condition beyond which specific service requirements for the structure or foundation are no longer met. Serviceability requirements for foundations are commonly expressed as limiting criteria for settlement or heave.

4.7.1 The serviceability limit state design of a foundation should conform to BS EN 1997-1:2004+A1:2013, **2.4.8**, **9.2** and **9.8**.

4.7.2 The terminology used to describe foundation movements should conform to BS EN 1997-1:2004+A1:2013, Annex H(1) and Figure H.1.

4.7.3 Damage to masonry walls owing to ground movement should be classified in accordance with *Behaviour of foundations and structures* [N1].

NOTE Guidance on building response to ground movements can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 26 *[2].*

4.7.4 Particular consideration should be given to differential settlement, since this is normally more damaging to a completed structure than total settlement.

4.7.5 Consideration should be given to any adverse effect that total or differential foundation settlement might have on services entering the structure.

4.7.6 Consideration should also be given to the adverse effect of differential settlement on services, pavements, and rail tracks, particularly at discontinuities, noting that part of the settlement normally occurs during construction.

4.7.7 Continuous structures that are sensitive to settlement should be avoided when large differential settlements are expected.

NOTE Structures may be split into a number of smaller independent units to minimize the effect of differential settlement between them. The stiffness of a framed structure may be reduced by modifying any cross bracing or altering the connections between the frame and the cladding.

4.8 Structural design

4.8.1 The structural design of concrete foundations should conform to BS EN 1992, except where stated otherwise in this standard (see **6.9**).

4.8.2 The structural design of steel foundations should conform to BS EN 1993.

4.8.3 The structural design of timber foundations should conform to BS EN 1995.

4.9 Execution

NOTE For guidance on archaeological finds, see Annex D.

4.9.1 General

4.9.1.1 The execution of concrete foundations should conform to BS EN 13670.

4.9.1.2 The execution of steel foundations should conform to BS EN 1090.

4.9.1.3 The execution of pile foundations should also conform to **6.10**.

4.9.1.4 The execution of underpinning should also conform to **B.9**.

4.9.1.5 The execution of helical steel pile foundations should also conform to **A.7**.

4.9.1.6 When a design necessitates a particular sequence of operations, these should be clearly indicated on the drawings or in the specification.

NOTE 1 Information about construction processes can be found in the ICE manual of geotechnical engineering (2012), Volume II, Section 8 *[1].*

NOTE 2 Attention is drawn to The Construction (Design and Management) Regulations 2015 *[3], with regards to health and safety requirements for construction works, in particular Regulations 13 and 15 which deal with duties of contractors.*

4.9.2 Temporary works

4.9.2.1 The design and construction of temporary excavations, trenches, pits and shafts should conform to BS 6031.

4.9.2.2 The procedural controls that should be applied to all aspects of temporary works should conform to BS 5975.

4.9.3 Working platforms

*COMMENTARY ON **4.9.3***

Working platforms are temporary structures that provide a foundation for heavy construction plant. Although temporary, these are safety critical structures since bearing failure will have serious consequences in the event of overturning of the construction plant.

It is important that working platforms are designed, installed, and operated appropriately for their intended use.

4.9.3.1 The design of temporary works should conform to PAS 8811 and PAS 8812. [13)]

4.9.3.2 Drilling and foundation equipment used for temporary works should conform to BS EN 16228.

NOTE 1 Geosynthetics incorporated into the construction of granular working platforms might provide beneficial effects that enhance the stability of the working platform.

NOTE 2 Guidance on ground conditions for construction plant can be found in Ground conditions for construction plant *[35].*

NOTE 3 Guidance on the design, installation, maintenance, and repair of ground-supported working platforms for tracked plant can be found in BRE Report 470 *[36].*

NOTE 4 Guidance on the design of geosynthetics can be found in CIRIA SP 123 *[37] and in manufacturers' publications.*

4.9.4 Ground improvement

4.9.4.1 The execution of deep mixing should conform to BS EN 14679.

4.9.4.2 The execution of grouting should conform to BS EN 12715.

4.9.4.3 The execution of jet grouting should conform to BS EN 12716.

4.9.4.4 The execution of hydraulically bound mixtures should conform to BS EN 14227.

4.9.4.5 The execution of deep vibration techniques (including vibro-compaction and vibro stone columns) should conform to BS EN 14731.

4.9.4.6 The execution of vertical drainage (including vertical band drains) should conform to BS EN 15237.

4.10 Testing

4.10.1 Tests on foundations should conform to prEN ISO 22477, where appropriate.

4.10.2 Testing of pile foundations should also conform to **6.11**.

4.11 Supervision, monitoring, and maintenance

4.11.1 Supervision of construction

Supervision of construction of foundations should conform to BS EN 1997-1:2004+A1:2013, **4.2**.

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13) PAS 8811 and PAS 8812 are expected to be published in 2015.

NOTE Guidance on technical supervision of site works can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 96 *[1].*

4.11.2 Monitoring

4.11.2.1 Monitoring of foundations should conform to BS EN 1997-1:2004+A1:2013, **4.5**.

4.11.2.2 Where serviceability criteria have been specified, or it is otherwise appropriate, the settlement and deformation of the following should be monitored systematically, in order to assess their performance:

- the supported structure;
- ground surface;
- adjacent infrastructure; and
- any buried infrastructure or utilities.

4.11.2.3 Where appropriate, instrumentation should be installed to:

- confirm design assumptions and check the predicted behaviour of the foundation; and
- confirm that the structure continues to perform as required following construction.

4.11.2.4 Monitoring needed to implement the observational method should conform to BS EN 1997-1.

NOTE Guidance on the principles of geotechnical monitoring can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 94 *[1].*

4.11.3 Maintenance

4.11.3.1 Maintenance of foundations should conform to BS EN 1997-1:2004+A1:2013, **4.6**.

4.11.3.2 If the design of a foundation relies on a dewatering system, a maintenance programme for dewatering should be specified.

4.12 Reporting

*COMMENTARY ON **4.12***

*BS EN 1997-1:2004+A1:2013, **2.8**(1)P, requires the "assumptions, data, methods of calculation and result of the verification of safety and serviceability" to be recorded in the Geotechnical Design Report (GDR).*

*Additionally, BS EN 1997-1:2004+A1:2013, **3.4.1**(1)P, requires the results of a geotechnical investigation to be compiled in a Ground Investigation Report (GIR), which "shall form a part of the Geotechnical Design Report'".*

This standard extends this reporting regime to include a Geotechnical Feedback Report (GFR) that contains full records of the works constructed. These as-built records include information that will assist with future maintenance, design of additional works, and decommissioning of the works. The GFR could also go some way to satisfying the requirements of CDM Regulations *[3] with regards to preparation of a health and safety file.*

None of these reports is the same as a geotechnical baseline report, which may be used on geotechnical projects for contractual purposes.

4.12.1 Ground Investigation Report

The Ground Investigation Report for a foundation should conform to BS EN 1997-1:2004+A1:2013, **3.4**.

4.12.2 Geotechnical Design Report

The Geotechnical Design Report for a foundation should conform to BS EN 1997-1:2004+A1:2013, **2.8**.

4.12.3 Geotechnical Feedback Report

*COMMENTARY ON **4.12.3***

The Geotechnical Feedback Report is also known as a "close-out report".

4.12.3.1 On completion of the works, a Geotechnical Feedback Report (GFR) should be prepared that covers the following broad classes of information:

- a record of construction and any changes to its design; and
- results of monitoring and testing conducted during construction.

4.12.3.2 The GFR should be tailored to suit the size and complexity of the works.

4.12.3.3 The record of construction should include, as appropriate:

- a general description of the works, including ground and groundwater conditions encountered;
- instability problems, unusual ground conditions, and groundwater problems, including measures to overcome them;
- contaminated and hazardous material encountered on site and the location of disposal, both on and off site;
- temporary works and foundation treatment, including drainage measures and treatment of soft areas and their effectiveness;
- types of imported and site-won materials and their use;
- any aspect of the specification or standards used that should be reviewed in view of problems encountered on site;
- any requirements for ongoing monitoring or abnormal maintenance requirements;
- any unexpected ground conditions that required changes to design;
- problems not envisaged in the Geotechnical Design Report and the solutions to them; and
- as-built drawings.

4.12.3.4 The results of monitoring and testing should include:

- details of any in-situ testing;
- test logs and test results;
- summary of site laboratory testing;
- location and details of instruments;
- readings from instruments (with dates) and predicted values;
- the results of compliance testing (e.g. in-situ density measurement, plate load tests); and
- data from monitoring instruments (e.g. piezometers, inclinometers, settlement gauges).

NOTE 1 The preparation of the GFR is particularly important where the observational method of design has been used.

NOTE 2 Guidance on the preparation of close-out reports can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 101 *[1].*

4.12.3.5 A copy of the Geotechnical Feedback Report should be provided to the owner/client.

5 Spread foundations

*COMMENTARY ON Clause **5***

This clause applies to the design and construction of:

- *pad foundations;*
- *strip foundations;*
- *raft foundations;*

Underpinning is covered in Annex B.

5.1 Choice and design of spread foundations

5.1.1 General

5.1.1.1 The design of spread foundations should conform to BS EN 1997-1:2004+A1:2013, Clause **6**, and Clause **4** and Clause **5** (this clause) of this standard.

NOTE Information about shallow (i.e. spread) foundations can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 53 *[1].*

5.1.1.2 Selection of a suitable type of spread foundation should take into account:

- the magnitude and disposition of structural loads;
- the bearing resistance of the ground;
- the settlement characteristics of the ground;
- the differential settlement that the supported structure can tolerate; and
- the need to found in stable soil.

NOTE Adjacent pad foundations may be combined or joined together with ground beams to support eccentric loads, to resist overturning, or to oppose horizontal forces. Walls between columns may be carried on ground beams spanning between pad foundations.

5.1.1.3 When the plan area covered by pad and strip foundations exceeds more than about one half of the superstructure footprint, consideration should be given to using a raft foundation instead.

NOTE Some key features of pad, strip, and raft foundations are summarized in Table 6.

Table 6 **Some key features of spread foundations**

Type	**Shape on plan**	**Aspect ratio**	**Used to support:**	**Constructed in:**
Pad	Square, circular, or rectangular	L˜ B	One or two columns	Mass or reinforced concrete
Strip	Rectangular	L >> B	Load-bearing wall or several closely-spaced columns	Mass or reinforced concrete
Raft	Square or rectangular	L˜ B	Entire structure or a substantial part of it	Reinforced concrete

5.1.2 Pad foundations

*COMMENTARY ON **5.1.2***

A pad foundation is a spread foundation whose length (L) and breadth (B) on plan are of similar magnitude (L˜ B). Pad foundations are commonly used to transmit structural loads onto a suitable bearing stratum at shallow depth below ground level. They are the simplest of all foundations and are generally used where groundwater is absent or can be readily controlled.

5.1.2.1 Pad foundations may be of various shapes, including circular, square, or rectangular. The shape of a pad foundation should be chosen to accommodate the effect of eccentricity arising from imposed moments and shear forces on the column and the method of construction.

NOTE Deep pad foundations (>3 m deep) may be used to carry heavy column loads.

5.1.2.2 The thickness of a pad foundation should not be less than 150 mm.

5.1.3 Strip foundations

*COMMENTARY ON **5.1.3***

A strip foundation is a spread foundation whose length on plan (L) is very much greater than its breadth (B), i.e. L >> B. Strip foundations are commonly used to support the walls of buildings. They are generally used where groundwater is absent or can be readily controlled.

5.1.3.1 On sloping sites, strip foundations should be founded on a horizontal bearing and stepped where necessary to maintain adequate depth.

5.1.3.2 The thickness of a strip foundation should be not less than 150 mm.

5.1.3.3 The breadth of a strip foundation should be chosen, taking into account normal construction tolerances (see **5.2**), so as not to overstress the ground beneath it.

5.1.4 Raft foundations

*COMMENTARY ON **5.1.4***

A raft foundation is a spread foundation whose length (L) and breadth (B) on plan are similar to that of the superstructure. Raft foundations are commonly used to support large or heavily loaded structures. They can prove to be a very cost-effective foundation solution.

5.1.4.1 Raft foundations may be used to support:

- lightly loaded structures on low strength natural ground, where it is necessary to spread the load across horizontally variable ground, fill, or weaker zones;
- structures that are sensitive to differential settlement;
- structures on ground where mining or other forms of subsidence are likely to occur and are thus a flexible structure and foundation is required;
- low rise dwellings and lightly framed structures on soils that are susceptible to excessive shrinking or swelling; or
- heavy structures that could otherwise be supported on many isolated foundations, occupying a large part of the structure’s footprint, when a more economic design is required

5.1.4.2 Settlement of raft foundations may be reduced by providing piles that redistribute loads from the upper layers of the ground to deeper strata.

NOTE Information about rafts can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 56 *[1].*

5.2 Actions and design situations

The design of concrete spread foundations cast directly onto the ground should take into account the permitted sectional deviations given in BS EN 13670.

5.3 Design considerations

COMMENTARY ON ***5.3***

Design and construction considerations for spread foundations are given in BS EN 1997-1:2004+A1:2013, ***6.4****. The design considerations given in this clause are more specific examples of the issues that can affect the performance of a spread foundation.*

5.3.1 General

5.3.1.1 The design of spread foundations should consider:

- the items listed in BS EN 1997-1:2004+A1:2013, **6.4** and in **4.2.2** of this standard;
- when relevant, the additional design considerations for spread foundations on rock given in BS EN 1997-1:2004+A1:2013, **6.7**;
- changes in groundwater conditions;
- long-term stability;
- sensitive clays and loose water-bearing sands and soils that change structure when loaded;
- the effect of any excavation on soil properties, particularly for foundations greater than 3 m deep;
- in low strength soils particularly, the potential for damage to services and drains caused by relative movement between the foundation and the ground when they are supported partly on each;
- the layout and design of service pipes and ducts to allow for their future maintenance without the need to break through the foundation;
- the potential for underground services, such as drains and water mains, to be damaged by shrinkage and swelling of clay soils;
- in connection with extensions of existing buildings:
  - differential movement of the foundations between new and existing structures;
  - where cracking and subsequent remedial work is not acceptable, provision of joints between extensions and existing buildings;
  - stability of existing foundations where they abut the foundations of an extension; and
  - the possibility of damage caused by the presence of chemicals.

5.3.1.2 Specialist advice should be sought where sensitive clays and loose water-bearing sands and soils that change structure when loaded are expected.

5.3.1.3 Whenever possible, the centre of area of a foundation or group of foundations should be located directly beneath the centre of gravity of the imposed load. When this is not possible, the effects on the structure of tilting and settlement of the foundation should be considered.

5.3.1.4 Where foundation support is provided by a number of separate bases these should, as far as practicable, be proportioned so that differential settlement is minimal.

5.3.2 Depth of foundation

5.3.2.1 Strip foundations of traditional brick and masonry buildings should be founded at a depth where the anticipated ground movements will not impare the stability or serviceability of any part of the building, taking due consideration of soil type and the influence of vegetation and trees on the ground.

5.3.2.2 The depth to the underside of foundations should be not less than 750 mm on low shrinkage clay soils, 900 mm on medium shrinkage clay soils, and 1 000 mm on high shrinkage clay soils. These depths might need to be increased in order to transfer the load onto satisfactory ground or where trees are located nearby.

NOTE 1 Information about low-rise buildings on shrinkable clay soils can be found in BRE Digests 240 *[38] and* 241 *[39].*

NOTE 2 Attention is drawn to the Building Regulations 2010 *[40], which state that "except where strip foundations are founded on rock, the strip foundations should have a minimum depth of 450 mm to their underside to avoid the action of frost. This depth, however, will commonly need to be increased in areas subject to long periods of frost or in order to transfer the loading onto satisfactory ground. "*

5.3.3 Trees and other vegetation

Consideration of trees and other vegetation in the design of spread foundations should conform to **4.2.3.4**.

5.3.4 Frost heave

*COMMENTARY ON **5.3.4***

Traditionally in the UK, foundations have been taken to a depth of at least 450 mm below ground level to avoid frost heave.

The design of foundations to avoid frost heave should conform to BS EN ISO 13793.

5.3.5 Heating

5.3.5.1 The design of a shallow foundation should take into account possible ground movement due to shrinkage of clay caused by:

- boiler installations;
- furnaces and kilns;
- underground cables and services;
- ground storage energy systems;
- other artificial sources of heat; and
- fill or other soils containing combustible materials.

5.3.5.2 Where excessive heat would otherwise be transmitted, the installation should be isolated from the soil and the foundation by a suitable form of construction. Where the installation is small, insulating materials might be adequate but some form of forced ventilation or cooling by circulated water might be required.

5.4 Calculation models

5.4.1 Bearing resistance - Models based on ground parameters

*COMMENTARY ON **5.4.1***

BS EN 1997-1:2004+A1:2013, Annex D, presents a sample analytical method for bearing resistance calculation that does not include depth, ground inclination, or rigidity factors. This clause gives an alternative method that includes these factors.

5.4.1.1 General

The design of spread foundations from ground parameters should conform to BS EN 1997-1:2004+A1:2013, **6.5.2.2.**

5.4.1.2 Coarse soils (sands and gravels)

5.4.1.2.1 As an alternative to the sample analytical method given in BS EN 1997-1:2004+A1:2013, **D.4**, the following expression may be used to calculate the ultimate bearing resistance of a spread foundation (R_v) on coarse soils using effective stress parameters:

$$R'_v/A' = c'N_c b_c s_c i_c d_c g_c r_c + q'N_q b_q s_q i_q d_q g_q r_q + 0.5\gamma' B' N_\gamma b_\gamma s_\gamma i_\gamma d_\gamma g_\gamma r_\gamma \qquad (17)$$

where:

c', q', and γ'	are as defined in BS EN 1997-1:2004+A1:2013, **D.4**;
B'	is defined in BS EN 1997-1:2004+A1:2013, **D.4**; and

the various coefficients N_c, b_c, s_c, etc. are defined below, after Poulos *et al.* [41]

NOTE Care is required in assessing submerged weight density term γ', particularly when groundwater is close to the foundation.

5.4.1.2.2 The bearing coefficients in equation (17) should be calculated from:

$$N_q = e^{\pi \times \tan\varphi}\tan^2(45° + \varphi/2)$$

$$N_\gamma = \begin{cases} a \times e^{b \times \varphi} & \text{for } \varphi > 0° \\ 0 & \text{for } \varphi = 0° \end{cases}$$

$$N_c = (N_q - 1)\cot\varphi \qquad (18)$$

5.4.1.2.3 The shape factors in equation (17) should be calculated from:

$$s_q = 1 + (B/L)\tan\varphi$$

$$s_\gamma = 1 - 0.4(B/L)$$

$$s_c = 1 + (B/L)(N_q/N_c) \qquad (19)$$

5.4.1.2.4 The load inclination factors in equation (17) should be calculated from:

$$i_q = \left[1 - \left(\frac{H}{V + A'c'\cot\varphi}\right)\right]^m$$

$$i_\gamma = \left[1 - \left(\frac{H}{V + A'c'\cot\varphi}\right)\right]^{m+1}$$

$$i_c = \begin{cases} i_q - \left(\dfrac{1 - i_q}{N_c \tan\varphi}\right) & \text{for } \varphi > 0° \\ 1 - \left(\dfrac{mH}{c'N_c A'}\right) & \text{for } \varphi = 0° \end{cases} \qquad (20)$$

5.4.1.2.5 The base inclination (also known as "foundation tilt") factors in equation (17) should be calculated from:

$$b_{\mathrm{q}} \approx b_{\gamma}$$

$$b_{\gamma} = \left(1 - \alpha \tan\varphi\right)^2$$

$$b_{\mathrm{c}} = \begin{cases} b_{\mathrm{q}} - \left(\dfrac{1 - b_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi}\right) & \text{for} \quad \varphi > 0° \\ 1 - \left(\dfrac{2\alpha}{\pi + 2}\right) & \text{for} \quad \varphi = 0° \end{cases} \tag{21}$$

5.4.1.2.6 The ground inclination (also known as "surface inclination") factors in equation (17) should be calculated from:

$$g_{\mathrm{q}} = \begin{cases} \left(1 - \tan\omega\right)^2 & \text{for} \quad \varphi > 0° \\ 1 & \text{for} \quad \varphi = 0° \end{cases}$$

$$g_{\gamma} \approx g_{\mathrm{q}}$$

$$g_{\mathrm{c}} = \begin{cases} g_{\mathrm{q}} - \left(\dfrac{1 - g_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi}\right) & \text{for} \quad \varphi > 0° \\ 1 - \left(\dfrac{2\omega}{\pi + 2}\right) & \text{for} \quad \varphi = 0° \end{cases} \tag{22}$$

5.4.1.2.7 The depth factors in equation (17) should be calculated from:

$$d_{\mathrm{q}} = 1 + 2\tan\varphi\left(1 - \sin\varphi\right)^2 \tan^{-1}\left(D/B\right)$$

$$d_{\gamma} = 1$$

$$d_{\mathrm{c}} = \begin{cases} d_{\mathrm{q}} - \left(\dfrac{1 - d_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi}\right) & \text{for} \quad \varphi > 0° \\ 1 + 0.33\tan^{-1}\left(D/B\right) & \text{for} \quad \varphi = 0° \end{cases} \tag{23}$$

5.4.1.2.8 The rigidity factors in equation (17) should be calculated from:

$$r_{\mathrm{q}} = \mathrm{e}^{\left[\left(-4.4 + 0.6B/L\right)\tan\varphi + \frac{3.07\sin\varphi \log_{10} 2I_{\mathrm{r}}}{1 + \sin\varphi}\right]}$$

$$r_{\gamma} = r_{\mathrm{q}}$$

$$r_{\mathrm{c}} = \begin{cases} r_{\mathrm{q}} - \left(\dfrac{1 - r_{\mathrm{q}}}{N_{\mathrm{c}} \tan\varphi}\right) & \text{for} \quad \varphi > 0° \\ 0.32 + 0.12\left(B/L\right) + 0.60\log_{10} I_{\mathrm{r}} & \text{for} \quad \varphi = 0° \end{cases} \tag{24}$$

where:

φ	is angle of shearing resistance of the soil;
a	is 0.0663 for a smooth foundation or 0.1054 for a rough foundation;
b	is 9.3 or 9.6 for a smooth or rough foundation, respectively (when φ is entered in radians); alternatively, b = 0.162 or 0.168, respectively (when φ is entered in degrees);
B	is the breadth of the foundation on plan;
L	is the length of the foundation on plan;
D	is the depth to the underside of the foundation;
A'	is the effective area of the foundation;

H	is the horizontal force applied to the foundation;
V	is the vertical force applied to the foundation;
m	is $(2 + B/L) / (1 + B/L)$ for loading in the direction of B or $(2 + L/B) / (1 + L/B)$ for loading in the direction of L;
α	is the inclination of the underside of the footing from the horizontal;
ω	is the inclination of the ground surface below the horizontal in the direction away from the foundation;
I_r	is $G / (c' + \sigma'_v \tan \varphi)$;
G	is the soil's shear modulus of elasticity;
c'	is the soil's effective cohesion; and
σ'_v	is the vertical effective stress on the foundation.

NOTE 1 The expressions given for N_c and N_q are identical to those in BS EN 1997-1:2004+A1:2013, Annex D. The expressions given for the shape, load inclination, and base inclination factors are also identical to those in Annex D, except for s_q and s_γ. The expressions given for d_q, g_q, and r_q, etc. are missing from Annex D.

NOTE 2 The expression given for N_γ is one of many that have been proposed in the geotechnical literature. The expression given (after Poulos [41]) is generally conservative when compared to other expressions.

5.4.1.2.9 The effects of load combinations involving large inclinations of force or large moments should be assessed using more advanced calculation models than those given in BS EN 1997-1:2004+A1:2013, **D.4**, or this clause.

NOTE Information about more advanced bearing resistance models can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 21 [2].

5.4.1.2.10 A concrete foundation cast directly against the ground may be considered "rough"; pre-cast concrete foundations should be considered "smooth".

5.4.1.3 Fine soils (silts and clays)

5.4.1.3.1 The ultimate bearing resistance of a spread foundation on fine soils should be calculated as the smaller of its undrained and drained bearing resistances.

5.4.1.3.2 Under undrained (usually short term) conditions, the ultimate bearing resistance of a spread foundation on fine soils should typically be calculated using total stress parameters.

5.4.1.3.3 Under drained (usually long term) conditions, the ultimate bearing resistance of a spread foundation on fine soils should be calculated from effective stress parameters, as described in **5.4.1.2**.

5.4.1.3.4 As an alternative to the sample analytical method given in BS EN 1997-1:2004+A1:2013, **D.3**, the following expressions (after Salgado *et al.* [42]) may be used, for design situations in which there is no load or ground inclination, to calculate the ultimate bearing resistance of a spread foundation (R_v) on fine soils using total stress parameters:

$$R_v/A' = N_c c_u s_c d_c + q_{vb}$$

$$N_c = \pi + 2$$

$$s_c = 1 + 0.21\frac{B}{L} + 0.17 \times \sqrt{\frac{D}{B}}$$

$$d_c = 1 + 0.27 \times \sqrt{\frac{D}{B}} \qquad (25)$$

where:

A'	is the effective plan area of the foundation;
c_u	is the undrained shear strength of the fine soil;
s_c	is shape factor;
d_c	is depth factor;
q_{vb}	is total overburden pressure at the underside of the foundation;
B	is the breadth of the foundation;
L	is the length of the foundation; and
D	is the depth to the underside of the foundation.

5.4.1.3.5 This expression should not be used if the ground surface, the applied load, or the base of the foundation is inclined.

NOTE 1 When the ground surface is inclined, omission of a ground inclination factor from the expression for R_v above is potentially unsafe.

NOTE 2 When the load is inclined, omission of a load inclination factor from the expression for R_v above is potentially unsafe.

NOTE 3 When the base is inclined, omission of a base inclination factor from the expression for R_v above is potentially unsafe.

5.4.2 Sliding resistance

Any earth pressure that is included in the sliding resistance of a spread foundation should be calculated at a strain level that is compatible with that assumed in the calculation of shear resistance along the foundation base.

5.4.3 Settlement

*COMMENTARY ON **5.4.3***

The magnitude of the settlement that will occur when foundation loads are applied to the ground depends on the rigidity of the structure, the type and duration of the loading, and the deformation characteristics of the ground.

The settlement of a spread foundation may be calculated using any of the following, as appropriate:

- methods based on the theory of elasticity;
- methods based on one-dimensional consolidation;
- for foundations on sand, the empricial method given in BS EN 1997-2:2007, **F.3**;
- methods using non-linear stress-strain models; or
- numerical models.

NOTE Guidance on the calculation of settlement for a shallow (i.e. spread) foundation can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 53 *[1].*

5.4.4 Combined bearing and settlement check using prescriptive measures

*COMMENTARY ON **5.4.4***

The design of many simple foundations has traditionally been checked against "allowable bearing pressures" which are normally very conservative estimates of the ultimate bearing resistance of the ground, selected on the basis of soil and rock descriptions. The settlement of a spread foundation that has been designed using allowable bearing pressures is commonly assumed to be acceptable.

In BS EN 1997-1, "allowable bearing pressures" are now called "presumed bearing resistance" and this method of design is termed a "prescriptive method".

5.4.4.1 General

Spread foundations may be designed using prescriptive methods based on presumed bearing resistance.

5.4.4.2 Presumed bearing resistance of coarse soils

5.4.4.2.1 Suggested values for the presumed design unit bearing resistance ($q_{\mathrm{Rv,pres,d}}$) of spread foundations on coarse soils, and located a distance above the water table at least equal to the foundation's breadth, may be estimated from:

$$q_{\mathrm{Rv,pres,d}} = \frac{0.5 \times N_{\gamma \mathrm{k}} \times B \times \gamma_{\mathrm{s,k}}}{\gamma_{\mathrm{Rv,SLS}}} \tag{26}$$

where:

$N_{\gamma,\mathrm{k}}$ is a bearing coefficient based on equation (18), using the characteristic angle of shearing resistance of the soil;

B is the breadth of the foundation;

$\gamma_{\mathrm{s,k}}$ is the characteristic weight density of the soil; and

$\gamma_{\mathrm{Rv,SLS}}$ is a partial factor on bearing resistance.

5.4.4.2.2 For foundations not exceeding 1 m in width that are subject primarily to permanent laoding, a value of $\gamma_{\mathrm{Rv,SLS}} \geq 2.0$ should be adopted.

5.4.4.3 Presumed bearing resistance of fine soils

5.4.4.3.1 Suggested values for the presumed design unit bearing resistance ($q_{\mathrm{Rv,pres,d}}$) of fine soils may be estimated from the expression:

$$q_{\mathrm{Rv,pres,d}} = \frac{(\pi + 2) \times c_{\mathrm{u,k}}}{\gamma_{\mathrm{Rv,SLS}}} \tag{27}$$

where:

$c_{\mathrm{u,k}}$ is the characteristic undrained shear strength of the soil; and

$\gamma_{\mathrm{Rv,SLS}}$ is a partial factor on bearing resistance.

5.4.4.3.2 For foundations not exceeding 1 m in width that are subject primarily to permanent loading, a value of $\gamma_{\mathrm{Rv,SLS}} \geq 3.0$ should be adopted.

5.4.4.4 Presumed bearing resistance of rocks

Suggested values for the presumed design unit bearing resistance ($q_{Rv,pres,d}$) of square pad foundations on rock (for settlements not exceeding 0.5% of the foundation width) may be obtained from BS EN 1997-1:2004+A1:2013, Annex G.

5.5 Materials

5.5.1 Concrete

Concrete and related products incorporated into spread foundations should conform to **4.3.6**.

5.5.2 Steel

Steel and related products incorporated into spread foundations should conform to **4.3.7**.

5.5.3 Timber

Timber and related products incorporated into spread foundations should conform to **4.3.8**.

5.5.4 Fill

Fill incorporated into spread foundations should conform to **4.3.3**.

5.6 Durability

5.6.1 Concrete

The durability of concrete used in spread foundations should conform to **4.4.2**.

5.6.2 Steel

The durability of steel used in spread foundations should conform to **4.4.3**.

5.6.3 Timber

The durability of timber used in spread foundations should conform to **4.4.4**.

5.7 Ultimate limit state design

5.7.1 General

5.7.1.1 The ultimate limit state design of spread foundations should conform to **4.6** and this subclause (**5.7**).

5.7.1.2 The ultimate limit state design of a foundation should conform to BS EN 1997-1:2004+A1:2013, **6.5**.

5.7.2 Bearing

The design value of the ultimate bearing resistance of a spread foundation (R_d) should be verified according to BS EN 1997-1:2004+A1:2013, **6.5.2**, and conform to expression (6.1) of that standard.

5.7.3 Sliding

5.7.3.1 The design value of the ultimate sliding resistance of a spread foundation (R_d) should be verified according to BS EN 1997-1:2004+A1:2013, **6.5.3**, and conform to expression (6.2) of that standard.

5.7.3.2 For drained conditions, R_d should be calculated according to expression (6.3a) of BS EN 1997-1:2004+A1:2013 in preference to (6.3b).

5.7.3.3 For undrained conditions, R_d should be calculated according to expression (6.4a) of BS EN 1997-1:2004+A1:2013 in preference to (6.4b).

5.7.4 Overturning

Overturning of a spread foundation should be prevented by verifying ultimate limit state EQU in accordance with BS EN 1997-1:2004+A1:2013, **2.4.7.2** and the UK National Annex to BS EN 1997-1:2004+A1:2013.

5.7.5 Global stability

The global stability of a spread foundation should conform to BS EN 1997-1:2004+A1:2013, **6.5.1**.

5.8 Serviceability limit state design

The serviceability limit state design of a foundation should conform to BS EN 1997-1:2004+A1:2013, **6.6**.

5.9 Structural design

5.9.1 General

5.9.1.1 The structural design of a foundation should conform to BS EN 1997-1:2004+A1:2013, **6.8**.

5.9.1.2 Spread foundations may be constructed using reinforced or plain (i.e. unreinforced or mass) concrete.

5.9.1.3 The design of reinforced concrete spread foundations should conform to BS EN 1992-1-1.

5.9.1.4 The design of plain (i.e. unreinforced) concrete spread foundations should conform to BS EN 1992-1-1:2004+A1:2014, Section 12.

5.9.1.5 Foundations that act as retaining walls should conform to BS 8002, particularly when:

- founded on sloping ground; or
- steps occur between adjacent ground floor slabs or finished ground levels.

5.9.2 Pad foundations

The thickness of the foundation should not be less than 150 mm.

5.9.3 Strip foundations

Reinforcement should be provided in strip foundations wherever an abrupt change in load or variation in ground support occurs.

5.9.4 Raft foundations

The structural design of a raft foundation should take into account the reduction in strength caused by holes, ducts, etc. used to accommodate service pipes, drains, and such like.

5.10 Execution

NOTE *Attention is drawn to Regulation 22 of* The Construction (Design and Management) Regulations, 2015 *[3], with regards to health and safety requirements for excavations.*

The execution of concrete spread foundations should conform to **4.9**.

5.11 Testing

5.11.1 Testing of a spread foundation should conform to prEN ISO 22477.

5.11.2 Plate loading tests should conform to prEN ISO 22476-13.

5.12 Supervision, monitoring, and maintenance

Supervision, monitoring, and maintenance of a spread foundation should conform to **4.11**.

5.13 Reporting

Reports for spread foundations should conform to **4.12.**

6 Pile foundations

*COMMENTARY ON Clause **6***

This clause applies to the design and construction of:

- *bored cast-in-place concrete piles;*
- *driven cast-in-place concrete piles;*
- *prefabricated piles, made of concrete, steel, cast iron, or timber;*
- *micropiles;*
- *helical steel piles;*
- *piled underpinning;*
- *pile groups.*

6.1 Choice and design of pile foundations

6.1.1 General

6.1.1.1 The design of pile foundations should conform to BS EN 1997-1:2014+A1:2013, Section 7, and Clause **4** and Clause **6** (this clause) of this standard.

NOTE Information about different types of bearing piles can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 81 *[1].*

6.1.1.2 Piles may be classified according to the ground disturbance caused by their installation, as shown in Table 7.

Table 7 **Classification of piles according to ground disturbance caused by installation**

Pile class	Examples	Classification A) for the purpose of selecting partial factors from BS EN 1997-1
High displacement B)	Driven cast-in-place concrete piles Precast concrete piles of solid section Closed-ended tubular steel piles Timber piles	Driven
Low displacement B)	Displacement auger piles (not screw piles) C) Steel bearing piles of H-section D) Open-ended tubular steel piles D) Helical steel piles	Driven
Replacement	Bored cast-in-place concrete piles (installed using a continuous flight auger [CFA])	CFA
	Bored cast-in-place concrete piles (installed using casing and drilling tools) Micro piles	Bored

A) BS EN 1997-1 provides partial factors for driven, CFA, and bored piles only.
B) The words "high" and "low" here refer to the degree of disturbance of the ground during installation.
C) May be classified as large displacement in dense, coarse soils or stiff, fine soils.
D) If these piles plug, then they should be re-classified as large displacement piles.

6.1.2 Bored cast-in-place concrete piles

Bored cast-in-place concrete piles may be constructed in supported or unsupported borings that are created by rotary drilling or continuous flight auger (CFA) equipment.

6.1.3 Driven cast-in-place concrete piles

Driven cast-in-place concrete piles may be constructed with or without permanent casing.

6.1.4 Prefabricated piles

6.1.4.1 Precast concrete piles

*COMMENTARY ON **6.1.4.1***

Prefabricated concrete piles are also commonly termed "precast concrete piles". Precast concrete piles may be reinforced or prestressed.

6.1.4.1.1 Precast concrete piles should be manufactured in accordance with BS EN 12794.

6.1.4.1.2 Precast concrete piles should be formed from one of the following sectional shapes:

- prismatic section;
- circular section with a solid core; or
- circular section with a hollow core.

6.1.4.1.3 The cross-section of a precast concrete pile may be constant or tapered over all or part of its length.

6.1.4.1.4 Precast concrete piles may be manufactured in single lengths or in segments connected with cast-in joints.

6.1.4.1.5 Precast concrete piles should be manufactured using normal weight concrete conforming to BS EN 206.

6.1.4.2 Steel bearing piles

Steel bearing piles may be formed from the following sectional shapes:

- H-sections;
- pipes; or
- box piles.

6.1.4.3 Helical steel piles

Helical steel piles (also known as "screw piles") should conform to Annex A.

6.1.4.4 Timber piles

NOTE Guidance on the design and installation of timber piling can be found in BRE Digest 479 [43].

6.1.5 Micro piles

*COMMENTARY ON **6.1.5***

Although there is an execution standard (BS EN 14199), there is currently no detailed UK specification for micro piles. Project-specific specifications – based on the principles set out in SPERW [N3], but adapted for the particulars of micro piles – are commonly used as a substitute.

Micro piles should be manufactured in accordance with BS EN 14199.

NOTE 1 See Guide to drafting a specification for micropiles *[44].*

NOTE 2 The website of the International Society for Micropiles [45] lists 50 papers on micro piles (but does not provide a standard specification).

NOTE 3 See Guidelines on safe and efficient underpinning and mini piling operations *[46].*

6.1.6 Piled underpinning

NOTE Guidance on underpinning can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 83 *[1].*

6.1.7 Pile groups

*COMMENTARY ON **6.1.7***

Pile groups are often used when the bearing resistance of individual piles is insufficient to carry load from the structure. Although pile groups provide greater bearing resistance, they might do so less efficiently than indivudal piles, leading to greater settlement during service owing to interaction between the piles in the group.

Group effects may be ignored when there are fewer than five piles in the group and the piles are spaced no closer than three times their diameter centre-to-centre.

NOTE Guidance on pile-group design can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 55 *[1].*

6.2 Actions and design situations

6.2.1 The design of concrete pile foundations should take into account the construction tolerances given in the relevant execution standard, as follows:

- for bored piles, BS EN 1536;
- for micropiles, BS EN 14199; and
- for displacement piles, BS EN 12699.

6.2.2 Pile foundations should be classified as Geotechnical Category 2 or higher, as defined in BS EN 1997-1:2004+A1:2013, **2.1**.

6.3 Design considerations

6.3.1 General

6.3.1.1 The design of pile foundations should consider:

- the items listed in BS EN 1997-1:2004+A1:2013, **7.4.2**;
- the possibility of piles imperfections and the impact they will have on the structure;
- where large soil movements due to swelling or shrinkage of upper layers of the ground are expected, sleeving the upper part of the pile shaft or providing reinforcement to resist the resulting forces in the pile (the pile should also have sufficient shaft resistance below the zone of swelling soil to resist the uplift forces);
- influence of different soil/rock layers below the base of the piles;
- potential for lateral soil movements adjacent to excavation and fill areas in the vicinity of piles.

6.3.1.2 The design of a pile foundation should be modified as necessary to account for any significant variation from the expected pile behaviour during pile driving or from expected ground conditions during boring for cast-in-place piles.

6.3.2 Ground investigation

Ground investigation for pile foundations should conform to BS EN 1997-2 as well as the specific requirements of:

- BS EN 1536 for bored piles;
- BS EN 12699 for displacement piles; and
- BS EN 14199 for micro piles.

6.3.3 Pile spacing

*COMMENTARY ON **6.3.3***

The resistance of individual piles is reduced in the vicinity of other piles. It is common to limit this reduction in resistance by specifying a minimum spacing between the piles. The relevant spacing differs for "friction piles", which derive the majority of their bearing resistance from their shafts, and for "end-bearing piles", which derive the majority of their bearing resistance from their bases.

6.3.3.1 When viewed on plan, the centre-to-centre spacing (s) of "friction" piles should satisfy:

$$s \geq \begin{cases} 3D & \text{for circular piles} \\ P & \text{for non circular piles} \end{cases} \quad (28)$$

and that of end-bearing piles:

$$s \geq \begin{cases} 2D & \text{for circular piles} \\ 2P/3 & \text{for non circular piles} \end{cases} \quad (29)$$

where:

P is the perimeter of the larger of two adjacent piles; and

D is the outside diameter of the larger of two adjacent piles.

6.3.3.2 Closer pile spacings may be used if it can be shown that any increased settlement of the piles arising from their interaction does not lead to a limit state being exceeded.

6.3.3.3 Closer pile spacings may be used when the piles form part of an earth retaining structure.

6.3.3.4 The choice of pile spacing should take into account the pile installation method, particularly when dealing with driven piles.

6.4 Calculation models

*COMMENTARY ON **6.4***

BS EN 1997-1 allows the ultimate bearing resistance of an individual pile to be determined from any of the following:

- *static pile formulae based on ground parameters;*
- *direct formulae based on the results of field tests;*
- *the results of static pile load tests;*
- *the results of dynamic impact tests;*
- *pile driving formulae; and*
- *wave equation analysis.*

6.4.1 Bearing resistance

6.4.1.1 General

6.4.1.1.1 Unless otherwise stated, the total bearing resistance of an individual pile (R_t) should be calculated from:

$$R_t = R_s + R_b \tag{30}$$

where:

R_s is the resistance of the pile shaft; and

R_b is the resistance of the pile base.

6.4.1.1.2 The bearing resistance of a pile foundation should be determined for the the appropriate failure criterion corresponding to an ultimate limit state of the foundation.

*NOTE BS EN 1997-1:2004+A1:2013, **7.6.1.1(3)** recommends that settlement of the pile top equal to 10% of the pile base diameter be adopted as the failure criterion when it is difficult to define the ultimate limit state.*

6.4.1.1.3 The base resistance calculated from models that assume plunging failure of the pile should be reduced appropriately to match failure criterion appropriate to the ultimate limit state of the pile.

6.4.1.2 Models based on ground parameters

6.4.1.2.1 General

*COMMENTARY ON **6.4.2.1.1***

BS EN 1997-1 uses several terms that differ from traditional British practice:

- *"unit resistance" refers to resistance per unit area (i.e. over an area of 1 m²);*

- *"ultimate unit shaft resistance" (with symbol q_s) is equivalent to "limiting shaft (or skin) friction" (traditional symbol f_s); and*
- *"ultimate base resistance" is the same as "limiting end-bearing pressure".*

A static pile formula based on ground parameters provides an estimate of the ultimate bearing resistance of the pile. The method's accuracy depends on the reliability of the chosen formula, the soil strength data to which it is applied, and site-specific pile installation methods.

6.4.1.2.1.1 The ultimate bearing resistance of a pile foundation may be calculated from static pile formulae using values of ground parameters obtained from field or laboratory tests on soil and rock.

6.4.1.2.1.2 The consequences of differences between the true ultimate bearing resistance of a pile and its calculated value (which might occur, for example, owing to differences between actual and the assumed ground conditions) should be considered where reasonably foreseeable.

6.4.1.2.1.3 The calculation of ultimate compressive resistance from ground parameters should conform to the "alternative method" given in BS EN 1997-1:2004+A1:2013, **7.6.2.3**.

6.4.1.2.1.4 If this method is used, the characteristic ultimate compressive resistance of an individual pile ($R_{c,k}$) should be calculated as:

$$R_{c,k} = R_{s,k} + R_{b,k} \tag{31}$$

where:

$R_{s,k}$ is the pile's characteristic ultimate shaft resistance; and

$R_{b,k}$ is the pile's characteristic ultimate base resistance.

6.4.1.2.1.5 The characteristic ultimate shaft resistance ($R_{s,k}$) may be calculated from:

$$R_{s,k} = \frac{\sum_{j=1}^{n} \left(A_{s,j} \times q_{s,j}\right)}{\gamma_{Rd}} \tag{32}$$

where:

$A_{s,j}$ is the total circumferential area of the pile shaft (in layer j);

$q_{s,j}$ is the average ultimate unit shaft resistance (in layer j) calculated from ground parameters;

n is the total number of layers in contact with the pile shaft; and

γ_{Rd} is a model factor. The value of γ_{Rd} should conform to the UK National Annex to BS EN 1997-1:2004+A1:2013.

6.4.1.2.1.6 The characteristic ultimate base resistance ($R_{b,k}$) may be calculated from:

$$R_b = \frac{A_b \times q_b}{\gamma_{Rd}} \tag{33}$$

where:

A_b is the total cross-sectional area of the pile base;

q_b is the ultimate unit base resistance calculated from ground parameters; and

γ_{Rd} is a model factor. The value of γ_{Rd} should conform to the UK National Annex to BS EN 1997-1:2004+A1:2013.

6.4.1.2.1.7 The value of q_b should include the contribution due to the total overburden pressure at the level of the pile base.

6.4.1.2.1.8 If the self-weight of the pile is omitted from the calculation of the action, as allowed by BS EN 1997-1, **7.6.2.1(2)**, then the overburden pressure should be omitted from the calculation of $R_{b,k}$, so that:

$$R_{b,k} = \frac{A_b \times (q_b - \sigma_{v,b})}{\gamma_{Rd}} \quad (34)$$

where:

$\sigma_{v,b}$ is total overburden pressure at the pile base.

6.4.1.2.1.9 The value of the model factor γ_{Rd} should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013.

NOTE The value of the model factor γ_{Rd} given in the UK National Annex to BS EN 1997-1:2004+A1:2013 varies with the amount of static pile load testing that is available to corroborate the calculation of bearing resistance. Background information about the purpose of the model factor can be found in Decoding Eurocode 7 *[47] and* Pile design to Eurocode 7 and the National Annex: Part 2 *[48].*

6.4.1.2.2 Coarse soils

6.4.1.2.2.1 In coarse soils, the ultimate unit shaft resistance in layer *j* ($q_{s,j}$) may be calculated from effective stress parameters, as follows:

$$q_{s,j} = K_{s,j} \times \tan\delta_j \times \delta'_{v,j} \quad (35)$$

where:

$K_{s,j}$ is an earth pressure coefficient (for layer *j*) against the pile shaft;

δ_j is the angle of interface (also known as "wall") friction between the pile and layer *j*; and

$\sigma'_{v,j}$ is the average vertical effective stress acting in the soil in layer *j*.

6.4.1.2.2.2 In the absence of reliable test data, values of K_s may be taken from Table 8. Alternative values of K_s may be used, provided there is previously documented evidence of the successful performance of the same type of pile in similar ground conditions using these alternative values.

Table 8 **Suggested values of K_s for piles installed in coarse silica soils**

Pile type		**Soil type**	**Typical coefficient, K_s A), B)**
Large displacement	Precast concrete (solid) Closed-ended tubular steel Timber Driven cast-in-place concrete	(all)	1.0–1.2
Small displacement	H-section steel bearing piles Open-ended tubular steel Helical steel	(all)	80% of large displacement value

Table 8 **Suggested values of K_s for piles installed in coarse silica soils**

Replacement C)	Continuous flight auger (CFA)	Clean medium-coarse sand Fine sand Silty sand Interlayered silt and sand	0.9 0.7–0.8 0.6–0.7 0.5–0.6
	Bored cast-in-place concrete Micro piles D)		0.7

A) K_s values may vary due to details of specific installation methods, soil layering, groundwater pressures, and elapsed time between installation and testing.
B) K_s values may be superseded by local static pile test data, provided comprehensive documentation is provided (i.e. factual test data, interpretation, local ground conditions, specific pile installation details, etc.).
C) Values taken from the ICE manual of geotechnical engineering (2012), Volume II, Chapter 54 *[1]*.
D) Higher values of K_s may be used for micropiles grouted under pressure.

6.4.1.2.2.3 Values of δ may be estimated from:

$$\delta = min\begin{cases} k_\delta \times \varphi'_{pk} \\ \varphi'_{cv} \end{cases} \quad (36)$$

where:

φ'_{pk} is the soil's peak angle of shearing resistance;

φ'_{cv} is the soil's constant-volume angle of shearing resistance determined in accordance with **4.6.2.4**; and

k_δ is a dimensionless coefficient.

6.4.1.2.2.4 In the absence of reliable test data, values of k_δ may be taken from Table 9.

Table 9 **Suggested values of k_δ for piles installed in coarse soils**

Pile type		**Coefficient k_δ**
Large displacement	Precast concrete (solid) Closed-ended tubular steel	0.67
	Driven cast-in-place concrete	0.9
	Timber	0.85
Small displacement	Steel bearing piles of H-section Open-ended tubular steel	0.67
	Helical steel piles	0.67 A) or 1.0 B)
Replacement	Continuous flight auger (CFA) Bored cast-in-place concrete Micro piles	1.0

A) Value along periphery of steel shaft (soil-to-steel boundary).
B) Value along periphery of helices (soil-to-soil boundary).

6.4.1.2.2.5 Depending on the pile installation method, the presence of fine soils overlying coarse soils can adversely affect the angle of interface friction in those underlying coarse soils. The value of δ should be selected appropriately when this is the case.

6.4.1.2.2.6 In coarse soils, the ultimate effective unit base resistance (q'_b) may be calculated from effective stress parameters, as follows:

$$q'_b = N_q \times \sigma'_{v,b} \quad (37)$$

where:

$\sigma'_{v,b}$ is the vertical effective stress at the pile base; and

N_q is a bearing pressure coefficient that depends on the soil's constant-volume angle of shearing resistance, φ_{cv}; the soil's density index, I_D; and the vertical effective stress at the pile base, $\sigma'_{v,b}$.

NOTE The value of N_q can be obtained from a wide range of theories, including those given in Load bearing capacity and deformation of piled foundations *[49],* Piling Engineering (3rd edition) *[50], and* The Engineering of Foundations *[51].*

6.4.1.2.2.7 The density index, I_D, (which is defined in BS EN ISO 14688-2) may be estimated from the results of field tests (e.g. Standard Penetration Test, Cone Penetration Test) using correlations given in BS EN 1997-2.

6.4.1.2.3 Fine soils

6.4.1.2.3.1 In fine soils, the ultimate unit shaft resistance in layer *j* ($q_{s,j}$) may be calculated from effective stress parameters, as follows:

$$q_{s,j} = \beta_j \times \delta_{v,j} \tag{38}$$

where:

β_j is an empirical coefficient (for layer *j*); and

$\sigma'_{v,j}$ is the average vertical effective stress acting in the soil in layer *j*.

6.4.1.2.3.2 In the absence of reliable test data, values of β for fine soils may be estimated from (see *Shaft friction on piles in clay: a simple fundamental approach* [52] and *Bearing capacity and settlement of pile foundations* [53]):

$$\beta = \begin{cases} (1 - \sin\varphi)\tan\varphi & \text{for normally consolidated clays} \\ 1.5(1 - \sin\varphi)\tan\varphi\sqrt{R_O} & \text{for overconsolidated clays} \end{cases} \tag{39}$$

where:

φ is the soil's angle of shearing resistance; and

R_O is the soil's overconsolidation ratio, given by $R_O = p'_{v,max} / p'_v$;

p'_v is the effective overburden pressure; and

$p'_{v,max}$ is the maximum effective overburden pressure that the soil has previously been subjected to.

6.4.1.2.3.3 Alternatively, the ultimate unit shaft resistance in layer *j* ($q_{s,j}$) may be calculated from total stress parameters, as follows:

$$q_{s,j} = \alpha_j \times c_{u,j} \tag{40}$$

where:

α_j is an empirical coefficient (for layer *j*) that depends on the strength of the soil, the effective overburden pressure acting on it, pile type, and method of execution; and

$c_{u,j}$ is the undrained shear strength of the soil in layer *j*.

NOTE Equation (40) is an empirical relationship between undrained shear strength measured using historical sampling and laboratory test practice (e.g. quick undrained triaxial compression texts on 100 mm diameter samples) and test data from static pile load tests using maintained load.

6.4.1.2.3.4 For piles located in ground that is subject to a reduction in stress (for example, within the zone of influence of deep excavations), equation (40) should only be used if allowance is made for this stress relaxation.

6.4.1.2.3.5 Values of α should be obtained from previous evidence of acceptable performance in static load tests on the same type of pile of similar length and cross-section and in similar ground conditions.

6.4.1.2.3.6 In the absence of reliable test data, values of α may be estimated from one of the methods given in this subclause (**6.4.1.2.3**).

6.4.1.2.3.7 In the absence of reliable test data, values of α for replacement piles (denoted α_{repl}) may be estimated from:

$$0.4 \le \alpha_{repl} = k_1\left(1 - k_2 \log_e \frac{c_u}{p_{ref}}\right) \le 1.0 \qquad (41)$$

where:

c_u is the undrained shear strength of the fine soil;

p_{ref} is 100 kPa; and

k_1 and k_2 are coefficients whose values may be taken as 0.45 and 1.0, respectively.

6.4.1.2.3.8 For replacement piles in glacial tills, the values of k_1 and k_2 in equation (41) may be taken as 0.75 and 0.75, respectively (see *Piling in 'boulder clay' and other glacial tills* [54]).

6.4.1.2.3.9 For bored piles in stiff overconsolidated clays (such as London, Gault, Lias, Oxford and Weald Clays), provided the bore is left open for less than 12 hours, then α_{repl} may be taken as 0.5 (see *Foundations no. 1* [55]).

6.4.1.2.3.10 Alternative values of α_{repl} may be used, provided there is previously documented evidence of the successful performance of the same type of pile in similar ground conditions using these alternative values.

6.4.1.2.3.11 In the absence of reliable test data, values of α for displacement piles (denoted α_{disp}) may be estimated from:

$$\alpha_{disp} = 0.5(c_u/\sigma'_v)^{-m} \qquad (42)$$

where:

c_u is the undrained shear strength of the fine soil;

σ'_v is the effective vertical stress (overburden pressure) acting on the soil; and

m is 0.25 for $c_u/\sigma'_v \ge 1$ and 0.5 for $c_u/\sigma'_v < 1$

6.4.1.2.3.12 Alternative values of α_{disp} may be used, provided there is previously documented evidence of the successful performance of the same type of pile in similar ground conditions using these alternative values.

6.4.1.2.3.13 In fine soils, the ultimate unit base resistance (q_b) may be calculated from total stress parameters, as follows:

$$q_b = N_c \times c_{u,b} \qquad (43)$$

where:

N_c is a bearing pressure coefficient that depends on the depth of the pile base; and

$c_{u,b}$ is the undrained shear strength of the soil at the pile base.

6.4.1.2.3.14 In the absence of reliable test data, the value of N_c may be calculated from:

$$N_c = 9 \times k_1 \times k_2 \tag{44}$$

where:

k_1 is a coefficient that accounts for insufficient embedment of the pile toe; and

k_2 is a coefficient that accounts for the stiffness of the bearing statum.

6.4.1.2.3.15 The value of k_1 in equation (44) should be calculated from:

$$k_1 = \frac{2}{3}\left(1 + \frac{L}{6B}\right) \tag{45}$$

where:

L is the depth of embedment of the pile toe into the bearing stratum; and

B is the pile breadth (or diameter).

6.4.1.2.3.16 Values of k_2 should be taken from Table 10.

Table 10 **Suggested values of k_2 for piles installed in fine soil**

Pile type	**Undrained shear strength of soil, c_u** kPa	**k_2**	**9 × k_2**
Bored, CFA [A)]	≤25	0.72	6.5
	50	0.89	8
	≥100	1.0	9
Driven [B)]		1.11	10

[A)] Values based on *FHWA Report No. NHI-10-016* [56]; k_2 may be interpolated for intermediate values of c_u.

[B)] Value based on *Salgado* [51].

6.4.1.3 Models based on the results of ground tests

6.4.1.3.1 General

6.4.1.3.1.1 The ultimate bearing resistance of a pile foundation may be calculated directly from the results of ground tests on soil and rock (i.e. without first converting those results to ground parameters).

6.4.1.3.1.2 The calculation of ultimate compressive resistance from the results of ground tests should conform to the main method given in BS EN 1997-1:2004+A1:2013, **7.6.2.3**.

6.4.1.3.1.3 If this method is used, the characteristic ultimate compressive resistance of an individual pile ($R_{c,k}$) should be calculated as the smaller of the following two values:

$$R_{c,k} = \min\begin{cases} \left(R_{c,calc}\right)_{mean} / \xi_3 \\ \left(R_{c,calc}\right)_{min} / \xi_4 \end{cases} \tag{46}$$

where:

ξ_3 and ξ_4 are correlation coefficients that depend on the number of tests performed;

$(R_{c,calc})_{mean}$ is the mean calculated ultimate compressive resistance of the pile; and

$(R_{c,calc})_{min}$ is the minimum calculated ultimate compressive resistance of the pile.

6.4.1.3.1.4 The calculated ultimate compressive resistance of an individual pile ($R_{c,calc}$) should be determined from:

$$R_{c,calc} = R_{s,calc} + R_{b,calc} \tag{47}$$

where:

$R_{s,calc}$ is the calculated ultimate shaft resistance; and

$R_{b,calc}$ is the calculated ultimate base resistance.

6.4.1.3.1.5 The calculated ultimate shaft resistance ($R_{s,calc}$) may be determined from:

$$R_{s,calc} = \sum_{i=1}^{n} \left(A_{s,i} \times p_{s,i}\right) \tag{48}$$

where:

$A_{s,i}$ is the total circumferential area of the pile shaft (in layer *i*);

$p_{s,i}$ is the ultimate unit shaft resistance (in layer *i*) obtained from a field test; and

n is the total number of layers in contact with the pile shaft.

6.4.1.3.1.6 The calculated ultimate base resistance ($R_{b,calc}$) may be determined from:

$$R_{b,calc} = A_b \times p_b \tag{49}$$

where:

A_b is the total cross-sectional area of the pile base; and

p_b is the ultimate unit base resistance obtained from a field test.

6.4.1.3.1.7 The values of the correlation coefficients ξ_3 and ξ_4 should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013.

6.4.1.3.2 Cone penetration tests (CPTs)

6.4.1.3.2.1 The ultimate unit shaft resistance in layer *j* ($p_{s,j}$) may be calculated from:

$$p_{s,j} = c_{s,j} \times q_{c,j} \tag{50}$$

where:

$c_{s,j}$ is an empirical coefficient (for layer *j*) that depends on soil and pile type; and

$q_{c,j}$ is the measured cone resistance in layer *j*.

6.4.1.3.2.2 The ultimate unit base resistance, at a settlement equal to 10% of pile diameter, ($p_{b,0.1}$) may be calculated from:

$$p_{b,0.1} = c_{b0.1} \times q_{c,b} \tag{51}$$

where:

$c_{b,0.1}$ is an empirical coefficient that depends on soil and pile type; and

$q_{c,b}$ is the average cone resistance measured over a distance ±1.5 pile diameters below the pile base.

6.4.1.3.2.3 In the absence of reliable test data, values of c_s and $c_{b,0.1}$ may be estimated from Table 11.

NOTE Further information about $c_{b,0.1}$ can be found in Comparing CPT and pile base resistance in sand *[57].*

Table 11 Values of the empirical coefficients c_s and $c_{b,0.1}$ according to soil and pile type

Soil type		c_s			$c_{b,0.1}$		
		Displacement piles		Replacement piles	Displacement piles		Replacement piles
		High displacement	Low displacement		High displacement	Low displacement	
Sand		0.0004–0.009 [A), B), C)]	0.0015–0.004 [A), B), C)]	0.003–0.006 [D), E)]	0.3–0.5 [F), G)]	0.15–0.25 [F), G)]	0.15–0.25 [H)]
Silt		0.006–0.01 [B), C)]		0.003–0.006 [D)]	Data not available		
Clay	Medium to high strength/ over-consolidated	0.007–0.017 [B), C)]		0.008–0.012 [D)]	0.8–1.3 [I)]	0.4–0.65 [I)]	0.34–0.66 [D)]
	Low strength/normally consolidated to lightly over-consolidated	Data not available			0.9–1.0		0.9–1.0

[A)] See *Estimation of load capacity of pipe piles in sand based on CPT results* [58].
[B)] See *An approximate method to estimate the bearing capacity of piles* [59].
[C)] See *Fundações para o silo vertical de 100000 t no Porto de Paranaguá* [60].
[D)] See *On the prediction of the bearing capacity of bored piles from dynamic penetration tests* [61].
[E)] See *Pile capacity by direct CPT and CPTu methods applied to 102 case histories* [62].
[F)] See *Evaluation of a minimum base resistance for driven pipe piles in siliceous sand* [63].
[G)] See *Investigations into the behaviour of displacement piles for offshore structures* [64].
[H)] See *Determination of pile base resistance in sands* [65].
[I)] See *ICP design methods for driven piles in sands and clays* [66].

6.4.1.3.3 Standard Penetration Tests (SPT)

6.4.1.3.3.1 The ultimate unit shaft resistance in layer j ($p_{s,j}$) may be calculated from:

$$p_{s,j} = n_{s,j} \times p_{ref} \times N_j \qquad (52)$$

where:

- $n_{s,j}$ is an empirical coefficient (for layer j) that depends on soil and pile type;
- p_{ref} is 100 kPa; and
- N_j is the measured (uncorrected) SPT blow count in layer j.

6.4.1.3.3.2 The ultimate unit base resistance, at a settlement equal to 10% of pile diameter, ($p_{b,0.1}$) may be calculated from:

$$p_{b0.1} = n_{b0.1} \times p_{ref} \times N_b \qquad (53)$$

where:

- $n_{b,0.1}$ is an empirical coefficient that depends on soil and pile type;
- p_{ref} is 100 kPa; and
- N_b is the measured (uncorrected) SPT blow count at the level of the pile base.

6.4.1.3.3.3 In the absence of reliable test data, values of n_s and $n_{b,0.1}$ may be estimated from Table 12.

Table 12 **Values of the empirical coefficients n_s and $n_{b,0.1}$ according to soil and pile type**

Soil type	n_s		$n_{b,0.01}$	
	Displacement	**Replacement**	**Displacement**	**Replacement**
Sand	0.033–0.043 [A)B)]	0.014–0.026 [A)B)C)]	2.9–4.8 [A)]	0.72–0.82 [C)]
Silt	0.018–0.03 [A)B)]	0.016–.023 [C)]	1.1–2.6 [A)B)]	0.41–0.66 [C)]
Clay	0.020–0.029 [A)B)]	0.024–0.031 [C)]	0.95–1.6 [A)B)]	0.34–0.66 [C)]

A) See *An approximate method to estimate the bearing capacity of piles* [59].
B) See *Fundações para o silo vertical de 100000 t no Porto de Paranaguá* [60].
C) See *On the prediction of the bearing capacity of bored piles from dynamic penetration tests* [61].

6.4.1.3.3.4 Correlations with SPT blow count should be treated with caution, since they are inevitably approximate and not universally applicable.

6.4.1.4 Models based on static pile load tests

*COMMENTARY ON **6.4.1.4***

In the UK, static pile load tests are mainly used to verify resistance calculated using estimated soil parameters.

Static pile load tests are ill suited for designing piles in variable ground conditions, where it is impossible to determine the resistance provided by different strata.

Pile designs based on the results of dynamic impact tests alone can be unreliable when downdrag occurs.

6.4.1.4.1 The ultimate bearing resistance of a pile foundation may be calculated directly from the results of static pile load tests.

6.4.1.4.2 The calculation of ultimate compressive resistance from static pile load tests should conform to BS EN 1997-1:2004+A1:2013, **7.6.2.2**.

6.4.1.4.3 If this method is used, the characteristic ultimate compressive resistance of an individual pile ($R_{c,k}$) should be calculated as the smaller of the following two values:

$$R_{c,k} = \min \begin{cases} (R_{c,m})_{mean}/\xi_1 \\ (R_{c,m})_{min}/\xi_2 \end{cases} \tag{54}$$

where:

ξ_1 and ξ_2	are correlation factors that depend on the number of tests performed;
$(R_{c,m})_{mean}$	is the mean measured ultimate compressive resistance of the pile; and
$(R_{c,m})_{min}$	is the minimum measured ultimate compressive resistance of the pile.

6.4.1.4.4 The values of the correlation factors ξ_1 and ξ_2 should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013, Table A.NA.9.

6.4.1.5 Models based on dynamic impact tests

COMMENTARY ON ***6.4.1.5***

According to BS EN 1997-1:2004+A1:2013, ***7.6.2.4(1)****P, the ultimate bearing resistance of a pile foundation may be calculated from dynamic impact tests provided the validity of the results "have been demonstrated by previous evidence of acceptable performance in static load tests on the same type of pile of similar length and cross-section and in similar ground conditions".*

In the UK, dynamic impact tests are mainly used to verify resistance calculated using estimated soil parameters.

Dynamic impact tests are ill suited for designing piles in variable ground conditions, where it is impossible to determine the resistance provided by different strata.

Pile designs based on the results of dynamic impact tests alone can be unreliable when downdrag occurs.

6.4.1.5.1 The calculation of ultimate compressive resistance from dynamic impact tests should conform to BS EN 1997-1:2004+A1:2013, **7.6.2.4**.

6.4.1.5.2 If this method is used, the characteristic ultimate compressive resistance of an individual pile ($R_{c,k}$) should be calculated as the smaller of the following two values:

$$R_{c,k} = \min \begin{cases} (R_{c,m})_{mean}/\xi_5 \\ (R_{c,m})_{min}/\xi_6 \end{cases} \tag{55}$$

where:

ξ_5 and ξ_6	are the correlation factors that depend on the number of tests performed;
$(R_{c,m})_{mean}$	is the mean measured ultimate compressive resistance of the pile; and
$(R_{c,m})_{min}$	is the minimum measured ultimate compressive resistance of the pile.

6.4.1.5.3 The values of the correlation factors ξ_5 and ξ_6 should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013, Table A.NA.11.

6.4.1.6 Models based on pile driving formulae

*COMMENTARY ON **6.4.1.6***

*According to BS EN 1997-1:2004+A1:2013, **7.6.2.5(2)**P, the ultimate bearing resistance of a pile foundation may be calculated from dynamic pile driving formulae provided the validity of the formulae has been demonstrated "by previous experimental evidence of acceptable performance in static load tests on the same type of pile, of similar length and cross-section, and in similar ground conditions".*

6.4.1.6.1 The ultimate bearing resistance of a pile foundation may be calculated from pile driving formulae. However, preference should be given to another method of calculating the resistance.

6.4.1.6.2 The calculation of ultimate compressive resistance from pile driving formulae should conform to BS EN 1997-1:2004+A1:2013, **7.6.2.5**.

6.4.1.6.3 If this method is used, the characteristic ultimate compressive resistance of an individual pile ($R_{c,k}$) should be calculated from equation (55).

6.4.1.6.4 If a pile exhibits reduced resistance when redriven, and the resistance does not increase again significantly, then care should then be exercised in applying pile driving formulae and preference given to designing on the basis of **6.4.2.1** or **6.4.2.2**.

6.4.1.6.5 Pile driving formulae should be used in combination with other verification methods, such as those given in **6.4.2.1**, **6.4.2.2**, and **6.4.2.3**.

6.4.1.7 Models based on wave equation analysis

*COMMENTARY ON **6.4.1.7***

*According to BS EN 1997-1:2004+A1:2013, **7.6.2.6(2)**P, the ultimate bearing resistance of a pile foundation may be calculated from wave equation analysis provided the validity of the analysis has been demonstrated "by previous evidence of acceptable performance in static load tests on the same pile type, of similar length and cross-section, and in similar ground conditions".*

6.4.1.7.1 The ultimate bearing resistance of a pile foundation may be calculated from wave equation analysis. However, preference should be given to another method of calculating the resistance.

6.4.1.7.2 The calculation of ultimate total resistance from wave equation analysis should conform to BS EN 1997-1:2004+A1:2013, **7.6.2.6**.

6.4.1.7.3 If this method is used, the characteristic ultimate compressive resistance of an individual pile ($R_{c,k}$) should be calculated from equation (55).

6.4.1.7.4 Wave equation analysis should be used in combination with other verification methods, such as those given in **6.4.2.1**, **6.4.2.2**, and **6.4.2.3**.

6.4.1.8 Modifications for downdrag (also known as "negative skin friction")

*COMMENTARY ON **6.4.1.8***

The term "downdrag" is used in this standard to refer to the phenomenon whereby the ground surrounding a pile settles a significant amount relative to the pile head. Downdrag is particularly relevant to piles installed in low strength clay or in coarse soils subject to upfilling or a lowering of the groundwater table.

The additional axial force in the pile due to downdrag is termed the "drag force" and the additional settlement of the pile is called the "drag settlement" (after A practical design approach for piles with negative skin friction *[67]).*

It is a common misconception that downdrag reduces the ultimate bearing resistance of a pile. In many situations, the settlement of the pile at the ultimate limit state is sufficient to cancel the effects of downdrag, resulting in no reduction in ultimate bearing resistance.

Although downdrag might have little effect on a pile's ultimate bearing resistance, the drag force does influence the structural design of the pile and the drag settlement influences its serviceability.

Downdrag causes shaft friction over the upper part of a pile to act as an additional force applied to the pile, instead of as a resistance. Because the direction (or "sign") of this shaft friction is reversed from the norm, it is commonly termed "negative skin friction".

The depth at which there is no relative movement between the pile and the surrounding ground is known as the "neutral plane". The neutral plane occurs where the ground settlement (s_g) at a particular depth equals the pile settlement (s_p) at the same depth, as shown in Figure 4.

The depth of the neutral plane can be over-predicted by linear elastic settlement analyses for friction piles in strata whose stiffness increases gradually with depth.

6.4.1.8.1 The characteristic compressive force ($F_{c,k}$) acting on a pile that is subject to downdrag should be calculated from:

$$F_{c,k} = P_{c,k} + W_k + P_{dd,k} \tag{56}$$

where:

$P_{c,k}$ is the characteristic compressive force applied to the pile by the structure;

W_k is the characteristic self-weight of the pile; and

$P_{dd,k}$ is the additional characteristic compressive force owing to downdrag, given by:

$$P_{dd,k} = \int_0^{L_{dd}} \left(C_s \times q_{s,k,sup}\right) dz \tag{57}$$

where:

L_{dd} is the length of pile subject to downdrag (defined in Figure 4);

C_s is the circumference of the pile shaft at depth z; and

$q_{s,k,sup}$ is the "superior" characteristic (defined below) unit shaft friction at depth z.

6.4.1.8.2 The "superior" characteristic unit shaft friction ($q_{s,k,sup}$) should be selected as a cautious upper estimate of the mean shaft friction acting over the length of pile that is subject to downdrag.

NOTE A cautious upper estimate of the mean is one that has a 5% probability of being exceeded during the design working life.

6.4.1.8.3 The characteristic shaft resistance ($R_{s,dd,k}$) of a pile subject to downdrag should be calculated from:

$$R_{s,dd,k} = \frac{\int_{L_{dd}}^{L} \left(C_s \times q_{s,k,inf}\right) dz}{\gamma_{Rd}} \tag{58}$$

where:

L is the total length of the pile;

L_{dd} is the length of pile subject to downdrag (equal to the depth of the neutral plane defined in Figure 6);

C_s is the circumference of the pile shaft at depth z;

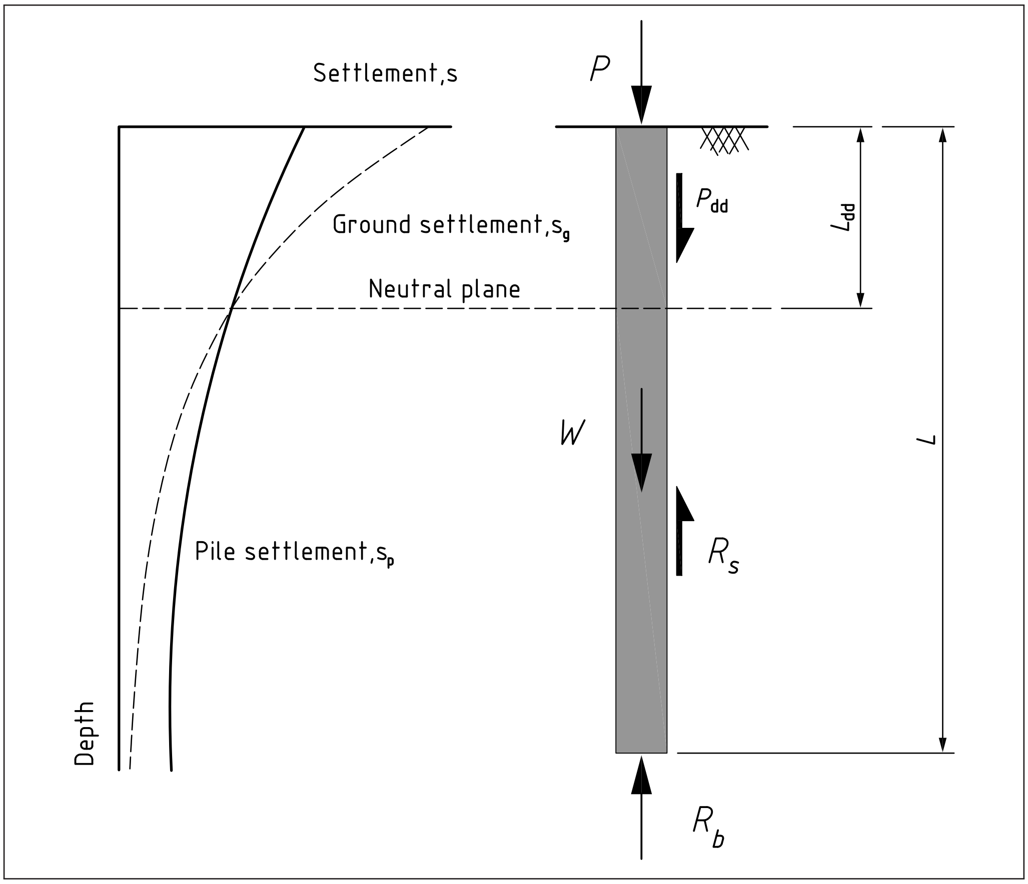
$q_{s,k,inf}$ is the "inferior" characteristic (defined below) unit shaft friction at depth z; and

γ_{Rd} is a model factor.

6.4.1.8.4 The "inferior" characteristic unit shaft friction ($q_{s,k,inf}$) should be selected as a cautious lower estimate of the mean shaft friction acting over the length of pile that is not subject to downdrag.

NOTE A cautious lower estimate of the mean is one that has a 5% probability of not being achieved during the design working life.

Figure 4 **Calculation model for downdrag**



6.4.2 Tensile resistance

*COMMENTARY ON **6.4.2***

BS EN 1997-1 allows the ultimate tensile resistance of an individual pile to be determined from any of the following:

- *static pile formulae based on ground parameters;*
- *direct formulae based on the results of field tests; and*
- *the results of static pile load tests.*

6.4.2.1 Models based on ground parameters

6.4.2.1.1 General

6.4.2.1.1.1 The calculation of ultimate tensile resistance from ground parameters should conform to the "alternative method" given in BS EN 1997-1:2004+A1:2013, **7.6.3.3**.

6.4.2.1.1.2 When this method is used, the characteristic ultimate tensile resistance of an individual pile ($R_{t,k}$) should be calculated as:

$$R_{t,k} = R_{s,k} \tag{59}$$

where:

$R_{s,k}$ is the pile's characteristic ultimate shaft resistance.

6.4.2.1.1.3 The characteristic ultimate shaft resistance ($R_{s,k}$) should conform to **6.4.1.2**.

6.4.2.1.1.4 The value of the model factor γ_{Rd} should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013.

6.4.2.1.2 Rock and rock masses

*COMMENTARY ON **6.4.2.1.2***

The resistance of a pile foundation in rock depends markedly on the method of pile construction and the roughness of the rock socket in which the pile is located.

6.4.2.1.2.1 The equations given in this subclause (**6.4.2.1.2**) should not be used for the verification of the ultimate limit state of piles in rock, unless the parameters used have been corroborated by the results of static pile load tests on similar piles in similar ground conditions.

NOTE The equations given in this subclause can be used for preliminary design without pile testing.

6.4.2.1.2.2 In weak to medium strong rock, the ultimate unit shaft resistance (q_s) may be calculated from (see *Piling Engineering (3rd edition)* [50]):

$$q_s = k_1 p_{ref} \left(\frac{q_u}{p_{ref}} \right)^{k_2} \tag{60}$$

where:

q_u is the rock's unconfined compression strength;

k_1 and k_2 are empirical coefficients that depend on rock and pile type; and

p_{ref} is given in Table 4.

NOTE Guidance on the calculation of unit shaft friction in rocks can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 54 *[1] and in* State of the art report: Analysis and design *[68].*

6.4.2.1.2.3 In the absence of reliable test data, values of k_1 and k_2 may be taken from Table 13.

6.4.2.1.2.4 In weak to medium strong rock, the ultimate unit base resistance (q_b) may be calculated from (see *Piling Engineering (3rd edition)* [50]):

$$q_b = k_3 p_{ref} \left(\frac{q_u}{p_{ref}} \right)^{k_4} \quad (61)$$

where:

q_u	is the rock's unconfined compression strength; and
k_3 and k_4	are empirical coefficients that depend on rock type; and
p_{ref}	is given in Table 4.

NOTE Guidance on the calculation of unit base resistance in rocks can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 54 *[1] and in* State of the art report: Analysis and design *[68].*

6.4.2.1.2.5 In the absence of reliable test data, values of k_3 and k_4 may be taken from Table 13.

6.4.2.1.2.6 The values of k_1 to k_4 should be selected very cautiously, unless there is static pile load test data to corroborate less cautious values.

6.4.2.1.2.7 The ultimate unit shaft and base resistances of piles in chalk should be determined in accordance with CIRIA Report C574, Chapter 8 [N2].

NOTE 1 Guidance on piled foundations in weak rocks can be found in CIRIA Research Report 181 *[69].*

NOTE 2 Guidance on rock-socketed shafts for highway structure foundations can be found in NCHRP Synthesis 360 *[70].*

Table 13 **Suggested values of the coefficients k_1 to k_4 for piles installed in rocks**

Rock type	Coefficient k_1	Coefficient k_2	Coefficient k_3	Coefficient k_4
(Generic)	0.63–1.26 A)	0.5 A)	—	—
Soft rock	1.0–1.29 B)	0.57–0.61 B)	—	—
Cemented materials	0.7–2.1 C)	0.5 C)	15 C)	0.5 C)

A) Values from *Results of tests to determine shaft resistance of rock socketed drilled piers* [71].
B) Values from *A design method for drilled piers in soft rock* [72].
C) Values from *Drilled shaft side resistance in clay soil to rock* [73].

6.4.2.2 Models based on the results of ground tests

6.4.2.2.1 The calculation of ultimate tensile resistance from the results of field tests should conform to the main method given in BS EN 1997-1:2004+A1:2013, **7.6.3.3**.

6.4.2.2.2 When this method is used, the characteristic ultimate tensile resistance of an individual pile ($R_{t,k}$) should be calculated as the smaller of the following two values:

$$R_{t,k} = \min \begin{cases} (R_{t,calc})_{mean} / \xi_3 \\ (R_{t,calc})_{min} / \xi_4 \end{cases} \quad (62)$$

where:

ξ_3 and ξ_4	are correlation factors that depend on the number of tests performed;

$(R_{t,calc})_{mean}$ is the mean calculated ultimate tensile resistance of the pile; and

$(R_{t,calc})_{min}$ is the minimum calculated ultimate tensile resistance of the pile.

6.4.2.2.3 The calculated ultimate tensile resistance of an individual pile ($R_{t,calc}$) should be determined from:

$$R_{t,calc} = R_{s,calc} \tag{63}$$

where:

$R_{s,calc}$ is the calculated ultimate shaft resistance.

6.4.2.2.4 The calculated ultimate shaft resistance ($R_{s,calc}$) should conform to **6.4.2.2**.

6.4.2.2.5 The values of the correlation factors ξ_3 and ξ_4 should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013, Table A.NA.10.

6.4.2.3 Models based on static pile load tests

6.4.2.3.1 The ultimate tensile resistance of a pile foundation may be calculated directly from the results of static pile load tests.

6.4.2.3.2 The calculation of ultimate tensile resistance from static pile load tests should conform to BS EN 1997-1:2004+A1:2013, **7.6.3.2**.

6.4.2.3.3 If this method is used, the characteristic ultimate tensile resistance of an individual pile ($R_{t,k}$) should be calculated as the smaller of the following two values:

$$R_{t,k} = \min\begin{cases}(R_{t,m})_{mean}/\xi_1 \\ (R_{t,m})_{min}/\xi_2\end{cases} \tag{64}$$

where:

ξ_1 and ξ_2 are correlation factors that depend on the number of tests performed;

$(R_{t,m})_{mean}$ is the mean measured ultimate tensile resistance of the pile; and

$(R_{t,m})_{min}$ is the minimum measured ultimate tensile resistance of the pile.

6.4.2.3.4 The values of the correlation factors ξ_1 and ξ_2 should be taken from the UK National Annex to BS EN 1997-1:2004+A1:2013, Table A.NA.10.

6.4.3 Transverse resistance

6.4.3.1 Models based on ground parameters

6.4.3.1.1 General

*COMMENTARY ON **6.4.3.1.1***

BS EN 1997-1 recommends that the following failure mechanisms should be considered when verifying the transverse resistance of an individual pile:

- *for short piles, rotation or translation as a rigid body (termed the "short pile mechanism"); and*
- *for long slender piles, bending failure of the pile, accompanied by local yielding and displacement of the soil near the top of the pile (termed the "long pile mechanism").*

6.4.3.1.1.1 The calculation of ultimate transverse resistance from ground parameters should conform to BS EN 1997-1:2004+A1:2013, **7.7.3**.

6.4.3.1.1.2 The characteristic ultimate transverse resistance of an individual pile ($R_{tr,k}$) should be calculated from:

$$R_{tr,k} = \min\left(R_{tr,short,k}, R_{tr,long,k}\right) \quad (65)$$

where:

$R_{tr,short,k}$ is the characteristic ultimate transverse resistance of an individual pile rotating or translating as a rigid body (the "short pile mechanism"); and

$R_{tr,long,k}$ is the characteristic ultimate transverse resistance of an individual pile that fails in bending near its head (the "long pile mechanism").

6.4.3.1.2 Coarse soils

6.4.3.1.2.1 In coarse soils, the characteristic ultimate unit transverse resistance of a pile ($R_{tr,k}$) may be calculated using Broms' method [74] from an expression of the form:

$$R_{tr,k} = func\left\{\varphi_k, \gamma_k, B, L, e, M_{Rk}\right\} \quad (66)$$

where:

func{…} denotes a function of the enclosed variables;

φ_k is the soil's characteristic angle of shearing resistance;

γ_k is the soil's characteristic weight density;

B is the pile's breadth;

L is the pile's embedded length;

e is the eccentricity of the transverse load applied to the pile; and

M_{Rk} is the pile's characteristic ultimate bending resistance.

NOTE Guidance on the precise form of the function to be used in equation (66) can be found in textbooks, such as The engineering of foundations *[51]* and Piles and pile foundations *[75].*

6.4.3.1.2.2 Alternatively, the characteristic ultimate unit transverse resistance of a pile in coarse soils may be calculated using Brinch Hansen's closed-form solution (see *The ultimate resistance of rigid piles against transversal forces* [76]).

NOTE Guidance on the use of Brinch Hansen's method can be found in textbooks, such as Pile design and construction *[77].*

6.4.3.1.3 Fine soils

6.4.3.1.3.1 In fine soils, the characteristic ultimate unit transverse resistance of a pile ($R_{tr,k}$) may be calculated using Broms' method [78] from an expression of the form:

$$R_{tr,k} = func\left\{c_{u,k}, B, L, e, M_{Rk}\right\} \quad (67)$$

where:

$c_{u,k}$ is the soil's characteristic undrained shear strength; and

the other symbols are as defined for equation (66).

NOTE Guidance on the precise form of the function to be used in equation (67) can be found in standard textbooks, such as The engineering of foundations *[51]* and Piles and pile foundations *[75].*

6.4.3.1.3.2 Alternatively, the characteristic ultimate unit transverse resistance of a pile in fine soils may be calculated using Brinch Hansen's closed-form solution (see *The ultimate resistance of rigid piles against transversal forces* [76]).

NOTE Guidance on the use of Brinch Hansen's method can be found in textbooks, such as Pile design and construction *[77].*

6.4.3.2 Models based on the results of field tests

The calculation of ultimate transverse resistance from the results of field tests should conform to BS EN 1997-1:2004+A1:2013, **7.7.3**.

6.4.3.3 Models based on static pile load tests

6.4.3.3.1 The ultimate transverse resistance of a pile foundation may be calculated directly from the results of static pile load tests.

6.4.3.3.2 The calculation of ultimate transverse resistance from static pile load tests should conform to BS EN 1997-1:2004+A1:2013, **7.7.2**.

6.4.4 Settlement

6.4.4.1 The settlement of a pile foundation may be calculated using any of the following models, as appropriate:

- theory of elasticity (see *Pile foundation analysis and design* [79]);
- t-z curves (see *Analysis and design of shallow and deep foundations* [80]);
- hyperbolic stress-strain model (see *A new method for single pile settlement prediction and analysis* [81]);
- numerical models, including:
    - the interaction-factor method (see *An analysis of the vertical deformation of pile groups* [82]);
    - the boundary element method (see *Non-linear analysis of pile groups* [83]);
    - finite element method;
- wave equation analysis, provided the validity of the analysis has been demonstrated by previous evidence of acceptable performance in static load tests on the same pile type, of similar length and cross-section, and in similar ground conditions; or
- other appropriate models not listed here.

6.4.4.2 In order to conform to BS EN 1997-1:2004+A1:2013, **7.4.1**, the validity of the pile settlement model that is used for design should be demonstrated by static pile load tests in comparable situations.

NOTE 1 Guidance on the calculation of settlement for single piles can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 54 *[1].*

NOTE 2 Guidance on the calculation of settlement of pile groups can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 55 *[1].*

6.4.5 Lateral displacement

6.4.5.1 The lateral displacement of a pile foundation may be calculated using any of the following models, as appropriate:

- theory of elasticity (see *Pile foundation analysis and design* [79]);
- p-y curves (see *Single piles and pile groups under lateral loading* [84]);
- subgrade reaction models;

- numerical models, including:
  - the interaction-factor method (see *An analysis of the vertical deformation of pile groups* [82]);
  - the boundary element method (see *Non-linear analysis of pile groups* [83]);
  - the finite element method; or
- other appropriate models not listed here.

6.4.5.2 Owing to rapid degradation of mobilized stiffness with pile head movement when using linear elastic and subgrade reaction models, preference should be given to an alternative method of calculating lateral displacement of a pile foundation.

6.4.5.3 Local near surface ground conditions can have a significant influence on an individual pile's response to lateral loading.

6.4.5.4 Pile head fixity can have a significant influence on a pile group's response to lateral loading.

6.5 Materials

6.5.1 Concrete

6.5.1.1 The materials and products incorporated into concrete piles should conform to **4.3.6**.

6.5.1.2 Materials and products incorporated into bored cast-in-place concrete piles should also conform to BS EN 1536 and BS EN 206:2013, Annex D.

6.5.1.3 Materials and products incorporated into driven cast-in-place concrete piles should also conform to BS EN 12699 and BS EN 206:2013, Annex D.

6.5.1.4 Materials and products incorporated into prefabricated concrete piles should also conform to BS EN 12699 and BS EN 12794.

6.5.1.5 Materials and products incorporated into micropiles should also conform to BS EN 14199 and BS EN 206:2013, Annex D.

NOTE 1 BS EN 206:2013, Annex D, includes the normative rules for concrete for special geotechnical work that were previously given in BS EN 1536:2010, BS EN 1538:2010, BS EN 12699:2001, and BS EN 14199:2005.

NOTE 2 At the time of publication, amendments to BS EN 1536:2010 and BS EN 1538:2010 are in preparation to remove the rules that are now contained in BS EN 206:2013, Annex D.

NOTE 3 Guidance on the use of low-strength concrete mixes for use in firm female secant piles can be found in BRE Information Paper IP 17/05 *[85].*

6.5.2 Steel

6.5.2.1 The materials used to fabricate steel piles should conform to **4.3.7**.

6.5.2.2 Materials used to fabricate H-section and tubular steel piles should also conform to BS EN 12699.

6.5.2.3 Hot finished tubular steel piles should also conform to BS EN 10210.

6.5.2.4 Cold formed tubular steel piles should also conform to BS EN 10219.

6.5.2.5 Hot rolled steel sheet piles used as bearing piles should also conform to BS EN 10248.

6.5.2.6 Cold formed steel sheet piles used as bearing piles should also conform to BS EN 10249.

6.5.2.7 Steel reinforcement used in the fabrication of concrete piles should conform to BS EN 1992-1-1 and BS EN 10080.

6.5.3 Timber

*COMMENTARY ON **6.5.3***

Certain types of softwood and hardwood are suitable for use as permanent piles. The choice depends upon availability in suitable sizes, the expected useful life, and the relative cost, including preservative treatment. Availability in the required lengths is frequently a limiting factor.

Commonly used softwoods include:

- *Douglas fir, imported to the UK in sections up to 400 mm square and 15 m long (or longer to special order);*
- *Pitch pine, imported to the UK in sections up to 500 mm square and 15 m long.*

Commonly used hardwoods include:

- *Greenheart (used for permanent works), imported to the UK rough-hewn in sections up to 475 mm square and up to 18 m long (or larger sections and lengths up to 24 m long to special order);*
- *Tropical hardwoods, imported in sections up to 9 m long.*

Straightness of grain is important in timber piles, particularly where hard driving is anticipated.

6.5.3.1 The materials used to fabricate timber piles should conform to **4.3.8** and BS EN 12699.

6.5.3.2 Timber piles should be free from any defects that could affect their strength or durability.

6.5.3.3 Piles should be ordered long enough to ensure that, after cut-off at the proper level, the top of the pile is clean, sound, and undamaged after driving.

6.5.3.4 When straight trunks of coniferous trees are used for timber piles, the bark should be removed and the timber pressure impregnated with preservative.

NOTE Strength classes for structural timber are defined in BS EN 338.

6.5.4 Cement grout

Cement grout incorporated into micropiles should conform to BS EN 14199.

6.6 Durability

6.6.1 Concrete

The durability of concrete piles should conform to **4.4.2**.

6.6.2 Steel

6.6.2.1 The durability of steel bearing piles should conform to **4.4.3**.

6.6.2.2 The durability of steel bearing piles should also conform to BS EN 1993-5.

NOTE 1 NA+A1:2012 to BS EN 1993-5:2007 gives UK values for loss of thickness per face due to corrosion of steel bearing piles in soils, with or without groundwater.

NOTE 2 A method of assessing the corrosivity of the ground surround buried steel pile can be found in Table 8.1 of the Design Manual for Roads and Bridges, Volume 2, Section 2, Part 6, BD 12/01 *[86].*

6.6.3 Timber

6.6.3.1 The durability of timber piles should conform to **4.4.4**.

6.6.3.2 Measures taken to prolong the life of timber piles should conform to BS EN 1995-1-1.

NOTE Timber piles may be designed with no protection against decay if they are kept below a moisture content of 22% or they are buried in the ground below the lowest permanent water table.

6.6.4 Cement grout

The durability of cement grout incorporated into micropiles should conform to BS EN 14199:2015, **7.6**.

6.7 Ultimate limit state design

6.7.1 General

The ultimate limit state design of a pile foundation should conform to **4.6** and this subclause (**6.7**).

6.7.2 Bearing

6.7.2.1 Individual piles

*COMMENTARY ON **6.7.2.1***

BS EN 1997-1 provides several alternative methods for verifying the ultimate compressive resistance of an individual pile, including methods based on:

- *static pile load tests;*
- *ground test results;*
- *dynamic impact tests;*
- *pile driving formulae; and*
- *wave equation analysis.*

*Traditional UK practice has been to verify the compressive resistance of an individual pile using ground test results in calculations based on soil mechanics theory. This approach is termed the "alternative procedure" in BS EN 1997-1:2004+A1:2013, **7.6.2.3**.*

6.7.2.1.1 The design value of the ultimate compressive resistance of an individual pile ($R_{c,d}$) should be verified according to the alternative procedure given in BS EN 1997-1:2004+A1:2013, **7.6.2.3**, and conform to expression (7.5) of that standard, namely:

$$R_{c,d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,k}}{\gamma_s} \qquad (68)$$

where:

$R_{b,k}$ is the characteristic value of the pile's (calculated) ultimate base resistance;

$R_{s,k}$ is the characteristic value of the pile's (calculated) ultimate shaft resistance; and

γ_b and γ_s are the partial factors given in the UK National Annex to BS EN 1997-1:2004+A1:2013, whose values depend on the level of pile testing that is performed to corroborate the calculations of resistance.

6.7.2.1.2 In the United Kingdom, the ultimate geotechnical compressive resistance of an individual pile may be verified using Design Approach 1 Combination 2 alone, since the values of the partial factors γ_b and γ_s given in the UK National Annex to BS EN 1997-1:2004+A1:2013 are such that Design Approach 1 Combination 1 cannot govern the design.

6.7.2.1.3 The design compressive force ($F_{c,d}$) applied to an individual pile at its ultimate limit state should be calculated from:

$$F_{c,d} = \sum_i \left(\gamma_{F,i}\psi_i P_{c,k,i}\right) + \gamma_G\left(W_k + P_{dd,k}\right) \tag{69}$$

where:

- $P_{c,k,i}$ is the *i*th characteristic compressive force applied to the pile by the structure;
- W_k is the characteristic self-weight of the pile;
- $P_{dd,k}$ is the additional characteristic compressive force owing to downdrag, given by equation (57);
- ψ_i is the corresponding combination factor for the *i*th force;
- $\gamma_{F,i}$ is the corresponding partial factor on actions for the *i*th force; and
- γ_G is the partial factor on permanent actions.

6.7.2.1.4 The additional characteristic compressive force owing to downdrag ($P_{dd,k}$) should only be included in equation (69) when the pile head displacement at the geotechnical ultimate limit state is less than the anticipated settlement of the ground surface. In that situation, the pile's bearing resistance should also be reduced accordingly, to:

$$R_{c,d} = \frac{R_{b,k}}{\gamma_b} + \frac{R_{s,dd,k}}{\gamma_s} \tag{70}$$

where:

- $R_{s,dd,k}$ is the characteristic shaft resistance of the pile when subject to downdrag, given by equation (58); and

the other symbols are as defined for equation (68).

6.7.2.1.5 The method used to determine the ultimate limit state of a pile may be based on any of the following criteria:

- the ultimate load measured at a settlement equal to 10% of the pile's diameter, as recommended in BS EN 1997-1:2004+A1:2013, **7.6.1.1**(3) when it is difficult to define the ultimate limit state from the load settlement curve;
- Chin's criterion [87], assuming a hyperbolic pile load v settlement curve; or
- Fleming's method [81], assuming separate hyperbolic shaft and base load vs settlement curves and elastic pile shortening;
- Davisson's offset limit [88];
- Butler and Hoy's slope and tangent method [89].

6.7.2.2 Pile groups

*COMMENTARY ON **6.7.2.2***

Provided the pile cap has adequate structural strength to redistribute axial loads across the group, full mobilization of an individual pile's geotechnical resistance can occur within a large pile group (comprising 5 or more piles) without the pile group reaching an ultimate limit state.

6.7.2.2.1 The compressive resistance of a pile group should be verified assuming the individual piles and the ground between them act as a block.

6.7.2.2.2 Where a row of piles is used to form a retaining wall, particular attention should be paid to the possibility of failure of that row when the retaining wall carries significant vertical load.

6.7.2.2.3 The design value of the ultimate compressive resistance of a pile group ($R_{\mathrm{group,c,d}}$) should be calculated from:

$$R_{\mathrm{group,c,d}} = \min\left(R_{\mathrm{sum,c,d}}; R_{\mathrm{block,c,d}}; R_{\mathrm{row,c,d}}\right) \qquad (71)$$

where:

$R_{\mathrm{sum,c,d}}$	is the sum of the ultimate compressive resistances of all the individual piles;
$R_{\mathrm{block,c,d}}$	is the ultimate compressive resistance of a block that encloses the pile group;
$R_{\mathrm{row,c,d}}$	is the ultimate compressive resistance of a row of piles within that block.

6.7.2.2.4 The design value of the ultimate compressive resistance of a large pile group subject to large horizontal loads and/or moments may alternatively be calculated from:

$$R_{\mathrm{group,c,d}} = R_{\mathrm{sum,c,d}} \qquad (72)$$

NOTE For the purposes of this clause, a pile group is considered "large" if it comprises five or more piles and has a cap that can redistribute axial loads across the group.

6.7.2.2.5 The sum of the ultimate compressive resistances of the individual piles ($R_{\mathrm{sum,c,d}}$) should be calculated from:

$$R_{\mathrm{sum,c,d}} = \sum_{j=1}^{j=n} \left(R_{\mathrm{c,d}}\right)_j \qquad (73)$$

where:

$(R_{\mathrm{c,d}})_j$	is the design value of the ultimate compressive resistance of pile j; and
n	is the total number of piles in the group.

6.7.2.2.6 In coarse soils, the design value of the ultimate compressive resistance of the block that encloses the pile group ($R_{\mathrm{block,c,d}}$) may be calculated from:

$$R_{\mathrm{block,c,d}} = \left(K_0 \times \overline{\sigma'_{\mathrm{v}}} \times \tan\varphi \times A_{\mathrm{s}} \times s_{\mathrm{block}}\right) + \left(N_{\mathrm{q}} \times \sigma'_{\mathrm{v,b}} \times A_{\mathrm{b}}\right) \qquad (74)$$

where:

K_0	is the soil's at-rest earth pressure coefficient;
$\overline{\sigma'_{\mathrm{v}}}$	is the average vertical effective stress over the side of the block;
φ	is the soil's angle of shearing resistance;

A_s is the total side area of the block (i.e. perimeter of block × pile length);

s_{block} is a shape factor chosen in accordance with **5.4.1.2**;

N_q is a bearing coefficient chosen in accordance with **5.4.1.2**;

$\sigma'_{v,b}$ is the vertical effective stress at the base of the block; and

A_b is the total base area of the block (i.e. width × breadth of block).

6.7.2.2.7 In fine soils, the design value of the ultimate compressive resistance of the block that encloses the pile group ($R_{block,c,d}$) may be calculated from:

$$R_{block,c,d} = \left(\overline{c_{u,d}} \times A_s \times s_{block}\right) + \left(N_c \times c_{u,b,d} \times A_b\right) \quad (75)$$

where:

$\overline{c_{u,d}}$ is the soil's average design undrained shear strength over the side of the block;

A_s is the total side area of the block (i.e. perimeter of block × pile length);

s_{block} is a shape factor chosen in accordance with **5.4.1.3**;

N_c is a bearing coefficient chosen in accordance with **5.4.1.3**;

$c_{u,b,d}$ is the soil's design undrained shear strength at the base of the block; and

A_b is the total base area of the block (i.e. width × breadth of block).

NOTE 1 The design value of the ultimate compressive resistance of a row of piles within the block ($R_{row,c,d}$) may be calculated in a similar fashion to the block as a whole, adjusting A_s and A_b accordingly.

NOTE 2 Except when the spacing between rows is variable or the action from horizontal forces or moments is large relative to the action from vertical forces, the compressive resistance of a row of piles does not normally govern the design and may therefore be ignored.

6.7.3 Tension/pull-out

6.7.3.1 Individual piles

*COMMENTARY ON **6.7.3.1***

BS EN 1997-1 provides two alternative methods for verifying the ultimate compressive resistance of an individual pile, including methods based on:

- *static pile load tests; and*
- *ground test results.*

*Traditional UK practice has been to verify the tensile resistance of an individual pile using ground test results in calculations based on soil mechanics theory. This approach is termed the "alternative procedure" in BS EN 1997-1:2004+A1:2013, **7.6.3.3**.*

6.7.3.1.1 The design value of the ultimate tensile resistance of an individual pile ($R_{t,d}$) should be verified according to the alternative procedure given in BS EN 1997-1:2004+A1:2013, **7.6.3.3**, and conform to expressions (7.15 and 7.16) of that standard, namely:

$$R_{t,d} = \frac{R_{t,k}}{\gamma_{s,t}} = \frac{R_{s,k}}{\gamma_{s,t}} \tag{76}$$

where:

$R_{t,k}$ is the characteristic value of the pile's (calculated) ultimate tensile resistance;

$R_{s,k}$ is the characteristic value of the pile's (calculated) ultimate shaft resistance; and

$\gamma_{s,t}$ is the partial factor given in the UK National Annex to BS EN 1997-1:2004+A1:2013, whose value depends on the level of pile testing that is performed to corroborate the calculation of resistance.

6.7.3.1.2 In the United Kingdom, the ultimate geotechnical tensile resistance of an individual pile may be verified using Design Approach 1 Combination 2 alone, since the values of the partial factors $\gamma_{s,t}$ given in the UK National Annex to BS EN 1997-1:2004+A1:2013 are such that Design Approach 1 Combination 1 cannot govern the design.

6.7.3.2 Pile groups

6.7.3.2.1 The tensile resistance of a pile group should be verified assuming the individual piles and the ground between them act as a block.

6.7.3.2.2 The design value of the ultimate tensile resistance of a pile group ($R_{group,t,d}$) should be calculated from:

$$R_{group,t,d} = \min\left(R_{sum,t,d}; R_{block,t,d}; R_{row,t,d}\right) \tag{77}$$

where:

$R_{sum,t,d}$ is the sum of the ultimate tensile resistances of all the individual piles;

$R_{block,t,d}$ is the ultimate tensile resistance of a block that encloses the pile group;

$R_{row,t,d}$ is the ultimate tensile resistance of a row of piles within that block.

6.7.3.2.3 The sum of the ultimate tensile resistances of the individual piles ($R_{sum,t,d}$) should be calculated from:

$$R_{sum,t,d} = \sum_{j=1}^{j=n} \left(R_{t,d}\right)_j \tag{78}$$

where:

$(R_{t,d})_j$ is the design value of the ultimate tensile resistance of pile j;

n is the total number of piles in the group.

6.7.3.2.4 In coarse soils, the design value of the ultimate tensile resistance of the block that encloses the pile group ($R_{block,t,d}$) may be calculated from:

$$R_{block,t,d} = K_0 \times \overline{\sigma'_v} \times \tan\varphi \times A_s \times s_{block} \quad (79)$$

where:

- K_0 is the soil's at-rest earth pressure coefficient;
- $\overline{\sigma'_v}$ is the average vertical effective stress over the side of the block;
- φ is the soil's angle of shearing resistance;
- A_s is the total side area of the block (i.e. perimeter of block × pile length); and
- s_{block} is a shape factor chosen in accordance with **5.4.1.2**.

6.7.3.2.5 In fine soils, the design value of the ultimate tensile resistance of the block that encloses the pile group ($R_{block,t,d}$) may be calculated from:

$$R_{block,t,d} = \overline{c_{u,d}} \times A_s \times s_{block} \quad (80)$$

where:

- $\overline{c_{u,d}}$ is the soil's average design undrained shear strength over the side of the block;
- A_s is the total side area of the block (i.e. perimeter of block × pile length); and
- s_{block} is a shape factor chosen in accordance with **5.4.1.3**.

6.7.3.2.6 The design value of the ultimate tensile resistance of a row of piles within the block ($R_{row,t,d}$) may be calculated in a similar fashion to the block as a whole, adjusting A_s accordingly.

NOTE Except when the spacing between rows is variable or the action from horizontal forces or moments is large relative to the action from vertical forces, the tensile resistance of a row of piles may be ignored since it does not normally govern the design.

6.7.4 Transverse resistance

6.7.4.1 Individual piles

*COMMENTARY ON **6.7.4.1***

*Although BS EN 1997-1:2004+A1:2013, **7.7.1(2)**P, requires verification that an individual pile "will support the design transverse load with adequate safety against failure", it gives no details of which partial factors are to be applied or their recommended values.*

6.7.4.1.1 The design value of the ultimate transverse resistance of an individual pile ($R_{tr,d}$) should be calculated from:

$$R_{tr,d} = \begin{cases} func\{\varphi_d, \gamma_d, B, L, e, M_{Rd}\} & \text{in coarse soils} \\ func\{c_{u,d}, B, L, e, M_{Rd}\} & \text{in fine soils} \end{cases} \quad (81)$$

where:

- φ_d is the soil's design angle of shearing resistance;
- γ_d is the soil's design weight density;

$c_{u,d}$ is the soil's design undrained shear strength;

M_{Rd} is the pile's design ultimate bending resistance; and

the other symbols are as defined for equation (66).

6.7.4.1.2 The design values of the soil parameters to be used in equation (81) should conform to BS EN 1997-1:2004+A1:2013, **2.4.6.2**, with the partial factors γ_{cu}, γ_{φ}, and γ_{γ} as specified in the UK National Annex to BS EN 1997-1:2004+A1:2013 for Design Approach 1, Combinations 1 and 2.

6.7.4.1.3 The effect of ground movement of the transverse resistance of an individual pile should be considered.

6.8 Serviceability limit state design

6.8.1 General

6.8.1.1 Serviceability limit states should be verified according to BS EN 1997:2004+A1:2013, **2.4.8(1)**P, by ensuring:

$$E_d \leq C_d \tag{82}$$

where:

E_d is the design effects of actions specified in the serviceability criterion; and

C_d is the limiting value of the relevant serviceability criterion.

6.8.1.2 Where appropriate, allowance should be made for elastic shortening of the pile shaft under axial loading.

6.8.2 Individual piles

*COMMENTARY ON **6.8.2***

In order to keep settlement to a minimum, it is common practice in the UK to limit the representative load on a friction pile to its characteristic shaft resistance (thereby discounting its base resistance). This is not a requirement of BS EN 1997-1:2004+A1:2013.

In this context, the relevant serviceability criterion C_d in equation (82) is the shaft resistance of the pile calculated for ultimate limit state conditions.

6.8.2.1 Settlement of a pile foundation may be verified by satisfying the following serviceability criterion:

$$F_{c,rep} \leq \frac{R_{s,k}}{\gamma_{s,SLS}} \tag{83}$$

where:

$F_{c,rep}$ is the representative value of the compressive force applied to the pile in its serviceability limit state;

$R_{s,k}$ is the characteristic value of the pile's ultimate shaft resistance; and

$\gamma_{s,SLS}$ is a partial factor for shaft resistance in the serviceability limit state.

NOTE The symbol $\gamma_{s,SLS}$ is introduced here to prevent potential confusion with the value of γ_s used under ultimate limit state conditions.

6.8.2.2 The value of $\gamma_{s,SLS}$ should be taken as a minimum of 1.2.

NOTE 1 With $\gamma_{s,SLS} \geq 1.0$, the settlement of an individual pile can be limited to less than 3% of its diameter, D; with $\gamma_{s,SLS} \geq 1.2$, the settlement can be limited to less than 1.5% D.

NOTE 2 Use of equation (83) can lead to uneconomic designs when:

- *the serviceability limit state has been otherwise verified by calculation or load testing;*
- *settlement of the pile is not a concern;*
- *the pile is installed by driving into competent ground; or*
- *the stiffness of the ground below the pile toe has been improved significantly (for example, by base grouting).*

6.8.2.3 The design value of the compressive force applied to an individual pile at its serviceability limit state should be calculated from equation (69) with the values of the partial factors $\gamma_{F,i}$ and γ_G normally taken as 1.0.

NOTE The symbol $\gamma_{s,SLS}$ is introduced here to prevent potential confusion with the value of γ_s used under ultimate limit state conditions.

6.9 Structural design

6.9.1 General

6.9.1.1 The structural design of pile foundations should conform to **4.8**.

6.9.1.2 The structural design of individual piles should consider:

- compressive and tensile resistance of the pile shaft;
- shear resistance of the pile shaft;
- bending resistance of the pile shaft;
- torsional resistance of the pile shaft;
- buckling resistance of the pile shaft, particularly in the absence of lateral restraint from the ground, for example if a gap opens up around the pile during installation (so-called "post-holing");
- weakening of the pile material as a result of corrosion or other forms of deterioration;
- connection of the pile to a pile cap or slab;
- flared pile heads;
- combinations of axial load and moments; and
- concrete crack widths.

6.9.1.3 For the purposes of its structural design, the design value of the compressive force applied to an individual pile at the ultimate limit state should be calculated from equation (64).

6.9.1.4 The likelihood of some degree of eccentric loading on foundations consisting of only one or two piles should be considered. The piles should be designed to resist the bending which results or the pile cap should be effectively restrained from lateral or rotational movements. The restraint and the pile section or both should be sufficient to resist the moments due to eccentric loading or other causes.

6.9.2 Bored cast-in-place concrete piles

*COMMENTARY ON **6.9.2***

*BS EN 1992-1-1:2004, **2.3.4.2** provides supplementary requirements for cast-in-place piles without permanent casing. These supplementary requirements involve reducing the diameter of these piles by 5% for the purposes of their structural design. This reduction is unnecessary when the execution of these piles conforms to BS EN 1536 or BS EN 14199.*

*BS EN 1992-1-1:2004, **2.4.2.5** provides partial factors for materials for foundations, including cast-in-place piles without permanent casing. A factor k_f is introduced as a multiplier to the partial factor for concrete γ_C, with a recommended value of 1.1. UK National Annex to BS EN 1992-1-1:2004, Table NA.1 specifies use of this recommended value in the UK. Thus, in permanent and transient design situations, the "effective" partial factor on cast-in-place concrete in piles without permanent casing is:*

$$k_f \times \gamma_C = 1.1 \times 1.5 = 1.65 \qquad (84)$$

*BS EN 1992-1-1:2004+A1:2014, **9.8.5** provides application rules for detailing cast-in-place concrete piles, in particular with regard to the minimum number and minimum diameter of longitudinal bars. These rules differ from the requirements of BS EN 1536:2010+A1:2015, which states:*

- ***7.5.2.3** For reinforced piles the minimum longitudinal reinforcement shall be four bars of 12 mm diameter; and*
- ***7.5.2.5** Spacing of longitudinal bars should always be maximized in order to allow proper flow of concrete but should not exceed 400 mm.*

6.9.2.1 The design compressive resistance ($R_{c,d}$) of the reinforced length of a cast-in-place pile may be calculated from:

$$R_{c,d} = f_{cd}A_{c,d} + f_{yd}A_{s,d} = \left(\frac{\alpha_{CC} \times f_{ck}}{k_f \times \gamma_C}\right)A_{c,d} + \frac{f_{yk}}{\gamma_S}A_{s,d} \qquad (85)$$

where:

f_{ck} and f_{cd}	are the characteristic and design compressive strengths of the reinforced concrete, respectively;
f_{yk} and f_{yd}	are the characteristic and design yield strengths of the steel reinforcement, respectively;
$A_{c,d}$ and $A_{s,d}$	are the cross-sectional area of the reinforced concrete and compressive steel reinforcement, respectively;
α_{CC}	is a factor taking into account the long-term reduction in strength, etc., of reinforced concrete; and
γ_C and γ_S	are partial factors on the strength of the reinforced concrete and the steel reinforcements, respectively.

6.9.2.2 The design compressive resistance ($R_{c,pl,d}$) of the unreinforced length of a cast-in-place pile should be calculated from:

$$R_{c,pl,d} = f_{cd}A_{c,d} = \left(\frac{\alpha_{CC,pl} \times f_{ck}}{k_f \times \gamma_C}\right)A_{c,d} \qquad (86)$$

where:

$\alpha_{CC,pl}$	is a factor taking into account the long-term reduction in strength, etc., of plain concrete; and

the other symbols are defined for equation (85).

6.9.2.3 Values of γ_C, γ_S, $\alpha_{CC,pl}$, and k_f should conform to the UK National Annex to BS EN 1992-1-1.

6.9.2.4 A cast-in-place pile (without permanent casing) that conforms to BS EN 1536 or BS EN 14199 should be considered to meet the "other provisions" specified in BS EN 1992-1-1:2004, **2.3.4.2**(2). Therefore, the diameter used in design calculations of cast-in-place piles without permanent casing should be as given in Table 14.

Table 14 **Design pile diameter for cast-in-place piles without permanent casing**

Nominal pile diameter, d_{nom}	**Design pile diameter, d_d**	
	In absence of other provisions	Conforms to BS EN 1536 or BS EN 14199
d_{nom} < 400 mm	$d_d = d_{nom} - 20$ mm	$d_d = d_{nom}$
400 ≤ d_{nom} < 1 000 mm	$d_d = 0.95\ d_{nom}$	$d_d = d_{nom}$
d_{nom} > 1 000 mm	$d_d = d_{nom} - 50$ mm	$d_d = d_{nom}$

6.9.2.5 Longitudinal bars in cast-in-place concrete piles should conform to BS EN 1536, as follows:

- the minimum diameter should be no less than 12 mm (instead of 16 mm, as stated in BS EN 1992-1-1);
- the minimum number of bars should be no fewer than 4 (instead of 6, as stated in BS EN 1992-1-1).
- the clear distance between bars should be no greater than 400 mm (instead of 200 mm, as stated in BS EN 1992-1-1).

6.9.2.6 Depending on the type and magnitude of loading, a cast-in-place concrete pile may be reinforced over its whole length, over part of its length, or merely provided with short splice bars at the top for bonding into a pile cap. If a cast-in-place concrete pile is required to resist tensile forces, its reinforcement should extend over the full length of pile that is subjected to those tensile forces (including into any enlarged base, if necessary).

6.9.2.7 Reinforcement should be provided to resist tensile forces that might arise due to swelling of unloaded ground. In the temporary condition, it might be acceptable not to fully reinforce the pile.

6.9.3 Driven cast-in-place concrete piles

The structural design of driven cast-in-place concrete piles should conform to **6.9.2**.

6.9.4 Prefabricated piles

6.9.4.1 Precast concrete piles

*COMMENTARY ON **6.9.4.1***

The structural design of precast concrete piles is often governed by the stresses that occur during handling (lifting, stacking, and transporting) and installation into the ground.

6.9.4.1.1 The manufacture of precast concrete piles should conform to BS EN 12794.

6.9.4.1.2 Precast concrete piles should be designed to withstand the stresses that occur during handling (lifting, stacking, and transporting) and installation into the ground.

6.9.4.1.3 Lifting points should be clearly marked on all precast concrete piles.

NOTE Guidance on the manufacture of precast concrete piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B2 and C2 Driven pre-cast concrete piles *(SPERW [N3]).*

6.9.4.2 Steel bearing piles

*COMMENTARY ON **6.9.4.2***

The structural design of steel bearing piles is often governed by the stresses that occur during installation into the ground.

Steel bearing piles should be designed to withstand the stresses that occur during handling (lifting, stacking, and transporting) and installation into the ground.

NOTE Guidance on a suitable sheet pile section to withstand driving in different ground conditions can be found in the Piling Handbook *[90].*

6.9.4.3 Helical steel piles

The structural design of helical steel piles should conform to **A.6**.

6.9.4.4 Timber piles

NOTE Guidance on the design of timber piling can be found in BRE Digest 479 *[43].*

6.10 Execution

NOTE 1 Guidance on the impact of piling on archaeology can be found in the English Heritage guidance notes on piling and archaeology *[91].*

NOTE 2 Attention is drawn to Regulation 22 of The Construction (Design and Management) Regulations 2015 *[3], with regards to health and safety requirements for excavations.*

6.10.1 General

The execution of pile foundations should conform to **4.9** and this subclause (**6.10**).

6.10.2 Bored cast-in-place concrete piles

The execution of bored cast-in-place piles should conform to BS EN 1536.

NOTE 1 Guidance on the installation of bored cast-in-place concrete piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B3 and C3, Bored cast-in-place piles *(SPERW [N3]).*

NOTE 2 Guidance on the installation of continuous flight auger piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B4 and C4, Piles constructed using continuous flight augers or displacement augers *(SPERW [N3]).*

6.10.3 Driven cast-in-place concrete piles

The execution of driven cast-in-place piles should conform to BS EN 12699.

NOTE Guidance on the installation of driven cast-in-place concrete piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B5 and C5, Driven cast-in-place piles *(SPERW [N3]).*

6.10.4 Prefabricated piles

6.10.4.1 Precast concrete piles

The execution of precast concrete piles should conform to BS EN 12699.

NOTE Guidance on the installation of precast concrete piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B2 and C2, Driven pre-cast concrete piles *(SPERW [N3]).*

6.10.4.2 Steel bearing piles

The execution of steel bearing piles should conform to BS EN 12699.

NOTE Guidance on the installation of steel bearing piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B6 and C6, Steel bearing piles *(SPERW [N3]).*

6.10.4.3 Helical steel piles

The execution of helical steel piles should conform to **A.7**.

6.10.4.4 Timber piles

*COMMENTARY ON **6.10.4.4***

Hard driving of timber piles is likely to broom the butts, crush the tips, and cause cracking. Such damage will lead to a loss of structural strength and might well nullify the effect of preservatives.

6.10.4.4.1 The execution of timber piles should conform to BS EN 12699.

6.10.4.4.2 Where untreated softwood piles form the foundation of a permanent structure, their heads should be cut off below the lowest anticipated ground-water level. If necessary, the pile heads may be extended in concrete or other suitable material.

6.10.4.4.3 The length of any timber piles embedded in a concrete pile cap should be sufficient to ensure full transmission of the applied load. Concrete cover around timber piles should be at least 150 mm and suitable to prevent splitting of the pile.

NOTE 1 Guidance on the installation of timber piles can be found in the ICE Specification for piling and embedded retaining walls, Sections B7 and C7 Timber piles *(SPERW [N3]).*

NOTE 2 Guidance on the installation of timber piling can be found in BRE Digest 479 *[43].*

6.10.5 Micro piles

The execution of micro piles should conform to BS EN 14199.

6.11 Testing

*COMMENTARY ON **6.11***

Static pile load tests are performed on test piles in order to determine the resistance versus displacement characteristics of the pile and the surrounding ground.

Test piles may be any of the following:

- *Preliminary piles – piles that are installed before commencement of the main piling works for the purpose of establishing the suitability of the chosen pile type and for confirming its design, dimensions, and bearing resistance;*
- *Trial piles – piles that are installed to determine the practicablity and suitability of the construction method for a particular application; and*
- *Working piles – piles that are incorporated into the finished works.*

6.11.1 General

Pile load tests should be performed under the circumstances identified in BS EN 1997-1:2004+A1:2013, **7.5.1**.

6.11.2 Investigation tests

*COMMENTARY ON **6.11.2***

Investigation tests typically involve loading a sacrificial pile up to its ultimate limit state.

Investigation tests have traditionally been known as "preliminary pile tests".

6.11.2.1 Investigation tests should conform to one of the methods described in **6.11.4**.

6.11.2.2 Investigation tests may be used to determine the expected performance of a pile type, under specific loading conditions, in the ground conditions relevant to the construction project.

6.11.2.3 At least 1 preliminary pile for every 500 working piles should normally be subjected to an investigation test.

6.11.2.4 Whenever practicable, piles that are to be subjected to investigation tests should be installed before commencement of the main piling works. Working piles that are installed before the results of investigation tests are available should be designed with the larger model factor given in the UK National Annex to BS EN 1997-1:2004+A1:2013, **A.3.3.2**.

6.11.2.5 Detailed notes should be taken during investigation tests to assist in their subsequent evaluation.

6.11.2.6 Investigation tests should be supervised by people experienced in pile testing.

NOTE Guidance on pile capacity testing can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 98 *[1].*

6.11.3 Control tests

*COMMENTARY ON **6.11.3***

Control tests typically involve loading a working pile up to a specified load in excess of its serviceability limit state.

Control tests have traditionally been known as "working pile tests".

6.11.3.1 Control tests should conform to one of the methods described in **6.11.4**.

6.11.3.2 Control tests may be used to determine the performance of a particular pile under working loads, in the ground in which it has been installed.

6.11.3.3 At least 1 working pile for every 100 working piles should normally be subjected to a control test.

6.11.3.4 Detailed notes should be taken during control tests to assist in their subsequent evaluation.

6.11.3.5 Control tests should be supervised by people experienced in pile testing.

6.11.4 Static load tests

6.11.4.1 General

Static load tests may be used to determine the ultimate static resistance of a single pile and its service performance under static load.

6.11.4.2 Axial compression

Tests that subject the pile to static axially loaded compression should be performed in accordance with the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B15 and C15, Static load testing of piles*.

6.11.4.3 Axial tension

Tests that subject the pile to static axially loaded tension should be performed in accordance with the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B15 and C15, Static load testing of piles*.

6.11.4.4 Transverse loading

Tests that subject the pile to static transversally loaded tension should be performed in accordance with the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B15 and C15, Static load testing of piles*.

6.11.4.5 Thermal integrity profiling

Integrity tests that use thermal integrity profiling should be performed in accordance with ASTM D7949-14.

6.11.5 Dynamic and rapid load tests

6.11.5.1 General

6.11.5.1.1 Dynamic and rapid load tests may be used to determine the ultimate static resistance of a single pile and its service performance under static load.

6.11.5.1.2 Rate effects (including creep), excess pore water pressures, and inertia effects (due to acceleration) may be present in a dynamic or rapid load test and might differ from what would be expected from an equivalent static load test. The results of dynamic and rapid load tests should be calibrated against static load test results in order to account for these effects.

6.11.5.2 Dynamic load testing

Tests that subject the pile to dynamic loading should be performed in accordance with the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B14 and C14, Dynamic and rapid load testing*.

6.11.5.3 Rapid load testing

Tests that subject the pile to rapid loading should be performed in accordance with the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B14 and C14, Dynamic and rapid load testing*.

6.11.6 Integrity tests

6.11.6.1 General

6.11.6.1.1 Integrity tests may be used to determine the integrity of cast-in-place piles, barettes, and retaining wall elements after their installation in the ground.

6.11.6.1.2 When performing integrity tests, one of the following should be used:

- impulse response method;
- sonic echo, frequency response, or transient dynamic steady-state vibration method;
- cross-hole sonic logging method; or
- thermal integrity profiling.

NOTE Guidance on pile integrity testing can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 97 *[1].*

6.11.6.2 Impulse response method

Integrity tests that use the impulse response method should be performed in accordance with the relevant parts of the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B13 and C13, Integrity testing*.

6.11.6.3 Sonic echo, frequency response, or transient dynamic steady-state vibration method

Integrity tests that use the sonic echo, frequency response, or transient dynamic steady-state vibration method should be performed in accordance with the relevant parts of the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B13 and C13, Integrity testing*.

6.11.6.4 Cross-hole sonic logging method

Integrity tests that use the cross-hole sonic logging method should be performed in accordance with the relevant parts of the *ICE Specification for piling and embedded retaining walls* (SPERW [N3]), *Sections B13 and C13, Integrity testing*.

6.12 Supervision, monitoring, and maintenance

6.12.1 Supervision of construction

The construction of a pile foundation should be supervised by an appropriately qualified and experienced person.

6.12.2 Monitoring

Monitoring of helical steel piles should conform to **A.8**.

6.12.3 Maintenance

6.12.3.1 If the design of the pile foundation relies upon a dewatering system, maintenance of that system should be ensured.

6.12.3.2 Where bolted connections to piles are used, maintenance of those connections should be ensured.

6.13 Reporting

6.13.1 Ground Investigation Report

The Ground Investigation Report for a pile foundation should conform to **4.12.1**.

6.13.2 Geotechnical Design Report

The Geotechnical Design Report for a pile foundation should conform to **4.12.2** and should also include, where appropriate:

- assumed design values for β, K_s, δ and N_q (defined in **6.4.1.2.2**) and any correlations on which these values are based;
- assumed design values for α and N_c (defined in **6.4.1.2.3** and any correlations on which these values are based;
- assumed design values for c_s and c_b (defined in **6.4.1.3.2**);
- assumed design values for n_s and n_b (defined in **6.4.1.3.3**); and
- any assumptions regarding groundwater.

6.13.3 Geotechnical Feedback Report

6.13.3.1 The Geotechnical Feedback Report for a pile foundation should conform to **4.12.3** and this clause (**6.13.3**).

6.13.3.2 The load test report required for all load tests by BS EN 1997-1:2004+A1:2013, **7.5.4**, should be included in the Geotechnical Feedback Report.

6.13.3.3 The Geotechnical Feedback Report for a pile foundation should also include, where appropriate:

- records of all pile installations;
- survey results of as-built pile locations relative to a site datum;
- survey results of as-built pile levels relative to surrounding ground levels;
- any variation from the specified construction works;
- the design action for which each pile has been designed;
- levels and measurements to determine ground heave or movement of any pile resulting from the piling operation;
- details of all obstructions, delays, and other interruptions to the piling operations;
- for prefabricated piles:
    - details of the type and positions of joints;
- for cast-in-place concrete piles:
    - the method used;
    - concrete mix details, including: concrete class, nominal maximum aggregate size; cement content, water/cement ratio, details and quantity of any admixtures or cement replacement material used;
    - reinforcement details, including: number, type, and size of main reinforcement bars, type and size of shear links, the reinforced length of pile; and
- record drawings showing the piles as constructed.

6.13.3.4 The Geotechnical Feedback Report should be prepared jointly by the pile designer and piling contractor.

6.13.3.5 The Geotechnical Feedback Report for a helical pile foundation should also conform to **A.9.3**.

Annex A (normative) Helical steel pile foundations

*COMMENTARY ON **Annex A***

This annex applies to the design and construction of helical steel piles.

Helical (screw) piles generally comprise a lead section, which comprises helix plates attached to a central steel shaft, and extension sections.

Helical steel piles are typically suitable for use in loose to medium dense coarse soils and low to medium strength fine soils.

A.1 General

The design of helical steel pile foundations should conform to BS EN 1997-1:2004+A1:2013, Section 7**,** Clause **4** and Clause **6** of this standard, and this annex.

A.2 Design considerations

A.2.1 General

A.2.1.1 The design of helical steel pile foundations should conform to **6.3.1**.

A.2.1.2 The helix pitch should not be less than 75 mm.

A.2.1.3 The gradient of the helix plates should be constant throughout their pitch.

A.2.1.4 The outer diameter of the helix should be circular in plan and undamaged by the forming process.

A.2.1.5 The shaft of a helical pile should be manufactured from circular hollow sections (CHS) or square sections, with the former being preferred.

A.2.1.6 To prevent formation of an annulus around the helical pile shaft, the diameter of any shaft extensions should be constant or decrease with depth. The diameter of any oversized connections should also be constant or decrese with depth.

A.2.1.7 Connections between shaft extensions should allow full transfer of bending moments (i.e. they should not be a pinned connection). Connections should not oversize the hole formed during installation to create an annulus around the helical pile shaft.

A.2.1.8 Where threaded connections are used, the direction of the thread should be opposite to the direction of rotation during installation. A locking device for the connection should be used.

A.2.1.9 The design of helical piles should be based on a conventional soil mechanics approach supported by pile testing data where this is combined with an empirical approach.

A.2.1.10 Helical piles should not be designed solely on empirical rules relating soil resistance to the applied torque measured during pile installation.

A.2.1.11 Particular consideration should be given to the effects of the following:

- actions that are applied repeatedly;
- actions with varying intensity;
- actions that produce a dynamic response in the structure or the ground; and
- water pressures.

A.2.2 Ground investigation

Ground investigation for helical pile foundations should conform to BS EN 12699.

A.2.3 Pile spacing

A.2.3.1 The plan location and inclination of helical piles should conform to BS EN 12699.

A.2.3.2 The bearing resistance of helical piles spaced closer than 4 helix diameters (centre-to-centre on plan) should be reduced to account for interaction between the helices.

A.2.4 Calculation models

NOTE 1 Guidance on the design of helical piles can be found in Helical piles – a practical guide to design and installation *[92].*

NOTE 2 The tensile resistance of a helical pile is typically calculated as the sum of the resistances of its helix plates acting in pull-out. This is known as the "reverse bearing capacity model". This calculation model ignores any beneficial effect of shaft friction.

A.3 Materials

A.3.1 Materials used to fabricate helical steel piles should conform to **6.5.2**.

A.3.2 Hot finished helical steel piles should also conform to BS EN 10210.

A.3.3 Cold formed helical steel piles should also conform to BS EN 10219.

A.3.4 Helical steel piles should be manufactured in accordance with BS EN 1090.

A.4 Durability

The durability of helical steel piles should conform to **6.6**.

A.5 Ultimate limit state design

A.5.1 Bearing

A.5.1.1 For the purpose of selecting partial factors from BS EN 1997-1, helical piles should be classified as shown in Table 8, Table 9 and Table 10.

A.5.1.2 For helical piles, the following potential ultimate limit states should be checked:

- failure of a helix in bearing;
- a combination of helix bearing failure and plugged shaft failure;
- base punching failure into underlying weak layers; and
- rotational failure mechanisms caused by the application of torque to the pile head (typically only relevant to single piles or pairs of piles).

A.5.1.3 The potential contribution to bearing resistance from friction on the pile shaft should be ignored, unless there is definitive evidence from pile load tests that the soil between the helices does not form a plugged shaft.

NOTE Plugged shaft failure might not need to be checked if there is sufficient vertical spacing between the helices to ensure that a plugged shaft does not form.

A.5.2 Tension/pull-out

The potential contribution to pull-out resistance from friction on the pile shaft should be ignored, unless there is definitive evidence from pile load tests that the soil between the helices does not form a plugged shaft.

A.6 Structural design

The structural design of helical piles should consider:

- compressive, tensile, and bending resistance of the shaft;
- torsional resistance of the shaft;
- buckling resistance of the shaft, particularly in the absence of lateral restraint from the ground, for example if installation postholing occurs;
- failure of the connections between shaft segments;
- structural failure of the helix plate;
- failure of the welds between the shaft and the helix; and
- weakening of the pile as a result of corrosion.

A.7 Execution

A.7.1 The penetration rate of a helical pile per revolution of the pile (v) should satisfy:

$$0.85\,p \le v \le 1.15p \qquad (A.1)$$

where:

p is the pitch between adjacent helices.

A.7.2 A variable axial force should be applied to the pile head to ensure that (A.1) is satisfied.

NOTE The axial force applied to the pile head to advance the pile into the ground is commonly known as the "crowd force" (or just "crowd").

A.7.3 If the penetration rate falls outside the limits given above, the design resistance of the pile should be re-assessed and downgraded appropriately unless testing validates the pile's resistance.

A.7.4 Installation of a helical pile should cease immediately if its penetration rate falls below 0.5 p. The pile's design resistance should be reassessed and downgraded appropriately.

A.7.5 If the rotation of a helical pile is reversed during installation (i.e. the pile is "back-spun"), then its design resistance should be re-assessed and downgraded, if appropriate, unless testing validates the pile's resistance.

A.7.6 Installation of a helical pile should cease immediately if the installation torque exceeds its maximum design value before the pile has reached its design depth of penetration.

A.7.7 The rotation speed during installation of a helical pile should:

- not exceed 20 revolutions per minute;
- be appropriate to the anticipated ground conditions;
- be appropriate for the responsiveness of the installation equipment; and
- be reduced appropriately where unacceptable penetration rates are observed.

A.7.8 If pre-drilling is used to assist pile installation, the diameter of the pre-drilled hole should not exceed the pile's shaft diameter. To prevent gaps occurring between the pile shaft and the surrounding ground, pre-drilling should not result in the removal of material.

A.7.9 Helical piles should be removed from the ground by unscrewing, in a similar manner to their installation.

A.7.10 A helical pile should not be re-used after removal without it first being confirmed by inspection (by either the manufacturer or installer) that the pile's condition is such that it continues to provide the required design strength and durability. The results of this inspection should be recorded on the piling record.

A.7.11 If a new pile is to be installed in the same location as a previous removed helical pile, proper allowance should be made in the design for the disturbance of the ground caused by the previous pile.

A.7.12 The mean installation torque measured over the final 1 m of installation should be used to check compliance with the design torque requirements.

A.7.13 If the torsional resistance of the pile shaft is exceeded prior to achieving the required depth of installation, either the installation should be terminated at the depth reached or the pile should be replaced by a new one with smaller diameter helices, fewer plates, or a larger shaft diameter.

A.7.14 If the minimum design installation torque is not achieved at the required depth of installation:

- the depth of penetration should be increased by adding extension sections to the pile; or
- the pile should be replaced by a new one with additional and/or larger diameter plates; or
- the pile should be relocated.

A.8 Monitoring

A.8.1 The following information should be collected during the execution of helical piles:

- installation torque should be monitored throughout installation and recorded during the last 1 m of penetration (at intervals not less than 0.5 m);
- rotation speed and penetration rate should be monitored throughout installation to ensure the pile is not flighting; and
- depth and diameter of any pre-drilling, details of any pile flighting, and details of any pre-augering should be recorded.

A.8.2 This information should be collected at intervals not exceeding 500 mm of penetration.

A.8.3 Appropriate adjustments should be made to the installation procedure and, when necessary, the pile resistance, based on the information recorded.

A.9 Reporting

A.9.1 Ground Investigation Report

The Ground Investigation Report for a helical pile foundation should conform to **4.12.1**.

A.9.2 Geotechnical Design Report

The Geotechnical Design Report for a helical pile foundation should conform to **6.13.2** and should also include, where appropriate, assumed design values for torque and any correlations on which these values are based.

A.9.3 Geotechnical Feedback Report

The Geotechnical Feedback Report for a helical pile foundation should conform to **6.13.3** and should also include, where appropriate:

- installation torque records and associated calibration data;

- number of pile rotations per minute;
- rate of penetration (distance travelled per revolution and per unit of time);
- depth and diameter of any pre-drilling;
- details of any pile flighting or augering;
- final depth of bottom plate.

Annex B (normative) Underpinning

*COMMENTARY ON **Annex B***

This annex applies to the design and construction of underpinning.

B.1 General

*COMMENTARY ON **B.1***

Underpinning is commonly employed to rectify distress caused to a building by excessive movement of its foundation, to extend foundations into the ground to facilitate future construction work, to accommodate additional loading applied to an existing building, to allow adjacent ground to be lowered, or to change the support system.

The design of underpinning should conform to BS EN 1997-1, and Clause **4** of this standard and this annex.

B.2 Choice of structure

NOTE Underpinning may be provided by:

- *extending the depth of existing pad or strip footings;*
- *constructing ground beams beneath existing walls by stooling (and supporting them on piers or piles);*
- *constructing needle beams through existing walls (and supporting them on piers or piles);*
- *constructing piles that are drilled through existing foundations.*

The choice of underpinning system should consider the loads to be carried, the sensitivity of the structure and the ground, and working conditions.

NOTE 1 Guidance about underpinning can be found in the ICE manual of geotechnical engineering (2012), Volume II, Chapter 83 *[1].*

NOTE 2 See Guidelines on safe and efficient basement construction directly below or near to existing structures *[93].*

NOTE 3 Guidance on health and safety in geotechnical engineering can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 8 *[2].*

NOTE 4 Further information about underpinning can be found in Underpinning and retention (2nd edition) *[94].*

B.3 Design considerations

B.3.1 Before underpinning is commenced, a comprehensive inspection, carefully recorded and photographed, should be undertaken to identify the condition of the structure that is to be underpinned and the condition of those structures adjacent to it.

B.3.2 The structure to be underpinned should be carefully examined for indications of differential foundation movement and for inherent weaknesses which might be accentuated during the process of underpinning, and the structure should be temporarily supported or strengthened. Particular care should be exercised when underpinning piers, columns or walls pierced by openings.

B.3.3 Underpinning systems should be designed in accordance with good soil mechanics practice taking into account all vertical and lateral loads imposed upon them, including particularly transient conditions that might arise during construction.

B.3.4 The whole foundation, including that part modified by underpinning, should continue to perform satisfactorily during and after underpinning. If the changed support conditions give rise to excessive differential movement, jacks should be installed temporarily or permanently to correct this. If a structure subsides unevenly, the whole structure should be underpinned and partial underpinning should not be used.

NOTE Guidance on condition surveys can be found in Appraisal of existing structures *[95],* Inspection manual for highway structures, Volumes 1 and 2 *[96], [97].*

B.4 Materials

B.4.1 Concrete

Concrete and related products incorporated into underpinning should conform to **4.3.6**.

B.4.2 Steel

Steel and related products incorporated into underpinning should conform to **4.3.7**.

B.4.3 Timber

Timber and related products incorporated into underpinning should conform to **4.3.8**.

B.4.4 Soils, rocks, and rock masses

Particular attention should be paid during the ground investigation for underpinning to determine:

- ground and groundwater conditions below and adjacent to the structure;
- conditions responsible for any excessive movement so that appropriate underpinning measures can be undertaken;
- the ground's load-bearing capacity where the underpinning is to be founded; and
- the disposition and effect of adjoining foundations and services.

B.5 Durability

B.5.1 Concrete

The durability of concrete and related products used for underpinning should conform to **4.4.2**.

B.5.2 Steel

The durability of steel and related products used for underpinning should conform to **4.4.3**.

B.5.3 Timber

The durability of timber and related products used for underpinning should conform to **4.4.4**.

B.6 Ultimate limit state design

B.6.1 The ultimate limit state design of underpinning involving spread foundations should conform to **5.7**.

B.6.2 The ultimate limit state design of underpinning involving pile foundations should conform to **6.7**.

B.7 Serviceability limit state design

B.7.1 The serviceability limit state design of underpinning involving spread foundations should conform to **5.8**.

B.7.2 The serviceability limit state design of underpinning involving pile foundations should conform to **6.8**.

B.8 Structural design

B.8.1 The structural design of underpinning involving spread foundations should conform to **5.9**.

B.8.2 The structural design of underpinning involving pile foundations should conform to **6.9**.

B.9 Execution

B.9.1 The execution of underpinning should conform to **4.9** and this subclause (**B.9**).

B.9.2 Before excavation is commenced, the live loads on the wall, pier or structure should be reduced as much as practicable where they are large in relation to the dead load, the adjoining owner's consent being obtained where necessary.

B.9.3 All excavations necessary for underpinning should be well supported, using concrete, steel, or timber, and strutted to an approved design, e.g. by an appropriate temporary or permanent geotechnical process, to prevent the surrounding ground from moving before it is supported by the permanent work.

B.9.4 Either steel trench sheeting or concrete poling boards should be used if the shoring cannot be withdrawn. Excavations for access to construct underpinning should be similarly treated and restricted to that essential for the work. The support removed when opening up underpinning excavations will increase the load on adjoining foundations unless it is fully compensated by shoring. These adjoining foundations should not be weakened or undermined by the excavation of access trenches or approach excavations.

B.9.5 Where underpinning will not give rise to excessive ground pressure, the procedure should be to execute underpinning in a series of legs, the length of each leg depending on the general character and condition of the structure to be underpinned, the intensity of the loading and the nature of the ground below. Generally, in brick and/or stone walls of normal type, each leg should be 1.0 m to 1.4 m in length; in walls capable of arching, the length of each leg may be increased accordingly.

B.9.6 Each series of legs should be planned to provide sufficient support between the legs under construction, and to ensure that the loads from the unsupported portions of the wall are distributed throughout the length of the wall. Attention should also be given to positions of openings and piers immediately above the foundations, so that sections of the structure carrying local heavy loads are not left unsupported. No fresh series of legs may be commenced until the preceding underpinning is completed and finally pinned. The sequence in which the underpinning is carried out should be executed in such a way as to avoid differential settlement and ensure load transfer.

B.9.7 The area of open excavations should not exceed 25% of the building's footprint. This limit should be reduced if the building comprises a number of isolated piers.

B.9.8 Where underpinning is required on account of the settlement of the foundation, or where the safe ground pressure is likely to be exceeded during the underpinning operations, the structure to either side of an underpinning leg should be needled and the load transferred to temporary bearings on ground capable of carrying the additional load or other support. The siting of these bearings should be determined to avoid undermining by any operations necessary to complete the underpinning leg. When this is complete, underpinning can commence.

B.9.9 Alternatively, it might be possible to introduce stooled reinforced concrete transfer beam prior to undertaking underpinning.

B.9.10 The construction of the underpinning leg should be commenced immediately after the bottom of the excavation has been exposed. In all cases where the bottom is likely to be affected by exposure to the atmosphere, the last section of the excavation should not be taken out unless the underpinning can proceed forthwith. The bottom of the excavation supporting the underpinning should be sealed with concrete immediately after inspection has shown it to be satisfactory.

B.9.11 Before the construction of a leg, the underside of the old wall or foundation should be cleaned and levelled ready for the new pinning. All joints against legs already constructed should be thoroughly cleaned. The underpinning leg should be constructed as quickly as possible up to within 75 mm to 150 mm of the underside of the old foundation ready for final pinning. The top of the new work should be left smooth and flat to facilitate the final pinning.

B.9.12 As soon as the concrete or brickwork is strong enough to support the load to be placed upon it, the final pinning should be carried out. The final pinning should consist of a fairly dry concrete mix, meaning that only sufficient water has been added to moisten the mixture so that it will remain a ball when squeezed in the hand (the maximum size of aggregate should be 10 mm). The mix should be rammed in hard to make solid contact with the soffit of the underpinned structure.

B.9.13 If the width of the foundation to be underpinned is greater than 1 m, it is advisable to leave the outer half more than 150 mm below the underside of the old foundation to facilitate the pinning up of the furthest part.

B.10 Monitoring

B.10.1 During underpinning, frequent checks for movement or distress in the structure should be made.

B.10.2 In most cases careful inspection is adequate but in special circumstances a detailed schedule of dilapidations should be prepared.

NOTE Crack monitoring, levelling and plumbing might be necessary as the work proceeds.

B.11 Reporting

Reporting for underpinning should conform to **4.12**.

Annex C (informative) Specific formations

C.1 London Clay

London Clay was deposited in marine conditions in the Eocene epoch (30 million years ago). London Clay is made up of various silty clay and sandy clayey silt units, separated by glaucontic rich horizons. Very fine sand and silt dustings, partings, and lenses are frequent in the siltier clays; and sand layers occur in the sandy clayey silts. Phosphatic and claystone nodules are quite common throughout the deposit.

The London Clay in central London is one of the most highly investigated soils in the world.

NOTE A summary of the characteristics of London Clay can be found in Some characteristics of London Clay *[98] and* The London Clay at T5 *[99].*

C.2 Gault Clay

The Gault Clay comprises a sequence of clays, mudstones, and thin siltstones with bands of phosphatic nodules of Middle and Upper Albian age. It outcrops in East Anglia, Wessex, Dorset, North East Kent, Surrey, and Hampshire.

Gault Clay causes a number of serious geotechnical problems, including ancient and recent landslides, and can contain sufficient sulfate and sulfuric acid for potential chemical attack on concrete. Seasonal shrinkage and swelling of this highly expansive soil can result in damage to buildings.

NOTE Guidance on the engineering geology of Gault Clay can be found in BGS Technical Report WN/94/31, Engineering Geology of British Rocks and Soils – Gault clay *[100].*

C.3 Lambeth Group

The Lambeth Group (previously known as the Woolwich and Reading Beds) is a complex sequence of gravels, sands, and clays that vary considerably both horizontally and vertically and whose properties range between those of an engineering soil and a rock.The Lambeth Group underlies much of south-east England, particularly in London and Hampshire, and, as a result, is frequently encountered in major construction projects. CIRIA identified the Lambeth group as one of the "economically important UK soils and rocks" (CIRIA C583 [101]).

NOTE 1 Guidance on the engineering properties of the Lambeth Group can be found in CIRIA C583 *[101].*

*NOTE 2 Guidance on the engineering geology of the Lambeth Group can be found in B*GS Open Report OR/13/006, Engineering Geology of British Rocks and Soils – Lambeth Group *[102].*

C.4 Glacial soils and tills

Glacial deposits are widespread throughout the world and are frequently encountered in the upland parts of the United Kingdom. Glacial tills and soil are amongst the most difficult to engineer, owing to their marked variation in both thickness and engineering properties.

NOTE 1 Guidance on the classification of glacial tills can be found in Chapter 4 of CIRIA C504 *[103].*

NOTE 2 Guidance on the engineering properties of glacial tills can be found in Chapter 5 of CIRIA C504 *[103].*

NOTE 3 Information about issues relevant to glacial soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 31 *[2].*

C.5 Problematic soils

C.5.1 Problematic soils include materials that display siginificant volume change, a distinct lack of strength, or are potentially corrosive. Problematic soils are profoundly influenced by the climatic regime in which they were developed. (See *ICE manual of geotechnical engineering, Volume II* [1] for further information).

C.5.2 Problematic soils include (but are not limited to): arid soils, tropical soils, glacial soils, collapsible soils, expansive soils, non-engineered fills, organics/peat soils, sulfate/acid soils, and soluable ground.

NOTE 1 Information about problematic soils and their issues can be found in the ICE manual of geotechnical engineering (2012), Volume I, Section 3 *[2].*

NOTE 2 Information about shrinkable (also known as expansive) soils can be found in the ICE manual of geotechnical engineering (2012), Volume I, Chapter 33 *[2].*

C.6 Chalk

Chalk forms the downland of southern England, the Wolds of eastern England, and the white cliffs of Antrim, East Yorkshire, Dover, and from the Seven Sisters to Dorset. The chalk is the UK's most important aquifer for potable water supply. A great deal of construction and infrastructure development is built on chalk.

NOTE 1 Guidance on the description and classification of chalk can be found in CIRIA C574, Chapter 3 *[N2].*

NOTE 2 Guidance on the mechanical properties of chalk can be found in CIRIA C574, Chapter 4 *[N2].*

C.7 Mercia Mudstone Group

The Mercia Mudstone group of rocks underlies much of northern, central, and southern England and parts of Northern Ireland. The engineering properties of the rocks and the derived soils are important as they are frequently encountered in excavations and as founding strata.

NOTE 1 Guidance on the description and classification of Mercia mudstone can be found in CIRIA C570, Section 2, Geological background *[104].*

NOTE 2 Guidance on the engineering properties of Mercia mudstone can be found in CIRIA C570, Section 5, Correlation of engineering properties to the SPT and Section 6, In-situ properties and behaviour of Mercia mudstone *[104].*

NOTE 3 Guidance on the engineering geology of Mercia mudstone can be found in BGS Report RR/01/02, Engineering geology of British rocks and soils: Mudstones of the Mercia Mudstone Group *[105].*

C.8 Lias Group

The Lias Group encompasses an important group of geological materials, comprising clay-rich mudstones interlayered with limestones. The outcrop of the Lias extends in a continuous band from the coast of Dorset in a north-north-easterly direction to Yorkshire, with outlying areas in Somerset and South Wales.

NOTE Guidance on the engineering geology of the Lias Group can be found in BGS Internal Report OR/12/032, Engineering Geology of British Rocks and Soils – Lias Group *[106].*

Annex D (informative)

Archaeological finds

A key element of the UK Government's policy for managing the historic environment in England is the "presumption in favour of sustainable development" [107]. To achieve this, building foundations are often constructed above important archaeological deposits or, in the case of piled foundations, through them.

The UK Government's National Planning Policy Framework [107] sets out core planning principles regarding management of change to the Historic Environment in England, including "conserve heritage assets in a manner appropriate to their significance, so that they can be enjoyed for their contribution to the quality of life of this and future generations".

Scottish Planning Policy in relation to archaeology finds requires planning authorities to protect archaeological sites and monuments as an important, finite, and non-renewable resource and to preserve them in-situ wherever possible. Where in-situ preservation is not possible, planning authorities require developers to undertake appropriate excavation, recording, analysis, publication, and archiving before and/or during development [108].

In Northern Ireland, the desirability of preserving archaeological sites and their settings is a first principle in assessing and determining planning applications for construction schemes. Consequently, the avoidance of known or suspected archaeological sites is a key element of the design process. Any archaeological remains found on a construction site should be preserved in-situ as the primary option. Where this is not possible then full recording (through appropriate excavation and survey methods) followed by timely and suitable public dissemination of the information and academics alike is to take place.

The UK Government's Planning Practice Guidance gives further advice on enhancing and conserving the historic environment [109].

Bibliography

Standards publications

For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 6349-1-3, *Maritime works – Part 1-2: General – Code of practice for geotechnical design*

BS 8103-1, *Structural design of low-rise buildings – Part 1: Code of practice for stability, site investigation, foundations, precast concrete floors and ground floor slabs for housing*

BS EN 335-1:1992, *Hazard classes of wood and wood-based products against biological attack – Part 1: Classification of hazard classes* [14)]

BS EN 335-2:1992, *Durability of wood and wood-based products – Definition of use classes – Part 2: Application to solid wood* [14)]

BS EN 335-3:1992, *Hazard classes of wood and wood-based products against biological attack – Part 3: Application to wood-based panels* [14)]

BS EN 338, *Structural timber – strength classes*

BS EN 1538:2010, *Execution of special geotechnical works – Diaphragm walls*

BS EN ISO 19901-4, *Petroleum and natural gas industries. Specific requirements for offshore structures. Geotechnical and foundation design considerations*

NA+A1:2012 to BS EN 1993-5:2007, *UK National Annex to Eurocode 3: Design of steel structures – Part 5: Piling*

PD 6484:1979, *Commentary on corrosion at bi-metallic contacts and its remediation*

Other publications

[1] BURLAND, J., CHAPMAN, T., SKINNER, H., and BROWN, M. (editors). *ICE manual of geotechnical engineering, Volume II, Geotechnical Design, Construction and Verification*. London: ICE Publishing, 2012, ISBN 978-0-7277-5709-8:

Section 8 – SUCKLING, T.P. (editor). *Construction processes;*

Chapter 52 – O'BRIEN, A.S. *Foundation types and conceptual design principles;*

Chapter 53 – O'BRIEN, A.S., and FAROOQ, I. *Shallow foundations;*

Chapter 54 – BELL, A., and ROBINSON, C. *Single piles;*

Chapter 55 – O'BRIEN, A.S. *Pile-group design;*

Chapter 56 – O'BRIEN, A.S., BURLAND, J.B., and CHAPMAN, T. *Rafts and piled rafts;*

Chapter 59 – ESSLER, R. *Design principles for ground improvement;*

Chapter 60 – SRBULOV, M., and O'BRIEN, A.S. *Foundations subjected to cyclic and dynamic loads*;

Chapter 80 – PREENE, M. *Groundwater control;*

Chapter 81 – WADE, S., HANDLEY, R., and MARTIN, J. *Types of bearing piles*;

Chapter 83 – JOLLEY, T. *Underpinning*;

[14)] Withdrawn and superseded by BS EN 335:2013.

Chapter 84 – SERRIDGE, C.J., and SLOCOMBE, B. *Ground improvement*;

Chapter 94 – DUNNICLIFF, J., MARR, W.A., and STANDING, J. *Principles of geotechnical monitoring*;

Chapter 96 – GLOVER, S., and CHEW, G. *Technical supervision of site works*;

Chapter 97 – FRENCH, S., and TURNER, M. *Pile integrity testing*;

Chapter 98 – BROWN, M., *Pile capacity integrity testing*;

Chapter 101 – LINDSAY, R., and KEMP, M. *Close-out reports.*

[2] BURLAND, J., CHAPMAN, T., SKINNER, H., and BROWN, M. (editors). *ICE manual of geotechnical engineering*, *Volume I*, *Geotechnical Engineering Principles, Problematic Soils and Site Investigation*. London: ICE Publishing, 2012, ISBN 978-0-7277-5707-4:

Section 3 – JEFFERSON, I. (editor). *Problematic soils and their issues;*

Section 4 – BRACEGIRDLE, A. (editor). *Site investigation;*

Chapter 8 – RANNER, D., and SUCKLING, T. *Health and safety in geotechnical engineering;*

Chapter 9 – O'BRIEN, A.S. and BURLAND, J.B. *Foundation design decisions;*

Chapter 14 – BURLAND, J.B. *Soils as particulate materials*;

Chapter 17 – BURLAND, J.B. *Strength and deformation behaviour of soils*;

Chapter 18 – SANDERSON, D.J. *Rock behaviour*;

Chapter 21 – POWRIE, W. *Bearing capacity theory;*

Chapter 25 – ROGERS, C.D.F. *The role of ground improvement*;

Chapter 26 – BURLAND, J.B. *Building response to ground movements*;

Chapter 31 – CLARKE, B. *Glacial soils;*

Chapter 33 – JONES, L.D., and JEFFERSON, I. *Expansive soils;*

Chapter 34 – BELL, F.G., CULSHAW, M.G., and SKINNER, H.D. *Non-engineered fills;*

Chapter 36 – CZEREWKO, M.A. and CIPPS, J.C. *Mudrocks, clays and pyrite.*

[3] GREAT BRITAIN. *The Construction (Design and Management) Regulations 2015*. The Stationery Office: London, 2015.

[4] NHBC Foundation/Arup, *Efficient design of piled foundations for low-rise housing.* NHBC Design Guide NF21, 2010, ISBN 978-1-84806-106-4.

[5] CHARLES, J.A., and WATTS, K.S. *Building on fill: geotechnical aspects* (BRE Report 424, 2nd edition), Watford: BRE, 2001. ISBN 1-86081-509-X.

[6] CASHMAN, P.M., and PREENE, M. *Groundwater lowering in construction: a practical guide*. London: Spon, 2001, ISBN 978-0-419-21110-5.

[7] POWERS, J.P., CORWIN, A.B., SCHMALL, P.C., and KAECK, W.E. *Construction dewatering and groundwater control: new methods and applications* (3rd edition). New York: Wiley, 2007, ISBN 978-0-471-47943-7.

[8] PREENE, M., ROBERTS, T.O.L., POWRIE, W., and DYER, M.R. *Groundwater control: design and practice* (CIRIA Report C515). London: CIRIA, 2015, ISBN 978-0-8601-7515-5.

[9] NHBC. NHBC Standards. Available from: www.nhbc.co.uk (last viewed 25/6/15).

[10] PHEAR, A.G., and HARRIS, S.J. *Contributions to Géotechnique 1948–2008: Ground Improvement*. Géotechnique, 2008, Volume 58, Issue 51, pp 399–404.

[11] KIRSCH, K., and BELL, A.L. *Ground Improvement (3rd edition)*. CRC Press, 2013. ISBN 978-0-415-59921-4.

[12] CHARLES, J.A., and WATTS, K.S. *Treated ground – engineering properties and performance (*CIRIA Report C572*)*. London: CIRIA, 2002, ISBN 978-0-86017-572-8.

[13] MITCHELL, J.M., and JARDINE, F.M. *A guide to ground treatment (*CIRIA Report C573*)*. London: CIRIA, 2002, ISBN 978-0-86017-573-5.

[14] RAWLINGS, C.G., HELLAWELL, E.E., and KILKENNY, W.M. *Grouting for ground engineering (*CIRIA Report C514*)*. London: CIRIA, 2000, ISBN 978-0-86017-514-8.

[15] BUILDING RESEARCH ESTABLISHMENT. *Design guide: Soft soil stabilisation. EuroSoilStab: Development of design and construction methods to stabilise soft organic soils* (EP 60), Garston: IHS BRE Press, 2002, ISBN 1-86081-599-5.

[16] BUILDING RESEARCH ESTABLISHMENT. *Specifying vibro stone columns (BRE Report BR 391).* Garston: CRC, 2000, ISBN 1-86081-380-1.

[17] INSTITUTION OF CIVIL ENGINEERS. *Specification for ground treatment.* London: ICE, 1987, ISBN 978-0-7277-4943-7.

[18] ASIRI NATIONAL PROJECT, *Recommendations on the design, construction and control of rigid inclusion ground improvements*, Paris: Presses des Pont, 2013, ISBN 978-285978-470-6.

[19] NORBURY, D. *Soil and rock description in engineering practice*. Caithness: Whittles Publishing, 2010, ISBN 978-1904445-65-4.

[20] BOND, A.J. *ReWaRD Reference Manual*. Banstead, Surrey: 1992-2014, available from www.geocentrix.co.uk (last viewed 25/6/15)

[21] BOLTON, M.D. *The strength and dilatancy of sands*. Géotechnique, 1986, Vol. 36(1), pp 65–78.

[22] JAMIOLKOWSKI, M., LADD, C.C., GERMAINE, J.T., and LANCELOTTA, R. *New developments in field and laboratory testing of soils*. Proceedings of the 11th International Conference on Soil Mechanics And Foundation Engineering, San Francisco, 1985, vol. 1, pp 57–153.

[23] SANTAMARINA, J.C, and DIAZ-RODRIGUEZ, J.A. *Friction in soils: micro and macroscale observations*. Pan-American Conference, Boston, 2003.

[24] TERZAGHI, K., PECK, R.B., and MESRI, G. *Soil mechanics in engineering practice* (3rd edition), New York: Wiley-Interscience, 1996, ISBN 0-47108-658-4.

[25] STARK, T.D., CHOI, H., and MCCONE, S. *Drained shear strength parameters for analysis of landslides.* Journal of Geotechnical and Geoenvironmental Engineering, 2005, 131(5), pp 575–588

[26] ATKINSON, J.H. *Non-linear soil stiffness in routine design*. Géotechnique, 2000, Vol. 50(5), pp 487–508.

[27] OZTOPRAK, S., and BOLTON, M.D. *Stiffness of sands through a laboratory database*. Géotechnique, 2013, Volume 63, Issue 1, pp 54–70.

[28] VARDANEGA, P.J., and BOLTON, M.D. *Stiffness of clays and silts: normalizing shear modulus and shear strain*. Journal of Geotechnical and Geoenvironmental Engineering, 2013, 139, pp 1575–1589.

[29] CLAYTON, C.R.I. *Stiffness at small strain: research and practice*. Géotechnique, 2011, Vol. 61(1), pp 5–37.

[30] VIGGIANI, G., and ATKINSON, J.H. *Stiffness of fine-grained soil at very small strains*. Géotechnique, 1995, 45(2), pp 249–265.

[31] CHARLES, J.A., and WATTS, K.S. *Building on fill: geotechnical aspects* (BRE Report 424, 2nd edition), Watford: BRE, 2001. ISBN 1-86081-509-X.

[32] GABA, A.R., SIMPSON, B., POWRIE, W., and BEADMAN, D.R. *Embedded retaining walls - guidance for economic design* (CIRIA C580), London: CIRIA, 2003, ISBN 0-86017-580-4. [15)]

[33] BUILDING RESEARCH ESTABLISHMENT. *Concrete in aggressive ground* (Special Digest 1). Bracknell: BRE Bookshop, 2005, ISBN 1-86081-754-8.

[34] BUILDING RESEARCH ESTABLISHMENT. *Marine borers and methods of preserving timber against their attack* (Technical Note 59). Watford: Princes Risborough Laboratory, 1972.

[35] STRATEGIC FORUM FOR CONSTRUCTION PLANT SAFETY GROUP (2014), *Ground conditions for construction plant*, London: Construction Plant-hire Association.

[36] SKINNER, H. *Working platforms for tracked plant: good practice guide to the design, installation, maintenance and repair of ground-supported working platforms* (BRE Report 470), Watford: BRE, 2004, ISBN 1-86081-700-9.

[37] JEWELL, R.A. *Soil reinforcement with geotextiles* (CIRIA Special Publication SP 123). London: CIRIA, 1996, ISBN 978-0860174257.

[38] BUILDING RESEARCH ESTABLISHMENT. *Low-rise buildings on shrinkable clay soils: Part 1* (BRE Digest 240). London: Construction Research Communications Ltd, 1993, ISBN 0-85125-609-0.

[39] BUILDING RESEARCH ESTABLISHMENT. *Low-rise buildings on shrinkable clay soils: Part 2* (BRE Digest 241). London: Construction Research Communications Ltd, 1990, ISBN 0-85125-377-6.

[40] GREAT BRITAIN. Building Regulations 2010 (SI No. 2214 2010) and subsequent amendments. London: The Stationery Office.

GREAT BRITAIN. Building (Amendment) (Wales) Regulations 2014. London: The Stationery Office.

SCOTLAND. Building (Scotland) Regulations 2004 (SSI No. 2004 406) and subsequent amendments. Edinburgh: The Stationery Office.

GREAT BRITAIN. Building Regulations (Northern Ireland) 2012 (SRNI No. 192 2012) and subsequent amendments. Belfast: The Stationery Office.

GREAT BRITAIN. Building Regulations (Isle of Man) Order 2003. SD 829/03. London: The Stationery Office.

[41] POULOS, H.G., CARTER, J.P., and SMALL, J.C. *Foundations and retaining structures – research and practice*, Proceedings of the 15th International Conference on Soil Mechanics and Foundation Engineering, 2001, Istanbul, Vol. 4, pp 2527–2606.

[42] SALGADO, R., LYAMIN, A.V., SLOAN, S.W., and YU, H.S. *Two- and three-dimensional bearing capacity of foundations in clay*, Géotechnique, 2004, Vol. 54, No. 5, pp 297–306.

[43] REYNOLDS, T.N. *Timber piles and foundations* (BRE Digest 479), Watford: BRE, 2003, ISBN 1-86081-661-4.

[44] DEEP FOUNDATIONS INSTITUTE MICROPILE COMMITTEE. *Guide to drafting a specification for micropiles*. Dallas: DFI, 2004.

[15)] It is anticipated that a revised version of C580 (with a new number, C760) will be published in 2016.

[45] International Society of Micropiles, website: www.ismicropiles.org (last viewed 25/6/15)

[46] ASUC. *Guidelines on safe and efficient underpinning and mini piling operations (3rd edition)*. Bordon: ASUC, 2015, ISBN 978-0-9545370-0-5, available from www.asuc.org.uk (last viewed 25/6/15)

[47] BOND, A.J., and HARRIS, A.J. *Decoding Eurocode 7*, London: Taylor and Francis, 2008, ISBN 978-0-415-40948-3.

[48] BOND, A.J. and SIMPSON, B. *Pile design to Eurocode 7 and the UK National Annex. Part 2: UK National Annex*, London: Emap, Ground Engineering, 2010, vol. 43, no 1, pp 28–31.

[49] BEREZANTZEV, V.G., KHRISTOFOROV, V.S., and GOLUBKOV, V.N. *Load bearing capacity and deformation of piled foundations*. Proceedings of the 5th International Conference of Soil Mechanics And Foundation Engineering, Paris, 1961, Vol. 2, pp 11–15.

[50] FLEMING, W.G.K., WELTMAN, A.J., RANDOLPH, M.F., AND ELSON, W.K. *Piling engineering* (3rd edition), Abingdon: Taylor & Francis, 2009, ISBN 0-415-26646-7.

[51] SALGADO, R. *The engineering of foundations.* New York: McGraw-Hill, 2008, ISBN 978-007-125940-9.

[52] BURLAND, J.B. *Shaft friction on piles in clay: a simple fundamental approach. In Ground Engineering*, 1973, Vol 6, No 3, pp 3–42.

[53] MEYERHOF, G.G. *Bearing capacity and settlement of pile foundations*. Journal of Geotechnical Engineering Division of the American Society of Civil Engineers 1976, 102 (GT3), pp 195–228

[54] WELTMAN, A.J., and HEALY, P.R. *Piling in 'boulder clay' and other glacial tills*. CIRIA Report PG5, London: CIRIA, 1978, ISBN: 978-0-86017-103-4.

[55] LONDON DISTRICT SURVEYORS' ASSOCIATION. *Foundations no. 1: Guidance notes for the design of straight shafted bored piles in London clay*. Bromley, Kent: LDSA, 2009, ISBN 0-9516518-6-2.

[56] BROWN, D.A., TURNER, J.P., and CASTELLI, R.J. *Drilled shafts: construction procedures and LRFD design methods* (FHWA Report No. NHI-10-016). Washington: Federal Higway Administration, 2010.

[57] WHITE, D.J. and BOLTON, M.D. *Comparing CPT and pile base resistance in sand*, Proceedings of the Institution of Civil Engineers Geotechnical Engineering, 158, January 2005, GE1, pp3–14.

[58] LEE, J., SALGADO, R., and PAIK, K. *Estimation of load capacity of pipe piles in sand based on CPT results*. Journal of Geotechnical and Geoenvironmental Engineering, 2003, Vol. 129, No 5, 391–403.

[59] AOKI, N., and VELLOSO, D.A. *An approximate method to estimate the bearing capacity of piles*. Proceedings of the 5th Pan-American Conference on Soil Mechanics and Foundation Engineering, Buenos Aires, 1975, Vol. 1, pp 367–376.

[60] AOKI, N., VELLOSO, D.A., and SALAMONI, J.A. *Fundações para o silo vertical de 100000 t no Porto de Paranaguá*, Proceedings of the 6th Brazilian Conference of Soil Mechanics and Foundation Engineering, 1978, Vol. 3, pp 125–132.

[61] LOPES, F.R., and LAPROVITERA, H. *On the prediction of the bearing capacity of bored piles from dynamic penetration tests. In Deep foundations on bored and auger piles* (ed. Van IMPE, W.), Rotterdam: Balkema, 1988, pp 537–540.

[62] ESLAMI, A. and FELLENIUS, B.H. *Pile capacity by direct CPT and CPTu methods applied to 102 case histories*. Canadian Geotechnical Journal, 1997, Vol. 34, No. 6, 886–904.

[63] LEHANE, B.M. and RANDOLPH, M.F. *Evaluation of a minimum base resistance for driven pipe piles in siliceous san*d. Journal of Geotechnical and Geoenvironmental Engineering, 2002, Vol. 128, No 3, 198-205.

[64] CHOW, F.L. *Investigations into the behaviour of displacement piles for offshore structures*. PhD Theses, University of London (Imperial College), 1997.

[65] LEE, J., and SALGADO, R. *Determination of pile base resistance in sands*. Journal of Geotechnical and Geoenvironmental Engineering, 1999, Vol. 125, No. 8, pp 673-683.

[66] JARDINE, R., CHOW, F., OVERY, R., and STANDING, J. *ICP design methods for driven piles in sands and clays*. London: Thomas Telford Publishing, 2005, ISBN 0-7277-3272-2.

[67] POULOS, H.G. *A practical design approach for piles with negative skin friction*. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, Vol 161, February 2008, Issue GE1, pp 19–27.

[68] SIMPSON, B., MORRISON, P., YASUDA, S., TOWNSEND, B., GAZETAS, G. *State of the art report: Analysis and design,* 2009, pp 2873–2929

[69] WALLACE, W.A., GANNON, J.A., MASTERTON, G.G.T, and MUIR-WOOD, D. *Piled foundations in weak rocks* (CIRIA Research Report R181). London: CIRIA, 1999, ISBN 978-0-8601-7494-3.

[70] TURNER J. *Rock-socketed shafts for highway structure foundations* (NCHRP Synthesis 360). Washington: National Cooperative Highway Research Program, 2006, ISBN 0-309-09768-1.

[71] HORVARTH, R.G., KENNY, T.C., and TROW, W.A. *Results of tests to determine shaft resistance of rock socketed drilled piers*. Proceedings of the International Conference on Structural Foundations on Rock, Sydney, 1980.

[72] ROWE, R.K., ARMITAGE, H.H. *A design method for drilled piers in soft rock*. Canadian Geotechnical Journal, 1987, Vol 13.3, pp 324–333.

[73] KULHAWY, F.H., PHOON, K-K. *Drilled shaft side resistance in clay soil to rock*. In Design and Performance of Deep Foundations: Piles and Piers in Soil and Soft Rock, pp 172–183.

[74] BROMS, B.B. *The lateral resistance of piles in cohesionless soils*. Journal of Soil Mechanics and Foundation Division, ASCE, 1964, 90(SM3), pp123–156.

[75] VIGGIANI, C., MANDOLINI, A., and RUSSO, G. *Piles and pile foundations.* London: Spon Press, 2012, ISBN 978-0-415-49066-5.

[76] BRINCH HANSEN, J. *The ultimate resistance of rigid piles against transversal forces*. Copenhagen: The Danish Geotechnical Institute, Bulletin No. 12, 1961, 16 pp.

[77] TOMLINSON, M., and WOODWARD, J. *Pile design and construction (6th edition)*, Boca Raton: CRC Press, 2015, ISBN 978-1-4665-9263-6.

[78] BROMS, B.B. *The lateral resistance of piles in cohesive soils*. Journal of Soil Mechanics and Foundation Division, ASCE, 1964, 90(SM2), pp 27–63.

[79] POULOS, H.G. and DAVIS, E.H. *Pile foundation analysis and design*, New York: Wiley, 1980, ISBN 978-0471020844.

[80] REESE, L.C., ISENHOWER, W.M., and WANG, S.-T. *Analysis and design of shallow and deep foundations*. New York: Wiley, 2006, ISBN 0-471-43159-1.

[81] FLEMING, W.G.K. *A new method for single pile settlement prediction and analysis*, Géotechnique, 1992, Vol. 42, No. 3, pp 411–425.

[82] RANDOLPH, M.F., and WROTH, C.P. *An analysis of the vertical deformation of pile groups*, Géotechnique, 1979, Vol. 29, No. 4, pp 423–439.

[83] BASILE, F. *Non-linear analysis of pile groups*. Proceedings of the Institution of Civil Engineers, Geotechnical Engineering, 1999, Vol. 137, No. 2, April, pp 105–115.

[84] REESE, L.C., and VAN IMPE, W.F. *Single piles and pile groups under lateral loading* (2nd edition). London: Taylor & Francis, 2011, ISBN 978-0-415-46988-3.

[85] QUILLIN, K., NIXON, P.J., MATTHEWS, J.D., and LONGWORTH, T.I. *Concretes with high ggbs contents for use in hard/firm secant piling*, BRE Information Paper IP 17/05, Watford: BRE, ISBN 1-86081-894-3.

[86] HIGHWAYS ENGLAND, *Design Manual for Roads and Bridges, Volume 2 – Highway Structures Design: (Substructures, Special Structures and Materials), Section 2 – Special Structures, Part 6, BD 12/01, Design of corrugated steel buried structures with spans greater than 0.9 metres and up to 8.0 metres*. London: The Stationery Office.

[87] CHIN, F.V. *Estimation of the ultimate load of piles not carried to failure*. Proceedings of the 2nd Southeast Asian Conference on Soil Engineering, 1970, pp 81–90.

[88] DAVISSON, M.T. *High capacity piles*. Proc. Lecture Series on Innovations in Foundation Construction, Chicago: American Society of Civil Engineers, 1972, March 22, pp 81–112.

[89] BUTLER, H.D., and HOY, H.E. *Users manual for the Texas quick-load method for foundation load testing*. Report FHWA-IP-77-8, Washington: Federal Highways Administration, Office of Development.

[90] ARCELORMITTAL. *Piling handbook* (8th edition, reprint 2008). Luxembourg: ArcelorMittal Commercial RPS S.à.r.l., 2008.

[91] WILLIAMS, J., SIDELL, J., and PANTER, I. *Piling and archaeology – an English Heritage Guidance Note*, 2007, Product Code 51 352.

[92] PERKO, H.A. *Helical piles – a practical guide to design and installation*. Hobokon, New Jersey: John-Wiley, 2009, ISBN 978-0-470-40479-9.

[93] HASLAM, S., and O'CONNOR, L. (with contributions from KITCHING, D. and JONES, P.). *Guidelines on safe and efficient basement construction directly below or near to existing structures*, Bordon: ASUCplus, 2013, ISBN 978-0-9554032-9-3, available from www.asuc.org.uk (last viewed 25/6/15)

[94] THORBURN, S., and LITTLEJOHN, G.S. (editors). *Underpinning and retention* (2nd edition). Glasgow: Blackie Academic & Professional, 1993. ISBN 0-7514-0094-7.

[95] INSTITUTION OF STRUCTURAL ENGINEERS. *Appraisal of existing structures* (3rd edition), London: IStructE, 2010, ISBN 978-1-906335-04-5.

[96] HIGHWAYS AGENCY. *Inspection Manual for Highway Structures – Volume 1: Reference Manual*, London: The Stationery Office, 2007, ISBN 9780115527975.

[97] HIGHWAYS AGENCY. *Inspection Manual for Highway Structures – Volume 2: Inspector's Handbook*, London: The Stationery Office, 2007, ISBN 9780115527982.

[98] HIGHT, D.W., MCMILLAN, F., POWELL, J.J.M, JARDINE, R.J., and ALLENOU, C.P. *Some characteristics of London Clay*. In TAN, T.S, PHOON, K.K., HIGHT, D.W., and LEROUEIL, S. (eds). *Characterisation and engineering properties of natural soils* (Volume 2), Lisse, the Netherlands: Swets & Zeitlinger, 2003, pp 851–908, ISBN 90-5809-539-8.

[99] HIGHT, D.W., GASPARRE, A., NISHIMURA, S., ANH-MINH, N., JARDINE, R.J. and COOP, M.R. *The London clay at T5: an overview of the characterisation study*. Géotechnique, 2007, 57(1), pp 3–18.

[100] FORSTER, A., HOBBS, P.R.N., CRIPPS, A.C., ENTWISLE, D.C., FENWICK, S.M.M., RAINES, M.R., HALLAM, J.R., JONES, L.D., SELF, S.J. AND MEAKIN, J.L. *Engineering geology of British rocks and soils: Gault clay.* British Geological Survey Technical Report, WN/94/31

[101] HIGHT, D.W., ELLISON, R.A., and PAGE, D.P. *Engineering in the Lambeth Group (*CIRIA Report C583*)*. London: CIRIA, 2004. ISBN 0-86017-583-9.

[102] ENTWISLE, D.C, HOBBS, P.R.N., NORTHMORE, K.J., SKIPPER, J., RAINES, M.R., SELF, S.J., ELLISON, R.A., and JONES, L.D. *Engineering geology of British rocks and soils: Lambeth Group*. BGS Report OR/13/006, Keyworth, Nottingham: 2013.

[103] TRENTER, N.A. *Engineering in glacial tills (*CIRIA Report C504*)*. London: CIRIA, 1999. ISBN 0-86017-504-9.

[104] CHANDLER R.J. and FORSTER, A. *Engineering in Mercia mudstone (*CIRIA Report C570*)*. London: CIRIA, 2002. ISBN 978-0-86017-570-4.

[105] HOBBS, P.R.N., HALLAM, J.R., FORSTER, A., ENTWISLE, D.C., JONES, L.D., CRIPPS, A.C., NORTHMORE, K.J., SELF, S.J., AND MEAKIN, J.L. *Engineering geology of British rocks and soils: Mudstones of the Mercia Mudstone Group*. BGS Report RR/01/02, Keyworth, Nottingham: 2002, ISBN 0852724039.

[106] HOBBS, P.R.N., ENTWISLE, D.C., NORTHMORE, K.J., SUMBLER, M.G., JONES, L.G., KEMP, S., BARRON, M. and MEAKIN, J.L. *Engineering Geology of British Rocks and Soils – Lias Group*. British Geological Survey Internal Report OR/12/032, Keyworth, Nottingham: British Geological Survey, 2012.

[107] DEPARTMENT FOR COMMUNITIES and LOCAL GOVERNMENT. *National Planning Policy Framework, 2012*, London, ISBN: 978-1-4098-3413-7, available from http://planningguidance.planningportal.gov.uk (Last viewed 25/6/15).

[108] THE SCOTTISH GOVERNMENT, *Scottish planning policy*, Edinburgh, 2013, ISBN: 978-1-78412-567-7, available from http://www.scotland.gov.uk/Topics/Built-Environment/planning/Policy (Last viewed 25/6/15).

[109] DEPARTMENT FOR COMMUNITIES and LOCAL GOVERNMENT. *Planning practice guidance*, 2014, London, available from http://planningguidance.planningportal.gov.uk/blog/guidance. (Last viewed 25/6/15.)

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