# Efficient design of piled foundations for low-rise housing

## Design guide

NHBC Foundation
Housing research in partnership with BRE Trust

ARUP

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February 2010

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Other assistance

The authors particularly wish to acknowledge the assistance of the following people:

Mr Tim Chapman, Arup
Mr John Jones, NHBC
Professor Brian Simpson, Arup

Assistance was also provided by:

Mr Norman Mure, Stent Foundations
Mr John Patch, Roger Bullivant
Mr Cliff Wren, Roger Bullivant

Photographs

Figures J4 and J5 courtesy of Roger Bullivant


NF21
Published by IHS BRE Press on behalf of the NHBC Foundation
February 2010
ISBN 978-1-84806-106-4

# FOREWORD

As part of the NHBC Foundation's mission to provide the industry with useful and relevant guidance, our latest publication considers piled foundations for low rise housing developments. This guide explores various design approaches and the associated environmental and economic advantages. Money can be saved by adopting a more efficient pile design and environmental benefits gained by reducing the use of natural resources.

The guide discusses the selection and design of piled foundations, with reference to the relevant design codes, standards and guidance which were current at the time of publication. The design of efficient foundations for low rise housing in the UK is also discussed in a more general sense.

A review of current practice was undertaken during the preparation of this guidance and the results used to identify areas where additional guidance would be useful. The guide attempts to address these, in particular in relation to the selection of foundations, site investigation practice, and design and construction methods for piled foundations.

Reflecting increasing public concern about environmental issues, this guide also addresses some of the main considerations. It deals with the environmental impacts of foundations in broad terms, and places some emphasis on the assessment of embodied carbon. Sustainability is also considered in terms of the potential benefits from use of 'geothermal piles'.

The NHBC Foundation's work helps promote good practice within the housebuilding industry and this guide is aimed at housebuilders, consultants, piling contractors and building control bodies. I hope you find this guide of use – I believe it provides a valuable resource in terms of the drive for efficiency and sustainability within the housebuilding industry.

**Rt. Hon. Nick Raynsford MP**

Chairman, NHBC Foundation

# ABOUT THE NHBC FOUNDATION

The NHBC Foundation was established in 2006 by the NHBC in partnership with the BRE Trust. Its purpose is to deliver high-quality research and practical guidance to help the industry meet its considerable challenges.

Since its inception, the NHBC Foundation's work has focused primarily on the sustainability agenda and the challenges of the government's 2016 zero carbon homes target. Research has included a review of microgeneration and renewable energy techniques and the groundbreaking research on zero carbon and what it means to homeowners and housebuilders.

The NHBC Foundation is also involved in a programme of positive engagement with government, development agencies, academics and other key stakeholders, focusing on current and pressing issues relevant to the industry.

Further details on the latest output from the NHBC Foundation can be found at www.nhbcfoundation.org.

## NHBC Foundation Advisory Board

The work of the NHBC Foundation is guided by the NHBC Foundation Advisory Board, which comprises:

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# EXECUTIVE SUMMARY

This guide has been prepared to provide housing sector guidance for the use of piled foundations for low-rise housing developments.

It is primarily intended to promote efficient design of piled foundations, and discusses specific guidance for the selection and efficient design of piled foundations, with reference to the relevant design codes, standards, and guidance that were current at the time of publication. The design of efficient foundations for low-rise housing in the UK has also been discussed in a more general sense.

A review of current practice was undertaken during the preparation of this guide and the results of the review were used to identify areas where additional guidance would be useful. The guide attempts to address these areas, in particular in relation to the selection of foundations, site investigation practice, and design and construction methods for piled foundations.

Reflecting the increasing emphasis on environmental issues, this guide also addresses some of the main considerations in this regard. It deals with the environmental impacts of foundations in broad terms, and places some emphasis on the assessment of embodied carbon. Sustainability is also considered in terms of the potential benefits from use of 'geothermal piles'.

# 1 Introduction

## 1.1 General

This guide has been prepared to provide housing sector guidance for the use of piled foundations for low-rise housing developments.

'Low-rise housing' is defined in BS 8103-1, *Structural Design of Low Rise Buildings*[1] as:

> "detached, semi-detached and terraced houses and flats (with not more than four self contained dwelling units per floor accessible from one staircase), or not more than three storeys above ground intended for domestic occupation and of traditional masonry construction with timber roof and floors of timber or concrete."

For the purposes of this guide, low-rise housing is defined as housing of four storeys or less, and is not limited to traditional masonry forms of construction.

Although a wide variety of forms of construction are available, those used for low-rise housing in the UK have traditionally been limited to either unreinforced masonry construction or less commonly concrete and steel-framed buildings, which are generally constructed with masonry infill panels. More recently, timber-framed buildings have become more common on account of their ease of construction, reduced construction times, and improved sustainability credentials. The masonry cladding to timber-framed structures is usually supported on the same foundations as the framed structure, and in terms of the tolerances to building movements, it is the cracking of the masonry and brittle finishes that are likely to be critical.

## 1.2 What is meant by 'efficient design'?

Increased efficiency can be considered in terms of direct reductions in foundation costs, related to the amount of resources/materials used, and also in terms of reductions in indirect costs, eg by preventing foundation failures requiring remedial works, or by reducing waste generated by excavations. The reduction of carbon emissions and embodied energy can also be considered as a measure of increasing 'efficiency'.

In terms of instances where 'efficiency' of design could have been improved, three broad situations can be considered:

A: where foundations have not met the design requirements, typically resulting in the need for remedial works.

B: where the type of foundation selected does not provide the best solution.

C: where foundations have performed adequately, but have been 'overdesigned'.

These situations can be developed further as follows:

**Case A:** Where foundation 'failure' occurs it is often related to sites where there has been a fundamental misunderstanding of the ground conditions and hence the manner in which the foundations will interact with them, for example where features or conditions on site have not been recognised (eg loose made ground in areas of backfilled ponds or swelling of clays following tree removal). A suitable site investigation including desk study and subsequent ground investigation is essential to identify and quantify hazardous ground conditions, to allow mitigation against the risks that these hazards pose, and inform the design.

Foundation failure could also relate to cases where the performance requirements of the building have not been considered and designed for, for example, where foundation movements are in excess of acceptable building movements.

**Case B:** Where a foundation type other than that which has been selected could have provided overall benefits and savings, for example where deep trench fill foundations have been used where other solutions such as rafts, piles or ground improvement may have been better solutions.

**Case C:** Where the ground may be well understood in terms of the stratigraphy, there may be a shortfall of information to allow 'efficient' design of the foundations. This could for example relate to inadequate strength data for the ground, with the foundation design becoming less efficient since the design assumptions made are more conservative than actually needed.

## 1.3 Content and objectives

This guide discusses the requirements of foundations for low-rise housing in the UK; it identifies areas of key concern and presents broad guidance on choice of foundation types. It draws on current codes, standards, and guidance and presents this in the context of the requirements of the UK low-rise housing market. For piled foundations it presents specific guidance for their selection and efficient design.

Efficient foundation design requires an understanding of the performance criteria of the building, such that design loads and allowable settlements are clearly defined. The performance criteria associated with low-rise housing are discussed in section 2.

Foundations must be selected and designed to ensure that the performance criteria of the building are met, and that consideration is given to the various issues discussed in section 3. It is critical that an adequate site investigation is undertaken to provide information on the ground conditions and associated ground hazards to inform this selection process. Ground investigation is discussed in detail in section 4. Recommendations for pile design and construction are given in section 5, including reference to design codes and best practice guidance. The environmental impact of foundation solutions is discussed in section 6, with particular reference to piled foundations, including consideration of embodied carbon and use of geothermal piles.

It is intended that the advice given in this guide is concise, practical, and accessible, and that it provides an introduction to the main issues to be considered. Further details to illustrate or expand on the information discussed in the main body of the text are provided in a series of appendices.

The guide should be read in conjunction with the appropriate British and European standards and health and safety regulations and guidance documents, to which reference is made throughout the guide. The design codes, legislative instruments, and associated guidance documents referred to are those that were in use at the time of publication. This guide is not a substitute for any part of the current codes and regulations.

The guide provides information with regard to the subject matter covered. The publisher, editor, and authors do not accept any responsibility for the contents or any loss or damage that might occur as a result of following or using data or advice given in this publication.

Appropriate ground engineering and geotechnical design requires a detailed knowledge of basic principles of soil mechanics and engineering judgement based on experience of ground conditions and ground risk. Professional advice should always be sought.

## 1.4 Intended readership

It is intended that the following will find this guide helpful:

- housebuilders and developers
- consultants
- piling contractors
- warranty providers
- building control bodies
- other professional advisors to housebuilders and developers.

## 1.5 Review of current practice

During the preparation of this guide, questionnaire surveys were conducted to gain knowledge relating to current practice and to identify areas of general concern in relation to the selection of foundation types, ground investigation practice, and design and construction methodologies adopted for foundations. The questionnaire surveys were prepared using web-based survey software and distributed to a wide variety of industry professionals, including a cross section of piling contractors, consultants, housebuilders, property developers, ground investigation contractors, building control engineers and regulators.

The surveys were undertaken to understand where additional guidance may be useful, and key issues identified are referred to in relevant sections of this document. A brief review of the surveys and the main conclusions is also presented in Appendix A.

# 2 Performance of low-rise housing foundations

## 2.1 Housing foundation requirements

Details of the requirements of the Building Regulations are discussed in Appendix B.

Approved Document A of the Building Regulations,[2] requires the foundations to be designed to transmit the building loads into the ground safely, and without causing movements of the structure or adjacent ground that may impair the stability of any part of an adjacent building.

In addition to the requirements of the Building Regulations, which are concerned primarily with health and safety, the foundation design must ensure that the movements associated with both the loading effects of the house (which can lead to settlement) and external effects associated with the underlying ground (which can lead to subsidence or heave) are within acceptable or tolerable levels.

Atkinson[3] presents definitions of settlement and subsidence as follows:

**Settlement:** Movement in the structure caused by the weight of the building compressing the underlying ground.

**Subsidence (and Heave)**: Movement in the structure caused by the loss of ground support below the foundation or as a result of volumetric changes, either expansion or shrinkage, in the founding materials.

Movements must be sufficiently controlled to ensure that they do not lead to unacceptable levels of distortion and cracking in the structure and associated services connections that may cause problems with the everyday use of the building.

## 2.2 Measurement of performance

There have been a number of published studies that have considered how to measure the performance of buildings, including low-rise housing, in terms of levels of damage.

To help reduce subjectivity in damage assessment, Burland and Wroth[4] developed damage criteria, with categories of damage determined on the basis of relative ease of repair, divided into aesthetic, serviceability and structural categories. These criteria allow the diagnosis of damage following the measurement of cumulative crack widths and other observations, such as sticking of windows and doors, water-tightness and distortions in walls. These damage criteria, which are included within BR 251, *Assessment of Damage in Low Rise Buildings*,[5] are reproduced within Table C1 in Appendix C of this guide, and summarised in Table 1.

Table 1

BRE damage categories and classifications

| Category of damage | Broad classification |
|---|---|
| 0–1 | Aesthetic |
| 2–3 | Serviceability |
| 4–5 | Structural |

Although general criteria such as these are useful tools, it should be recognised that what constitutes 'acceptable' levels of damage is very subjective.

From the householder's perspective, the lower levels of damage that may lead to aesthetic cracking may cause concern out of proportion to the significance of the damage. Visible cracking of finishes will usually occur long before the serviceability of the house is compromised or the structural integrity of the building is impaired.

## 2.3 Causes of damage

Movements and cracking in the housing superstructure can be caused by a number of factors that are not linked to the adequacy of the building's foundations. These other factors include, but are not limited to, frost attack, thermal expansion and contraction of building materials, drying shrinkage and chemical attack of construction materials, poor design detailing and workmanship, and the weakening of construction materials with time, often due to lack of maintenance. These factors generally result in levels of damage limited to the aesthetic categories as defined by the BRE classification, and at these lower levels of damage it can be difficult to distinguish damage resulting from these factors from those associated with movements in the underlying ground.

Useful discussions of these common causes of damage can be found in *Subsidence of Low Rise Buildings* by the Institution of Structural Engineers,[6] Driscoll and Crilly,[7] and Driscoll and Skinner.[8]

Although aesthetic damage cannot usually be attributed to foundation movements, it is recognised that damage in the higher categories of serviceability and structural damage is commonly caused by movements of the foundations.

An example of structural damage resulting from foundation movements is presented in Figure 1.

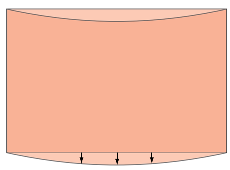
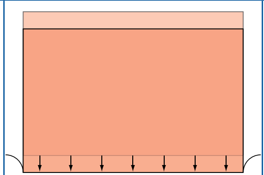
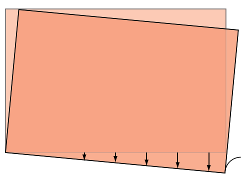
**Figure 1** Example of structural damage of masonry caused by foundation movements.

## 2.4 Acceptable limits for foundation movement

Although total settlement and tilt settlement can be critical considerations, damage to buildings is generally due to differential movement (Box 1). As well as vertical displacements of one part a structure relative to another, horizontal strains can be developed and contribute to damage of buildings. Both vertical and horizontal strains can result from ground movements imposed on the building from ground subsidence or heave.

If the entire foundation moves by an equal amount relative to the adjacent ground, the structure itself is unlikely to experience damage; however, there may be impacts on the overall performance, for example the services entering and leaving the building will at a certain point become strained or fractured.

**Box 1 Differential, tilt and total settlements**

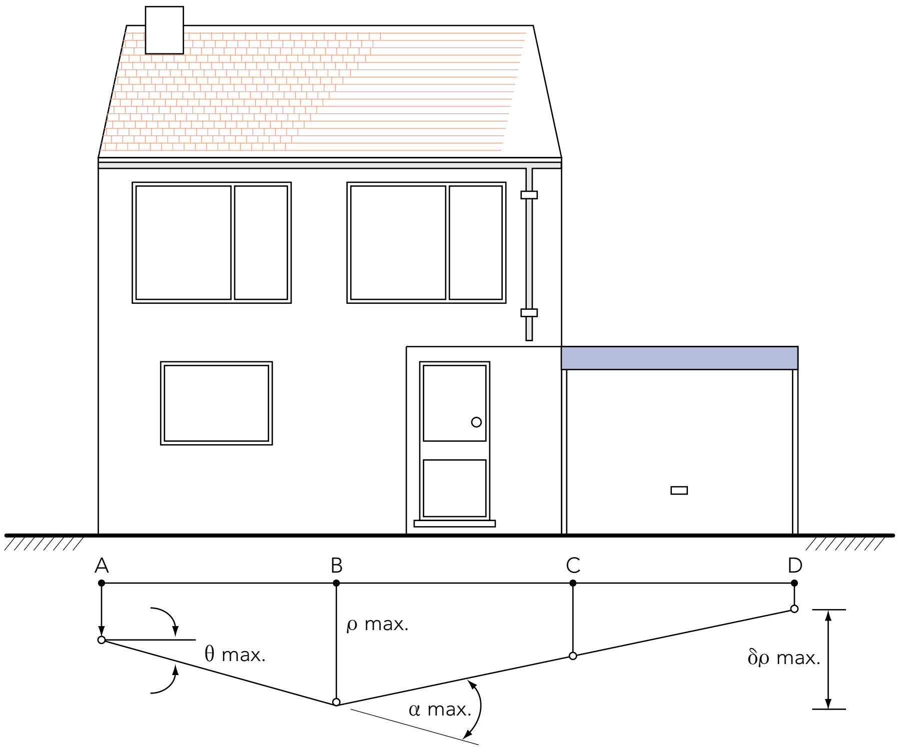
| | | |
|---|---|---|
| Differential settlement (sagging mode shown) |  | Differential settlements are generally of greatest concern in relation to potential cracking and structural damage of low rise buildings. Tolerances to differential movements can be very small due to the brittle masonry and surface finishes generally used which cannot readily tolerate significant strains |
| Total settlement |  | Where the entire structure settles by a uniform amount, large settlements can be accommodated without significant damage to the structure itself. There is, however, potential for damage to services connections as a result of the settlement of structures relative to the adjacent ground or vice versa |
| Tilt settlement |  | If a structure deforms in pure tilt, with no angular distortion, cracking will often not occur even with relatively large displacements. Such 'pure tilt' of low rise housing is likely to become of concern to householders long before the building reaches a dangerous condition. There may also be issues associated with services connections as with total settlement |

There have been a number of published studies that have aimed to understand and quantify the acceptable movements that can be accommodated by structures. These have been based on either a review of published literature and case studies or the application of more theoretical approaches to assist in understanding of the fundamental behaviour of structures, including those constructed of load-bearing masonry, and those built using steel or concrete frames. A discussion of this research is presented in Appendix C.

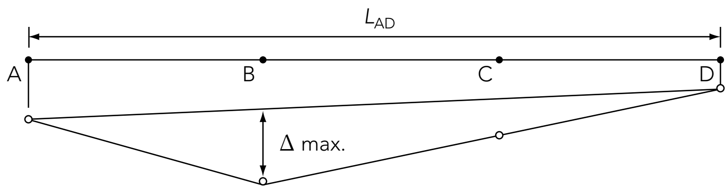
### 2.4.1 Tolerance to differential settlements

Differential settlements are usually defined in terms of either 'deflection ratio' or 'angular distortion'. Definitions of these parameters are illustrated in Figure 2.

(a) Definitions of settlement $\rho$, relative settlement $\delta\rho$, rotation $\theta$, and angular strain $\alpha$




(b) Definitions of relative deflection $\Delta$, and deflection ration $\Delta/L$




(c) Definitions of tilt $\omega$, and relative rotation (angular distortion) $\beta$

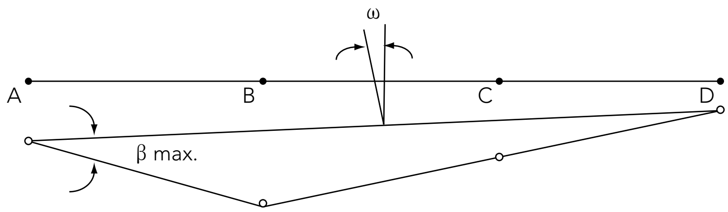



**Figure 2** Definitions of foundation movement (after Burland, Broms, and de Mello).[9]

While different construction materials and housing layouts will have different tolerances to movement, in general the most critical consideration for both load-bearing masonry construction and framed construction is cracking of either the brittle plaster finishes or the masonry itself under tensile stress. For load-bearing walls, either shear strain or bending will be critical, depending on the aspect ratio of the walls and the construction type. For framed buildings the critical failure mechanism will generally be shear failure of the brittle masonry infill panels. The onset of cracking occurs when the 'limiting tensile strain' values of the construction materials have been reached.

A summary of the various acceptable differential movement criteria that have been proposed for both load-bearing masonry and framed buildings, in terms of either angular distortion or deflection ratio, is presented in Table C2 in Appendix C.

For unreinforced masonry construction, hogging type failures (where the edges of the structure settle more than the parts in between) are more critical. A number of the studies discussed in Appendix C have concluded that hogging-related damage will generally occur at half the differential settlement that leads to similar levels of damage under sagging movements. The reason given for this is that greater tensile restraint is generally mobilised at the base of a masonry panel from the foundation it sits on than the restraint developed at the top of the wall from the roof structure.

For the case of unreinforced masonry walls, tolerances based on the onset of visible cracking have been determined by a number of studies, which generally relate to onset of aesthetic damage categories in the BRE classification system. Deflection ratios proposed by Burland and Wroth, which are discussed in Appendix C, have been recommended as limiting values of movement for design purposes in Construction Industry Research and Information Association (CIRIA) Technical Note 107.[10] These include limits of deflection ratios of between 1:2500 in the case of sagging and 1:5000 in the case of hogging. This is illustrated in Figure 4.

These results emphasise that unreinforced masonry construction is very susceptible to aesthetic damage as a result of differential movements (Fig. 4). On the basis of this published guidance, limits of deflection ratio/angular distortion are proposed to prevent aesthetic cracking, which if applied to foundation design would be very difficult to achieve in practice.

**Figure 3** Example of damage, underpinning works are currently underway to prevent further movement occurring.

Larger deflection ratios will be tolerable however where the controlling criterion is to limit the cracking to prevent serviceability-related categories of Damage Category 2 or greater from occurring.

BS EN 1997-1[11] recommends that, when considering serviceability limit states, unless specific limits have been derived for a particular structure, the acceptable angular distortions may be taken as 1/500 where sagging is the critical mode of failure, and 1/1000 where hogging is likely to be critical.

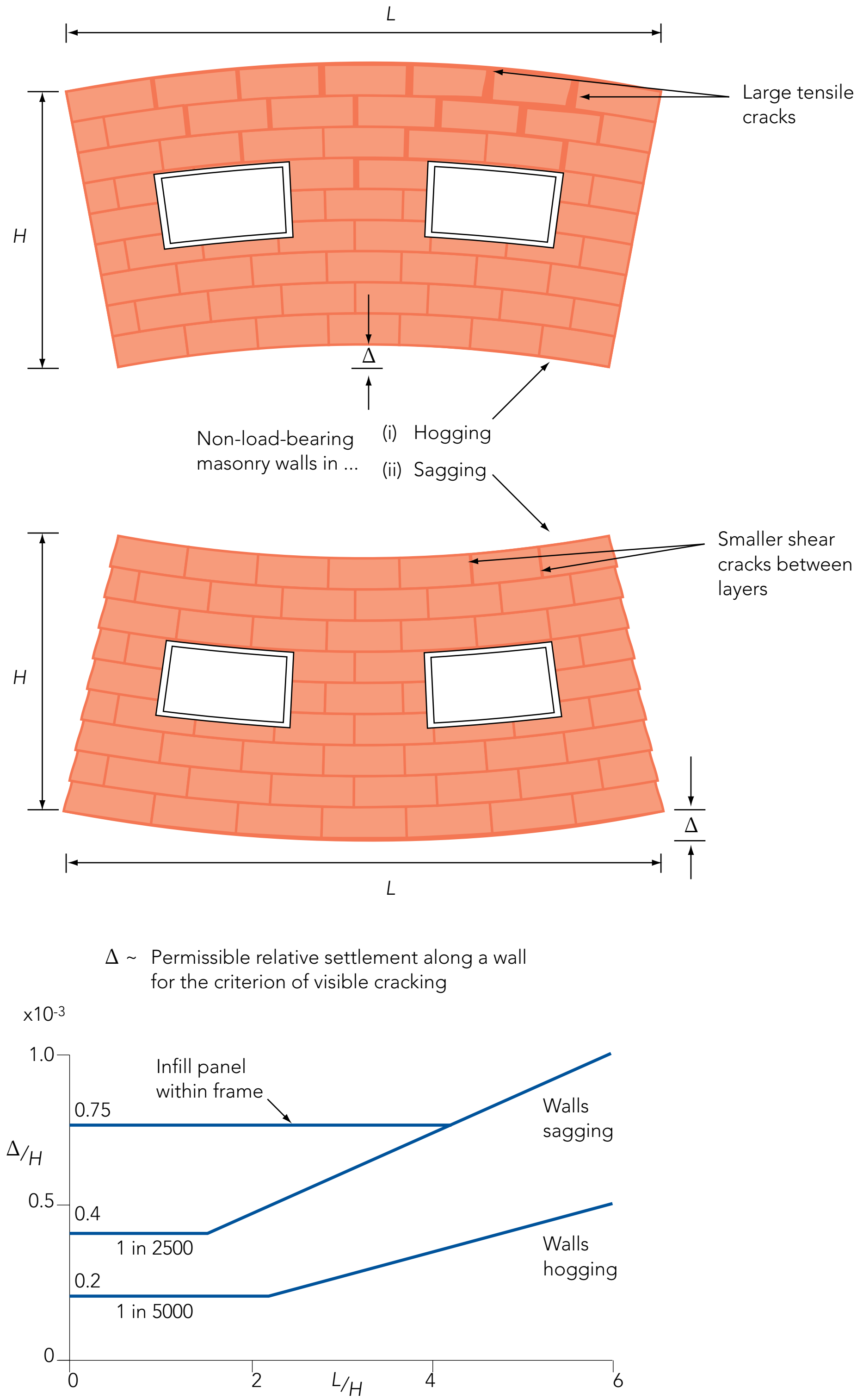



**Figure 4** Limits of deflection ratio for onset of visible cracking in unreinforced masonry walls.[10]

### 2.4.2 Tolerance to total settlements

Although traditional low-rise housing has been shown to be sensitive to very small differential settlements, total settlements are less critical, and large settlements can be acceptable structurally if all of the support positions settle by the same amount. However, as the total settlements of a building increase, so too does the potential for damage to the service connections and impairment of drainage serving the property, eg where drainage runs are designed with minimal falls and subsequent settlement of the structure reduces the falls below acceptable levels.

The potential for large movements of adjacent ground relative to the house should also be considered, eg if ground levels have been raised over soft compressible materials and houses are supported on piles, the surrounding ground is likely to settle relative to the house, such as the example shown on Figure 5.

Where there is relative movement between the building and the surrounding ground, the use of more flexible services and ductile materials may be needed. More brittle services, eg those associated with drainage, can be dealt with by use of flexible joints, for example in the form of a rocker pipe.

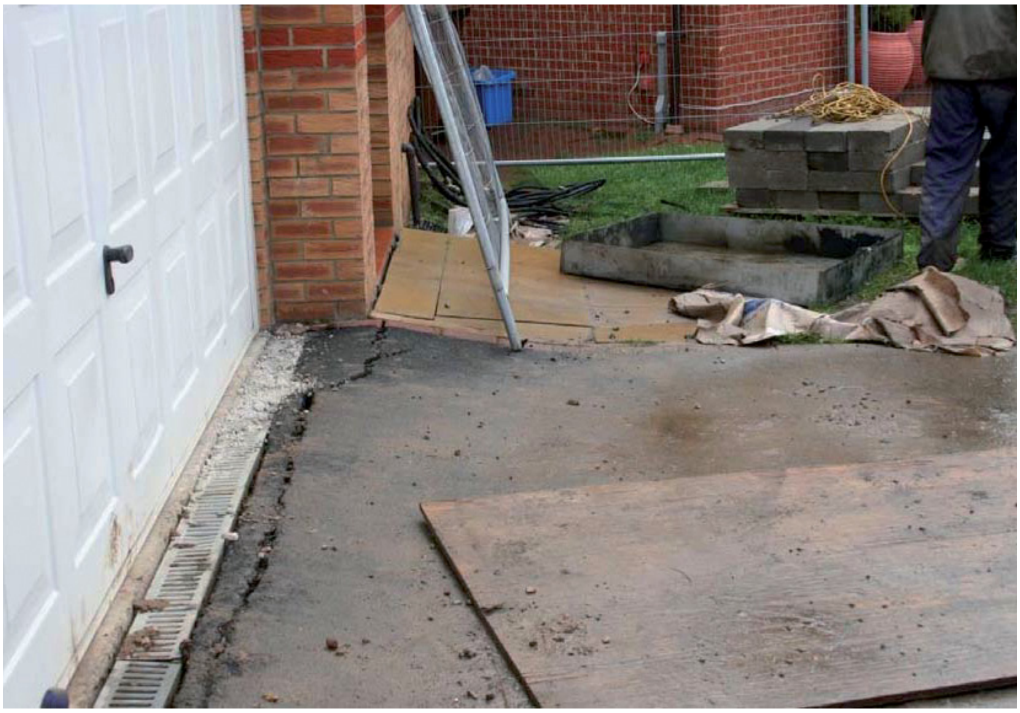


**Figure 5** Following the raising of the site levels, the ground has settled by up to 200 mm relative to the structure, causing problems with access to the houses, and with drainage and other service connections.

### 2.4.3 Tolerance to tilt settlements

Although building foundation design needs to consider the risk of unacceptable tilt settlements, this is more of an issue for buildings that are founded on raft type foundations, and houses supported on piles will not normally be susceptible to these kinds of movements.

BRE Digest 475[12] provides indicative acceptable values for tilting of low-rise housing, and concludes that, although dependent on the layout of the building and the perception of the occupiers, tilt generally becomes noticeable at ratios of between 1/200 and 1/250. Ultimate limits of tilt in the region of 1/50 are given as the point at which the building may be regarded as being in a dangerous condition. A design limit value of 1/400 is proposed by the BRE guidance.

## 2.5 Design to accommodate movements

The very low tolerances to differential movements for typical UK low-rise housing are a result of the form of construction and materials used. Unreinforced masonry construction, particularly where higher strength mortar is used, is especially intolerant to differential movements, and cracking can occur even with very small movements.

Consideration can be given to reducing the sensitivity of the structure to relative movements by design detailing and specification of materials that can accommodate greater relative movements.

The UK code of practice for use of masonry, BS 5628-3[13] provides guidance on the design and specification of materials and components for masonry walls.

The tolerances to differential movements can be increased for instance by the use of less brittle materials, such as softer mortars and less brittle finishes, which will not crack until greater movements have occurred.

Details of masonry mortars of different designations and compressive strengths that can be specified are provided in BS 5628-3 and are presented in terms of their relative ability to accommodate differential movement. Mortar generally becomes softer with lower cement or higher lime contents, allowing correspondingly greater levels of movement to be accommodated. Stronger mortars will be more brittle, accommodate less movement, and have a lower tolerance to movement.

The use of bed reinforcement in masonry walls, at the base and top of the wall, can allow better distribution of the stresses and strains associated with differential movements, particularly where abrupt differential movements are possible, and at weaknesses such as window and door openings.[14] BS 5628-3 discusses ways in which reinforcement can be used in masonry walls to minimise cracking in this way.

BS 5628-3 also provides examples of details which allow provision for differential movements by inclusion of compressible horizontal joints, in both load-bearing masonry and framed structures.

Other guidance for design to minimise the effects of differential movements is provided in CIRIA Technical Note 107.[10]

## 2.6 Summary

If efficient foundation designs are to be developed, an understanding of the acceptable levels of movement of the structures that they support is required.

Although it is important that total movements, tilt movements and differential movements are all given consideration, it is the differential movements, where parts of the structure move relative to others, that are likely to be the most critical.

Attempts to define acceptable movements have generally been made in terms of differential movements. Conclusions of a number of published studies are discussed in Appendix C.

There are a number of factors which can influence the acceptable limits of differential movements for low-rise housing, including:

- the form of construction
- construction materials used
- housing plan layouts
- number of storeys/aspect ratio.

Although the acceptable movements of a given house will be dependent on the particular superstructure design and materials used, it has been concluded that in the UK it is generally the brittle behaviour of masonry that will be the most critical consideration, not only for houses of traditional load-bearing masonry construction, but also those with concrete, steel, or timber frames where masonry panels are typically used.

Categories of damage have been determined on the basis of relative ease of repair, divided into aesthetic, serviceability and structural categories. For house foundation design, movements should be controlled to ensure that serviceability of the building is not impaired.

A number of different criteria have been proposed for acceptable differential movements, as summarised in section 2.4.1. In terms of serviceability limits, BS EN 1997-1 (Eurocode 7) recommends that in the absence of a specific assessment for a given building structure, 'angular distortions' up to 1/500 are likely to be acceptable in cases of 'sagging', and 1/1000 in cases of 'hogging'.

As discussed in section 2.5 with reference to some of the guidance available, the susceptibility to damage as a result of differential movements can be reduced by careful design detailing, including the use of less brittle materials, the incorporation of movement joints, and the reinforcement of masonry. Such measures should be considered as a means of reducing the risks of cracking and serviceability-related issues for low-rise houses.

Although total settlements are generally less critical in terms of potential for damage to the building, damage can occur to services connections and drainage associated with the building where there is large relative movement between the building and the surrounding ground, and this should be considered in the design.

In relation to tilt settlement, current BRE guidance[12] recommends that tilt should be limited to 1:400 for low-rise buildings in order to ensure that serviceability-related issues do not occur.

# 3 Choice of foundations

## 3.1 Options available

Low-rise houses are typically founded on footings at shallow depth, unless ground conditions are unfavourable or there are ground hazards present. The UK government's commitment to the redevelopment of 'brownfield' land and its targets for volumes of new housing, together with the relatively high value of property and land prices, has led in recent years to increasing proportions of developments on more 'marginal' sites. Such sites are more likely to have associated ground hazards, and require alternative foundations to meet the design requirements discussed in section 2 of this guide.

There are a number of options that can be considered for house foundations on more difficult ground (Fig. 6). These include:

- deep trenchfill
- deep pads and ground beams
- rafts
- piles and ground beams.

Subject to any limitations that may be imposed by the warranty provider, improvement of the ground can also be considered by a variety of techniques including:

- pre-loading/surcharging
- dynamic compaction
- vibro-replacement – ie using stone columns
- vibro-compaction
- grouting
- lime or cement stabilisation.

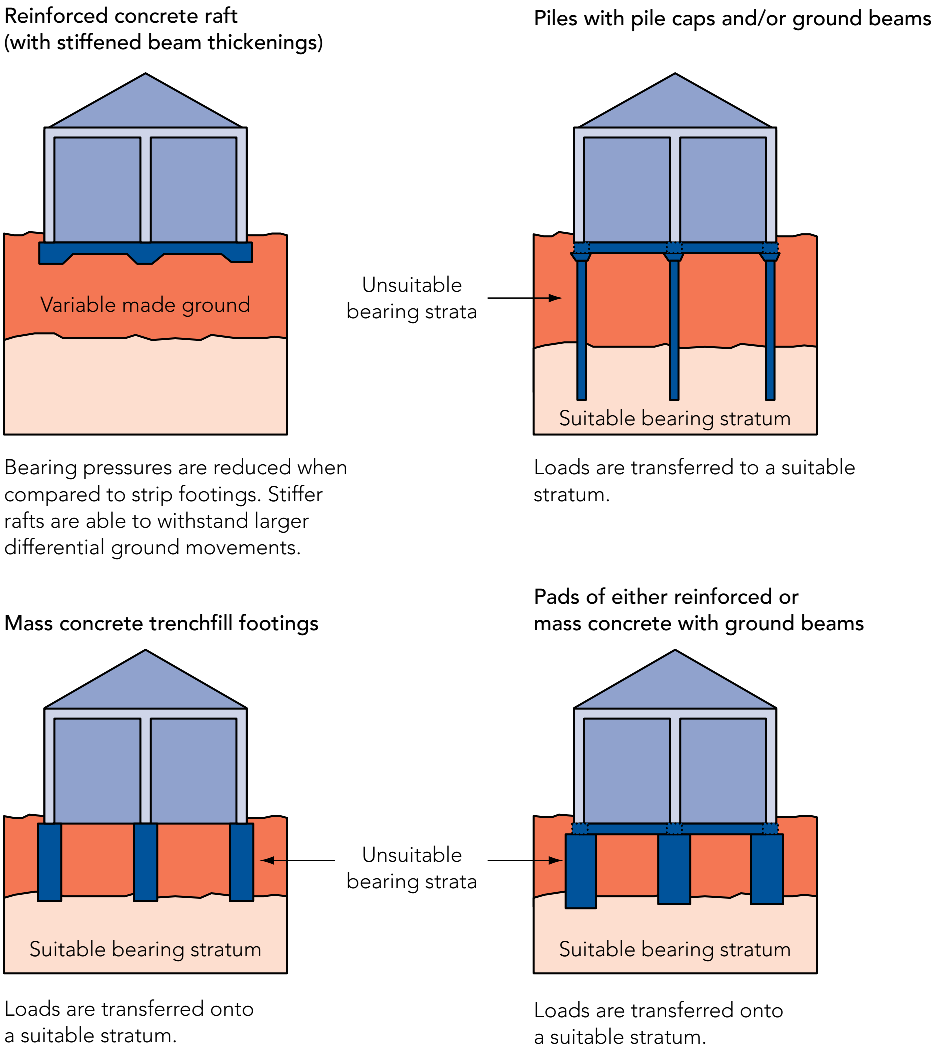



**Figure 6** Options for house foundations on more difficult ground.

A site-specific assessment will be needed to determine the best and most effective foundation solution. There are a wide variety of different factors to consider when selecting the best solution for any given site. Ground conditions will influence the foundation solution selected, as will site logistics, costs of materials, transportation and construction, programming issues and health and safety and environmental considerations.

## 3.2 Ground conditions and ground hazards

The understanding of the ground conditions at the site is key to arriving at the most efficient foundation solution. The importance of the site investigation in managing ground risk is discussed in section 4. Commonly encountered ground-related hazards in the UK are briefly discussed below under headings of settlement, subsidence/heave and ground chemistry.

### 3.2.1 Ground hazards related to settlement

#### Soft ground

Excessive consolidation settlements can occur from loading of soft compressible soil even under the relatively light loads associated with low-rise housing. Where the thickness of soft compressible material is variable across the footprint of the building this can also lead to unacceptable differential or tilt settlements.

Settlements of soft ground from the upfilling of sites should be considered in the choice and design of house foundations, and often can have a major impact. Differential settlements that can occur between the houses and adjacent landscaping and hardstanding areas should also be considered.

### Made ground – filled site

Made ground is commonly placed in an uncontrolled manner and will often be variable in terms of its composition and consistency. Loads imposed by the building or by placement of additional fill can again lead to unacceptable total and differential settlements both in relation to the foundation design and in the potential settlements of adjacent areas of landscaping and hardstanding relative to the houses.

### Down-drag

Where sites underlain by soft ground or made ground are upfilled, and where the building loads are transmitted to an underlying firm stratum, the settlement of the soft or loose materials can result in the development of down-drag loads (negative skin friction) on the foundations. This most commonly applies to piled foundations.

### Former buildings or structures

The presence of buried foundations and other structures can form hard spots beneath the foundations, resulting in the potential for abrupt differential movements. Obstructions can also lead to problems with foundation construction, eg during the installation of piles.

## 3.2.2 Ground hazards related to subsidence/heave

### Shrinking/swelling clays

Where sites are underlain by clays with high shrinkage and swelling potential, changes in moisture content as a result of seasonal variations, changes in the vegetation cover, or leaking pipes or drains can result in changes of volume and subsequent ground movements. Both vertical and lateral movements of the foundations can occur as a result of such volumetric changes, with damage occurring where movements exceed those tolerable by the supported structure.

Movements are typically limited to materials within 2 or 3 m of the ground surface, although the affected depth can be greater.

Particular problems have occurred where shallow strip or trench-fill foundations have been used, but also where piles and ground beams have not been designed to accommodate the volumetric changes and resist the forces imposed as a result of swelling and shrinkage.

Foundations should be constructed to prevent damaging movements caused by shrinkage and swelling.

Although the heavy clays which are susceptible to shrinkage and swelling are most prevalent in the south-east of the UK, they also occur locally elsewhere.

Further guidance for construction of low-rise houses on shrinkable clay soils is presented in BRE Digests 240 and 241,[15,16] and good practice guidance for construction is presented by Driscoll and Skinner.[8]

### High and fluctuating groundwater

A fluctuating groundwater table can lead to changes in the effective stress conditions in the foundation soil, which may lead to soil movements and reduced bearing capacity beneath foundations. Fluctuating or high groundwater conditions can also lead to problems during construction, including excessive groundwater inflow when excavating for shallow foundations or boring for piled foundations.

### Mining and underground cavities

The presence of sub-surface cavities can significantly influence foundation selection. The main risk associated with cavities is related to collapse-induced subsidence beneath or adjacent to foundations. Dependent on the size of the cavity and magnitude of the collapse, foundation damage could vary from minor subsidence to total foundation failure. Sub-surface features include both man-made cavities (such as old mine workings) (Fig. 7) and natural solution cavities within certain geological strata (such as limestone which is susceptible to dissolution).

**Figure 7** Subsidence associated with collapse of mining-related voids.

### Collapsible deposits

Certain deposits can exhibit collapse movements under changes in loading or groundwater conditions. This behaviour can occur in both natural and filled ground and can represent a significant risk to foundations.

### Sloping or unstable ground subject to landslip

Slope movements due to slip (Fig. 8) or creep can occur on clay sites with even very shallow slopes. The NHBC Standards[17] require that slopes with gradients greater than 1:10 are assessed, as even such shallow slopes can be susceptible to slope movements which could impact on the integrity of the foundations.

Where sloping ground is present, the potential for slope movements, including both land-sliding processes and creep, should be assessed.

**Figure 8** Damage due to landslip.

### 3.2.3 Ground hazards related to ground chemistry

#### Aggressive ground conditions

Substances may be present in soil or groundwater which can lead to degradation of concrete or steel foundations, and affect the long-term performance of the foundations. High levels of sulfates when combined with low pH can be particularly aggressive to concrete, and routine testing for these should be undertaken. Guidance on testing and design of appropriate protection measures for buried concrete is presented in BRE Special Digest 1.[18] Guidance relating to steel piles is presented in the Steel Construction Institute's steel bearing piles guide.[19]

#### Contamination

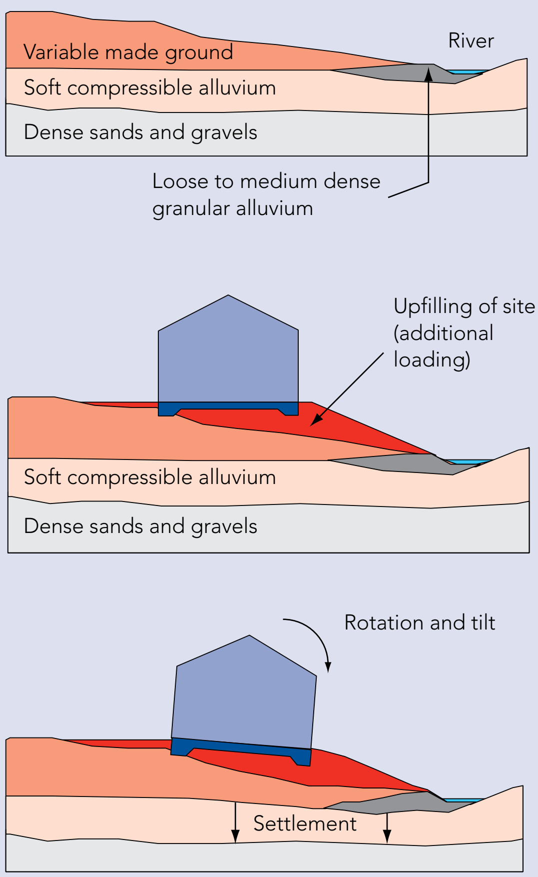
Foundation selection in contaminated ground requires assessment of several issues. The nature and characteristics of the contamination should be well understood to allow selection and design of foundation types.

Careful assessment of the potential impacts relating to foundation installation is required, in particular to avoid creation of pathways for contamination in solid liquid or gaseous phases that could allow the migration of these contaminants either to the surface or to underlying groundwater. *Piling and Penetrative Ground Improvement Methods on Land Affected by Contamination*[20] identifies the risks associated with different piling methods and presents guidance for the management of this risk. Further details of pollution control for foundation works is presented in section 6.

## 3.3 Examples of foundation problems – what can go wrong?

Figures 9 to 12 illustrate examples of what can go wrong with foundations.

**Tilt settlement following upfill of sloping site underlain by soft compressible materials**




For a proposed development of six properties, an investigation comprising 15 trial pits to 3 m depth was undertaken. Loose to medium dense granular made ground was identified. While the base of the made ground was not proved in the majority of the pits, shallow trial pits closer to the river recorded granular alluvium underlying the made ground

The sloping site was filled prior to construction of the houses, with levels raised by up to 3 m locally. Due to the presence of made ground with variable density, a raft foundation was adopted to control differential settlements. The soft alluvial deposits present beneath the site were not identified

The upfilling of the site caused consolidation settlement of the soft compressible alluvium. Following construction of the houses this consolidation settlement was still ongoing. The settlements were greater beneath the areas where most fill had been placed, ie under the greater applied load, leading to excessive tilt settlements in a number of the properties

**Figure 9** Failure of raft foundations as a result of excessive tilt.

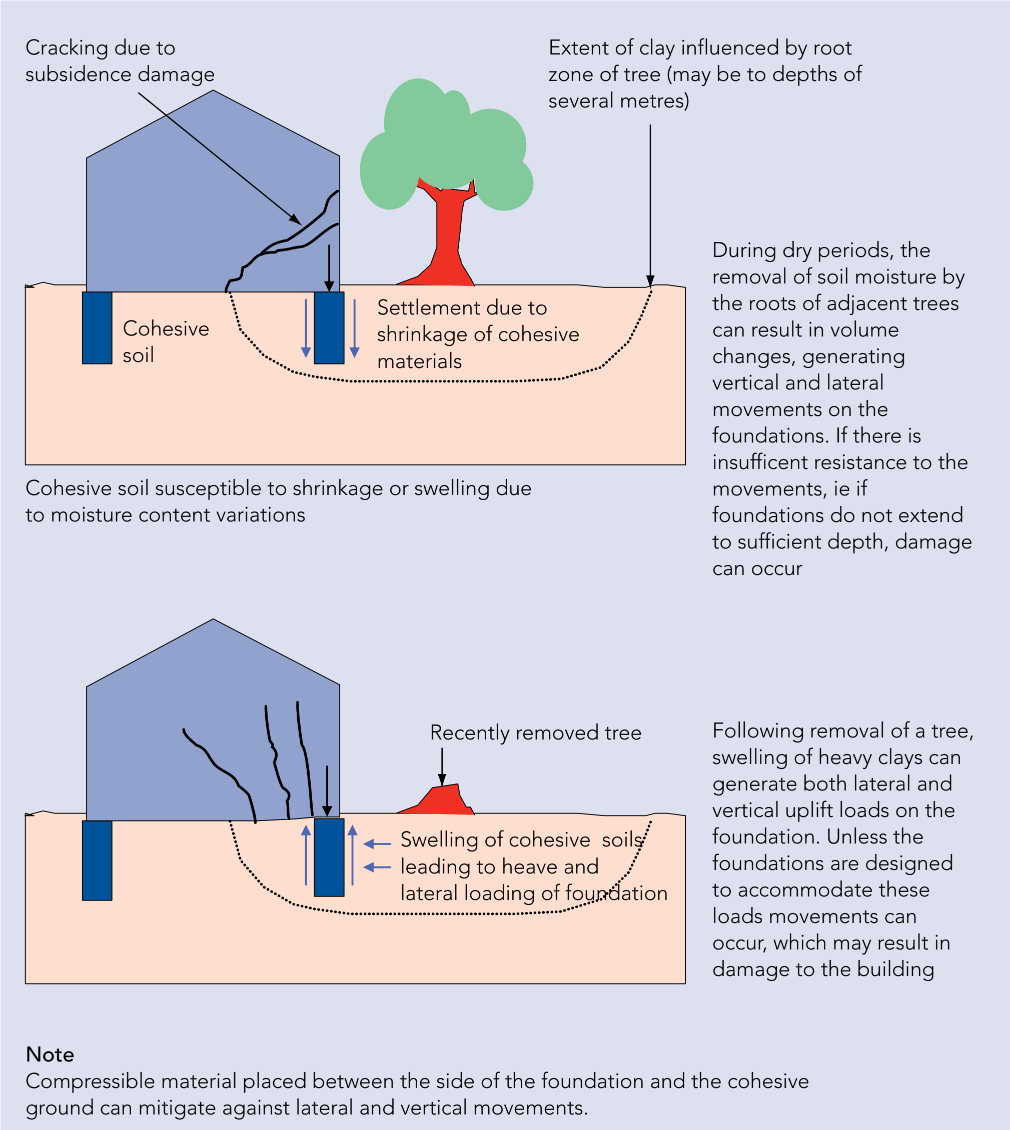



**Figure 10** Failure of trenchfill due to heave/shrinkage of high plasticity clay. Soil movements disrupting trenchfill footings leading to differential vertical and lateral movements.

**Figure 11** Damage to a property as a result of heaving clay soil beneath the ground beams where piled foundations have been used.

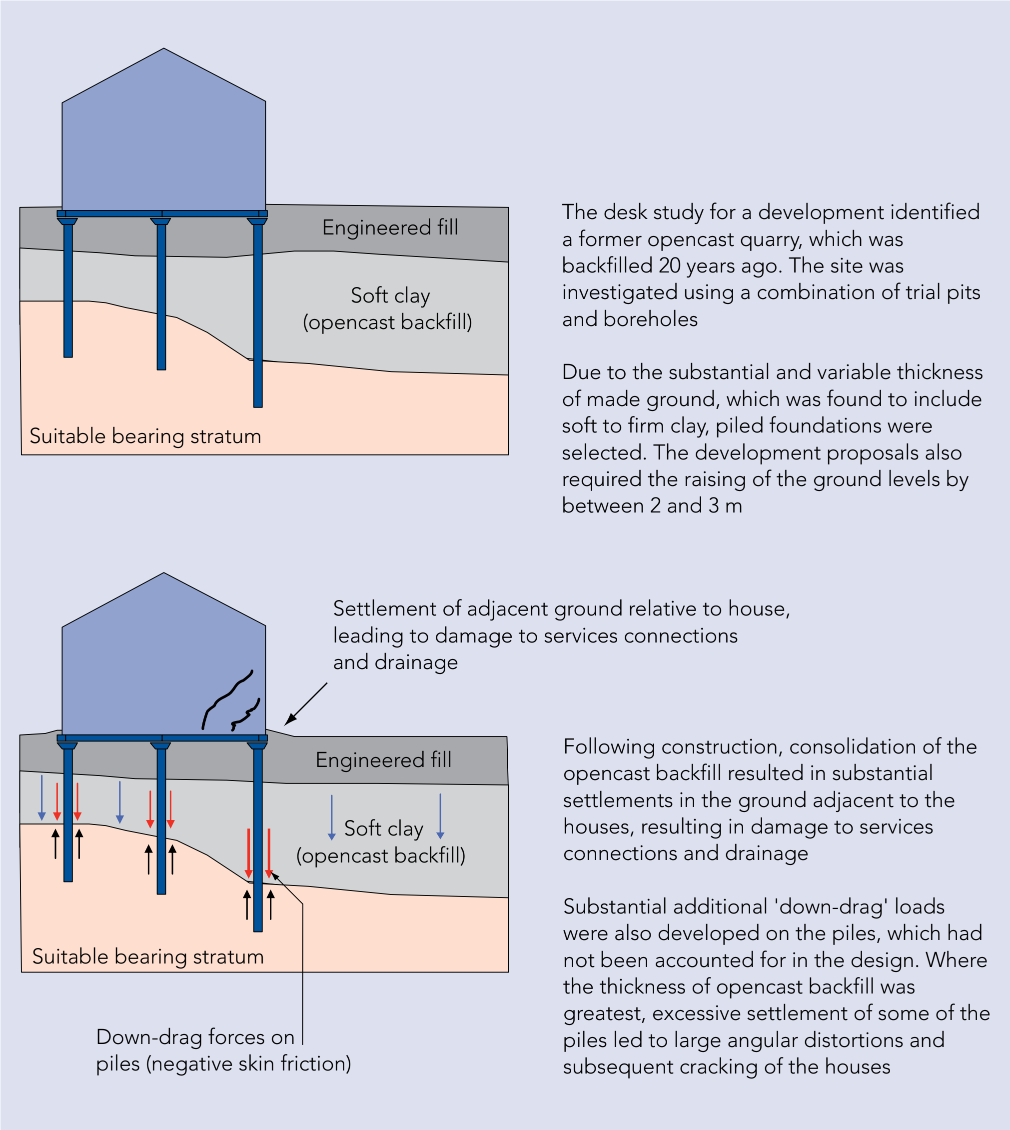



**Figure 12** Excessive settlement following raising of site levels, and failure of piles due to down-drag forces.

## 3.4 Other considerations for foundation selection

In addition to the understanding of the ground and groundwater conditions, the following issues should be considered when selecting foundations:

### 3.4.1 Site logistics

Site constraints can include:

- plant access and working space restrictions, including headroom and width restrictions
- storage area requirements for materials and ancillary plant
- temporary platform requirements to support construction plant. (This is usually subject to specific design to accommodate plant loadings; for guidance see *Working Platforms for Tracked Plant* [21])
- the efficiency of construction may be reduced with greater activity, eg there will be a limit to the amount of construction plant which can effectively operate on a given site at any one time.

### 3.4.2 Costs of materials, transportation, and construction

Factors which influence choice of cost effective foundation solutions include:

- transportation distances for materials and plant
- raw material costs
- speed of installation
- costs of specialist skills
- temporary works costs.

### 3.4.3 Programming issues

The foundation construction is often on the critical path of the programme. The time required for construction of different types of foundations depends on the types of plant used and the methods of installation. The following may also impact on the programme:

- the time required for the installation of temporary support, groundwater control measures or working platforms
- the time required for the procurement of designs for different foundation types
- the availability and mobilisation times of the contractor
- the availability of plant.

### 3.4.4 Health and safety considerations

When selecting foundations, the health and safety implications of foundation design and construction should be considered. The design and construction must comply with the latest safety regulations, including the Construction (Design and Management) (CDM) Regulations 2007.[22] The approved code of practice – 2007[23] gives useful and practical guidance on the CDM 2007 Regulations. The following issues are emphasised:

- The identification of risks early on is critical for the planning and management of those risks.
- Where there is a separate foundation designer and substructure designer, good communication is critical between the parties, eg for pile design, the piles and ground beams are often designed by different organisations.
- The designers responsible for both the selection of foundations and the detailed design should be defined.

### 3.4.5 Environmental considerations

With increasing regulation of site processes, the greater importance of achieving and demonstrating 'sustainability', and the growing number of developments on land affected by contamination, there is an increasing emphasis on 'environmental considerations' when planning all stages of housing developments, including the types of foundations used. Considerations include:

- impacts of vibration and noise
- effects of plant movements and site traffic
- potential impacts of contamination on hydrogeology and water courses
- generation of waste
- volumes of materials used
- carbon dioxide ($CO_2$) emissions or 'embodied carbon'
- potential for exploitation of ground source heat.

These issues, in relation to foundation selection and with reference to the key environmental regulation, legislation and planning requirements, are discussed in section 6.

## 3.5 Summary

The decision process to arrive at the best foundation solution will be influenced by many factors that will have implications for both the cost of construction and the safe construction of the works.

The housebuilder may often favour the use of a foundation solution that is familiar, even where the use of a different foundation type may provide advantages. The use of foundation types that are less familiar, and that may require specialist plant and sub-contractors, may for instance be seen to involve additional risks associated with loss of control and potential for escalating costs and programme.

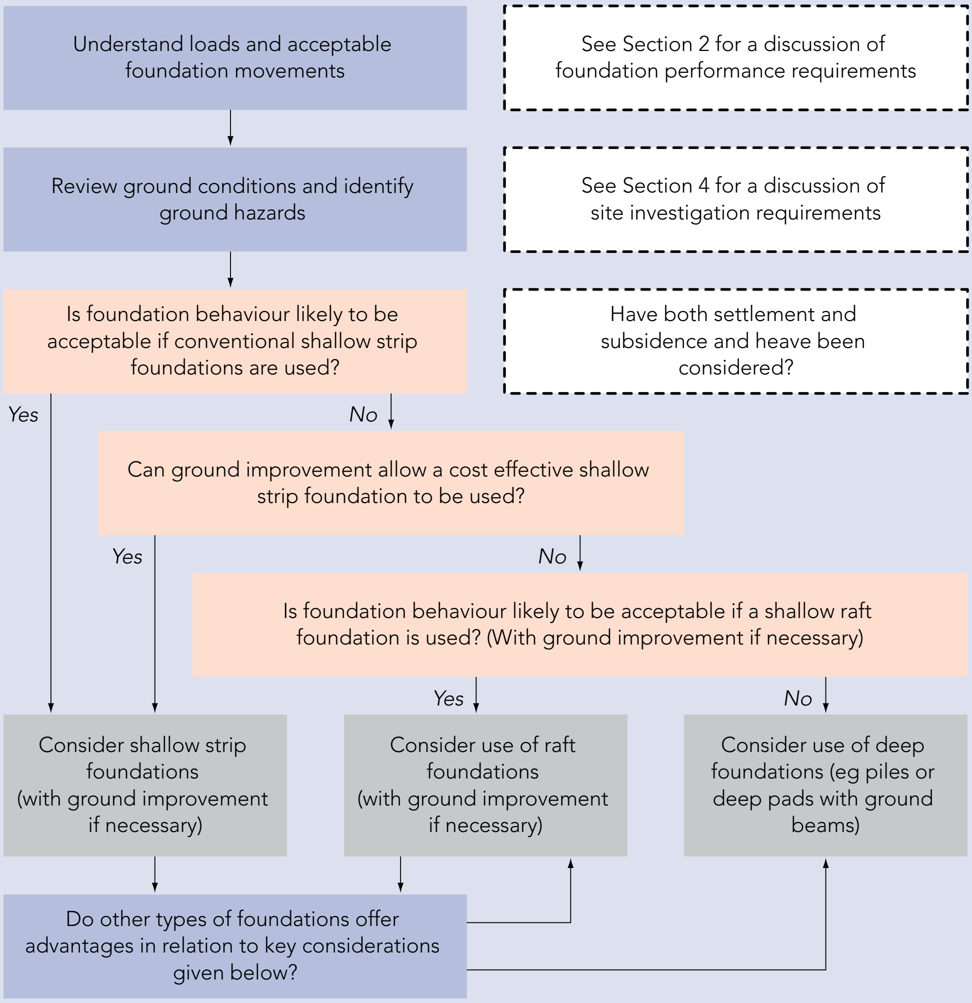
A thorough review of available solutions undertaken by suitably qualified ground engineering professionals can result in the selection of foundations that are more cost effective and efficient, and which carry less overall risk.

Understanding the ground conditions and associated ground risk is of key importance. The earlier in the development programme that these issues are understood, the greater the opportunity to develop effective and efficient solutions. Site investigation is discussed in section 4 of this guide.

Assessments of foundation costs should consider both direct and indirect costs, including the costs of materials, transport, operational costs and timescales of installation (including any temporary works requirements), and disposal costs of waste arisings. The site constraints and limits that these will impose on operations will impact on the programme and construction cost.

Foundation selection will also be influenced by environmental regulations associated with noise and vibration, contamination and waste generation, and issues of sustainability.

Figure 13 (on page 22) summarises the main considerations for foundation selection.




### Key considerations when selecting foundations

- Costs of materials, transport, installation and removal of waste arisings
- Programme
- Plant access, working restrictions and storage area requirements
- Temporary works requirements
- Health and safety considerations
- Impacts of contamination and ground gases on human health and soil and groundwater
- Limitations on noise and vibration
- Minimisation of waste and materials used
- Potential for exploitation of ground source heat.

**Figure 13** Considerations for foundation selection.

# 4 Site investigation

## 4.1 General

"Without site investigation, ground is a hazard".[24]

Of primary importance to developing efficient, economical and effective foundation design is the identification of ground hazards, and minimisation of uncertainty in the ground conditions.

Site investigations should be undertaken to understand the geotechnical and geo-environmental properties of the ground. Investigations should be phased to ensure that they can be effectively targeted, with the findings of an initial site appraisal, including desk study and site walkover survey, informing the subsequent intrusive ground investigation.

It is important that appropriate advice is obtained from qualified geotechnical specialists at all stages of the development, from the initial site appraisal and investigation stage through to design and construction and any subsequent verification and validation testing.

## 4.2 Importance of site investigation to manage ground risk

The risks associated with the ground will generally be reduced with increased investment in site investigation. The cost of the site investigation should be weighed up against:

- the reduction in uncertainty concerning the ground conditions and corresponding reductions in construction risks (which may carry significant financial consequence)
- the potential savings that can be made in design.

The costs of not undertaking adequate site investigation at the early stages will often far exceed the costs of the investigation. These costs may include:

- delays to projects – impacts on cash flow and construction costs
- additional costs due to the need for additional investigation and design, and remediation and repair or reconstruction costs
- damage to reputation, affecting sales of houses.

Chapman and Marcetteau[25] showed that in the UK, about a third of construction projects are significantly delayed, and of those projects, half of the delays are caused by problems in the ground.

The foundations are often on the critical path of a housing development, and any delays are likely to affect all subsequent activities, including the completion dates for projects.

Early data gathering at key strategic points in a development will help with the management and reduction of the ground risks.

Inadequate site investigations can often also result in the adoption of more conservative design parameters and the need for higher factors of safety as a result of the greater level of uncertainty.

## 4.3 Survey of current practice

The survey of current practice discussed in Appendix A identifies a number of concerns with current ground investigation practice. These issues can be summarised as follows:

### 4.3.1 Quantity of ground investigation

More than 60% of respondents to the survey considered that insufficient ground investigation is generally undertaken for low-rise housing developments. This perception was more strongly felt by those who undertake the foundation designs. By contrast, the housebuilders and developers who responded to the survey generally considered the amount spent on investigation to be adequate.

Concerns were also raised that ground investigations are often skewed towards the contamination-related issues, with inadequate consideration given to obtaining information relating to the engineering characteristics of the ground necessary for the design of effective and efficient foundations.

### 4.3.2 Quality of ground investigation information

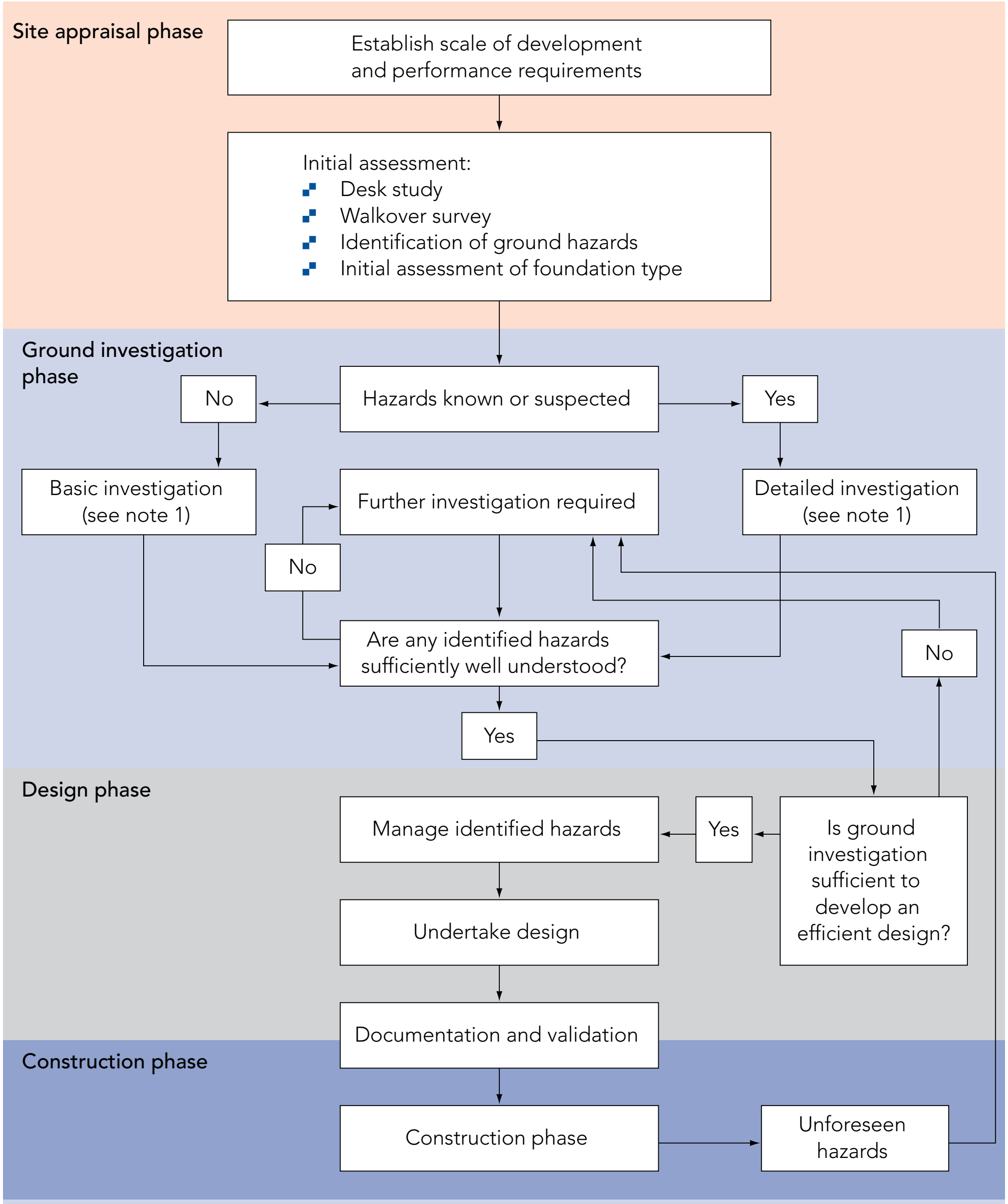
Pile designers generally considered that the quality of the ground investigation information provided to them was often insufficient to allow them to develop the most efficient designs.

A more detailed summary of the typical concerns that have been identified, including a breakdown of how frequently these are encountered, is presented in Appendix A.

In addition to the concerns about the adequacy of investigations to both manage ground risk and derive appropriate design parameters, it appears that basic information necessary to make best use of the available investigation data is often not provided, including the positions and elevations of boreholes and trial pits relative to a fixed datum, and a site plan showing these locations in relation to the proposed development.

## 4.4 Key requirements of site investigation

A procedural flow chart for managing site investigation is presented in Figure 14. (This is compatible with the flowchart given in Part 4.1 of the NHBC Standards,[17] and uses the same definitions, but in addition to ground hazards, Figure 14 also includes consideration of design information.)




Notes
1. Definitions of 'basic' and 'detailed' investigations are as given in Part 4.1 of NHBC standards (www.nhbc.co.uk).
2. The investigations should be scoped to both understand the ground hazards to manage the risks and to develop efficient foundation designs.

**Figure 14** Procedural flow chart for managing site investigation.

## 4.4.1 Site appraisal phase

A desk study and site walkover survey should always be undertaken to establish basic site conditions, identify potential ground hazards and inform the initial assessment of foundation options.

A conceptual site model should also be developed at this stage to allow the key issues to be understood and easily communicated. The conceptual site model should be refined by the findings of the subsequent ground investigation.

### 4.4.2 Ground investigation phase

The ground investigation should investigate ground hazards identified during the site appraisal stage, provide engineering records of the ground and groundwater conditions at the exploratory holes, and provide in situ and laboratory testing sufficient to enable characteristic values of the geotechnical parameters and coefficients to be established for foundation design. The exploratory hole records and descriptions should be prepared in accordance with Eurocode 7, BS EN 1997-2.[26] The positions of exploratory holes should be accurately recorded in terms of both plan co-ordinates and elevation in relation to a fixed datum, and presented on a site plan.

It is important that the investigation is undertaken to sufficient depth to explore the soil or rock properties both around and beneath the proposed foundations, and that the spacing of exploratory holes is sufficient to take into account the variability of the ground anticipated on the basis of the initial site appraisal, as well as any particular regulatory requirements imposed for the site.

The depth and spacing of boreholes and trial pits required for a given site will be a matter for professional judgement, and will be dependent on the degree of uncertainty in terms of both the ground hazards anticipated and the relative variability of the ground conditions anticipated. The assessment of 'uncertainty' should be based on the conclusions of the initial site appraisal including the desk study and site walkover surveys.

Eurocode 7 provides general advice concerning the depth and frequency of exploratory holes for ground investigation. For raft foundations it is recommended that investigation depths should be 1.5 times the minimum plan dimension of the raft, and for piled foundations minimum investigation depths should generally be the deepest of the following:

- pile length + foundation width
- pile length + 5 m
- pile length + 3 × pile diameter.

The primary concern is that the site investigation extends to sufficient depth to include all strata that may influence the behaviour of the foundation, eg soft or loose materials present at depth may remain undetected if the investigation does not extend deep enough. The depth of required investigation below the anticipated founding level may only need to be minimal eg if the pile foundation is built on competent strata with distinct geology, and where weaker strata or structural weaknesses and voids are unlikely to occur at greater depth.

Ground investigations should be developed to reduce the uncertainties in relation to both the geotechnical and geo-environmental ground risks to an acceptable level, and provide the information to enable the most efficient and cost-effective foundation designs to be developed.

The requirements of in situ monitoring and testing and laboratory testing will be site specific. The suggested scope of monitoring and testing presented in Table 2 is recommended as a minimum level of geotechnical and geo-environmental information for sites where the initial site appraisal indicates that piled foundations may be required.

Table 2

| Suggested minimum in situ and laboratory tests (for pile design) |
|---|
| **In situ monitoring and testing** |
| **Density and strength**: In situ testing, eg standard penetration testing (SPT), should be undertaken within all strata. SPTs should be undertaken at regular intervals within boreholes, and a maximum spacing between tests of 1.5 m is recommended (see note 1)<br><br>**Groundwater**: Measurement and recording of groundwater levels from groundwater strikes and monitoring of standpipes/piezometers installed within boreholes. Monitoring should be undertaken over a reasonable time period to investigate variation<br><br>**Ground gases**: Measurement and monitoring of ground gases including methane and carbon dioxide where potential gassing hazard has been identified (see note 2) |
| **Laboratory testing** |
| **Classification tests**: Including soil size grading, moisture content, and Atterberg limits for cohesive strata<br><br>**Strength of materials**: Undrained shear strength for cohesive strata (eg from triaxial testing of undisturbed samples taken from boreholes or from vane tests)<br>Compressive strength of rock samples (point load tests can be used, ideally correlated with unconfined compressive strength [UCS] tests)<br><br>**Aggressivity to concrete**: Water-soluble sulphate and pH tests of soil and groundwater<br><br>**Contamination**: Testing for contamination of soil and groundwater targeted to the conditions and requirements of the individual site (see note 3) |

Notes

1. Alternative methods such as cone penetrometer testing (CPT) or dynamic probing where continuous profiles are obtained can also be used.
2. Best practice guidance is provided in NHBC document *Guidance on Evaluation of Development Proposals on Sites where Methane and Carbon Dioxide are Present*. 4th edition.[27]
3. Best practice guidance is provided in R&D Publication 66 *Guidance for the Safe Development of Housing on Land Affected by Contamination*, prepared by NHBC and the Environment Agency.[28]

### 4.4.3 Design and construction phases

Following the ground investigation phase, unexpected ground hazards may have been identified, construction proposals may have changed, or additional regulatory requirements may be imposed, which require additional investigation to be undertaken.

Additional ground investigation may also be recommended following the main investigation due to limited access or other practical limitations that prevented full investigation of the site, eg due to the presence of existing buildings or obstructions which may only be removed during the enabling works for the construction phase. It is important that in such instances both the reasons for undertaking the additional investigation and the risks associated with not undertaking it are made clear.

## 4.5 Importance of consulting specialists

Investigations of the ground should be designed and undertaken by competent and suitably qualified and experienced consultants or specialists.

For advice relating to the appointment of appropriate geotechnical personnel, guidance and details of suitable organisations can be found on the website of the Association of Geotechnical Specialists. Various guidance documents are also available, including the *UK Specification for Site Investigation*[29] and the NHBC Standards[17], which provide advice for the appointment of suitable people for both basic investigations, and consultants or specialists for more detailed investigations, as defined in the NHBC Standards.

The skills and level of experience required will be dependent on the nature of the ground hazards at the site; however, the following skills and experience are essential:

- experience and knowledge of undertaking desk studies and site walkover surveys, and designing ground investigations on similar sites

- the ability to both recognise and identify hazards, and understand the implications of these on the proposed development
- the ability to understand the engineering properties of the strata and the groundwater regime at a site and how these will impact on the proposed development
- the ability to effectively and comprehensively communicate the implications of identified ground risk and advise on appropriate foundation designs and remediation and mitigation options
- understanding of the appropriate legislation and regulatory frameworks, including health and safety legislation.

It is important to recognise that although many consultants and specialist organisations will have access to the skills necessary to provide advice on all aspects of ground risk, some specialists may only be commissioned to provide advice in relation to certain ground-related risks, eg discharge of conditions relating to contaminated land. Geo-environmental specialists who are experienced in assessing soil and groundwater contamination may not be experienced in the selection and design of foundations.

It is important that the scope of the site investigation commission provided to the consultant or specialist clearly identifies the range of services that are required in order to ensure that the ground risks are effectively managed.

## 4.6 Summary

The purpose of site investigation is to identify and understand the ground conditions at the site, identify and quantify hazards, and derive suitable characteristic parameters for design purposes. The costs of not undertaking adequate site investigation at the early stages can often far exceed the costs of the investigation.

If ground hazards are to be effectively managed and efficient foundation designs developed, it is important that the ground risks identified during the site investigation are recorded and communicated to all parties throughout all stages of a development.

A conceptual site model and ground hazard and risk register should be prepared during the site investigation and ground investigation phases, and maintained and updated during the design and construction and subsequent verification and validation phases of a development.

# 5 Pile design and construction

## 5.1 General

As discussed in section 3, there will usually be a range of foundation solutions that may be used for a given site with given ground conditions, and a number of considerations will then influence the choice of foundation.

Where piles have been selected as the favoured foundation option, a number of issues should be considered to determine the best pile type and to prepare an efficient design. These should account for:

- ground and groundwater conditions
- method of pile installation/construction
- building performance requirements
- health and safety considerations
- 'environmental' issues.

Some of the key issues in relation to each of these are discussed in section 5.2.

The selection of pile layout and spacing is related to the configuration of the ground beams needed to suit the superstructure requirements. The optimisation of a piled foundation needs to consider the combined pile and ground beam solution, as discussed in section 5.3.

For pile design and construction, of key importance is the understanding of ground conditions in terms of ground hazards and the selection of design parameters, for which an adequate site investigation should be undertaken.

With an improved understanding of the performance requirements of the supported structure and the load settlement response of the pile, design efficiency can be optimised. More comprehensive ground investigation can provide reductions in uncertainty in relation to the geotechnical parameters used for design, and pile testing allows the pile response under loading to be better understood, with subsequent reductions in the factors of safety used.

To gain best value from the process of pile design and construction and the subsequent validation and testing, the importance of seeking advice from appropriately qualified geotechnical specialists throughout the process cannot be overemphasised. Effective communication of information throughout the process is also essential.

## 5.2 Key considerations for pile design and construction

Some of the key issues that are relevant for pile design for low-rise housing developments are listed below.

### 5.2.1 Ground conditions and hazards

- Has sufficient desk study and ground investigation been undertaken to assess the ground risks and develop the ground model?
- Is the groundwater regime well understood and have potential changes in groundwater conditions been considered?
- Has direct or indirect measurement of the strength and stiffness properties of all strata been obtained?
- What degree of variability of ground conditions can be anticipated across the site?
- Is there potential to encounter buried man-made or natural obstructions during the piling works?
- Are the ground levels going to be raised or lowered during the development?
- Is there a risk of compressible strata generating negative skin friction forces?
- Is there a potential for swelling and shrinkage of clays that could affect pile performance?
- Have the effects of any chemicals present within the ground been considered in terms of durability and long-term integrity of the pile?
- Are there other ground-related effects that could induce movements and forces on the piles, such as the presence of sloping ground, differing levels of excavation, or surcharge either side of pile, etc?
- Eurocode 7 (BS EN 1997-01) presents more detailed guidance on the situations that should be considered for design of piles. In particular, reference can be made to Clause 2.2.

### 5.2.2 Pile installation/construction

- Is there adequate working space?
- Have the effects of pile construction on adjacent foundations, buildings and occupants been considered?
- Can the required pile installation tolerances be achieved?
- Have installation-induced stresses been considered and can the integrity of the pile be affected by installation?
- Have the impacts of unstable ground and presence of groundwater on pile installation been considered?
- Is there likely to be difficulty in achieving the required depth/penetration to satisfy the design, eg due to the presence of obstructions or dense strata?

### 5.2.3 Performance requirements of buildings and foundations

- Has an assessment of acceptable foundation settlements been made, including differential movements that can be accommodated?
- Is there sufficient design information in terms of the pile loading requirements and specification of acceptable pile movements?
- Has the design of the pile under all combinations of applied loads been considered?
- Is there an understanding of the fixity conditions between the pile and ground beam construction? Are the designers of both the piles and ground beams aware of each other's design assumptions, and are they consistent?
- Have forces/moments due to construction tolerances been allowed for in the design of the piles and ground beams?
- Has static load testing of test piles been considered as a means of developing a more efficient design?
- Will static load testing be undertaken to verify the pile settlement response of working piles following installation?

### 5.2.4 Health and safety considerations

- Can the foundations safely support the building loads?
- Does the design and construction methodology ensure that the stability of adjacent structures is not compromised, eg as a result of installation-induced ground movements?
- Has a risk assessment process been implemented by both the designer and main contractor?
- Have the results of the risk assessment been properly communicated?
- Can the proposed foundations be safety constructed given the particular site constraints?
- Are solid or liquid contaminants or ground gases present within the ground which could present a health hazard, and has suitable mitigation of these risks been considered?
- Where there are interfaces between design responsibilities, eg between design of ground beams and piles, has the design work been co-ordinated?
- Has an assessment of noise, dust and vibration been considered, and are these within acceptable limits?
- The pile design and construction must comply with the latest safety regulations, including the Construction (Design and Management) Regulations 2007.[22] The approved code of practice – 2007 (ACOP)[23] gives useful and practical guidance on the CDM 2007 Regulations. Approved Document A of the Building Regulations defines responsibilities in terms of stability of the supported and adjacent structures.
- Guidance is also provided in the ICE *Specification for Piling and Embedded Retaining Walls*[30] and on the website of the Federation of Piling Specialists (FPS).

### 5.2.5 Environmental considerations

- Have the impacts of piling been considered in terms of potential effects on groundwater, eg the potential for creation of contamination pathways?
- Have noise and vibration issues been considered?
- Has the disposal of contaminated arisings been considered?
- Has potential exploitation of ground source heat been considered?
- Sources of additional guidance on the general issues that should be considered for pile design and construction are listed in section 8.

## 5.3 Optimising the ground beam and pile layout

The design of piled foundations cannot be considered in isolation from the design and arrangement of the ground beams, and pile caps if used, which transfer the building loads to the piles.

Where piles are closely spaced, the size of the ground beams and reinforcement required to span between the piles will reduce; however, this does not necessarily lead to the best overall solution since the piles may well be less 'efficient' in terms of their number and load capacities.

There is likely to be an optimum solution of pile spacing and beam size where the overall cost of the combined system is minimised; however, this may not always be realised in practice. The layout of the house will impose a certain pile layout and there will only be a limited opportunity to increase or decrease the pile spacing. Single piles at all the corners and also at midway points along the walls are typically adopted. This results in the pile spacing for typical UK housing usually being between 3 m and 5 m.

Ground beams for low-rise housing can either be pre-cast or constructed in situ. Where the soil is firm enough to form a stable face, in situ ground beams can be cast directly within trenches. In poor ground some type of formwork may be required.

Pre-cast ground beams are cast to predetermined lengths in the factory and are placed to span between the pile-caps, which may themselves be either pre-cast or alternatively cast in situ on top of the piles. Ground beam and pile cap details are illustrated in Figure 15.

The pile design will need to consider both the load combination applied from the building, and the fixity of the pile head into the sub-structure. The pile design should allow for additional loading effects developed as a result of the normal construction tolerances of 75 mm in plan as well as the 1:75 verticality tolerances of the piles. If the pile head is designed as a pinned connection, ie unrestrained against rotation, this can potentially require less reinforcement at the interface between the pile and beam. However, under horizontal loading, larger bending moments may be developed in the pile. These can be reduced by fixing the pile head, but this will generally require more reinforcement in the ground beams. Typical ground beam reinforcement for cast in situ ground beams are shown on Figures 16 and 17.

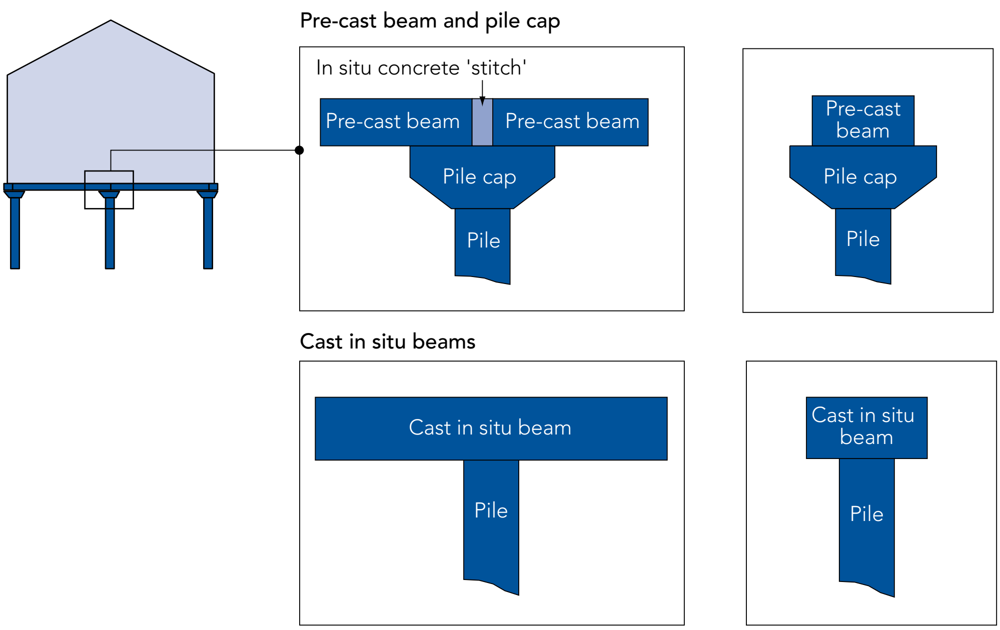



**Figure 15** Typical pile and beam arrangements.

**Figure 16** Pile and ground beam reinforcement.

**Figure 17** Ground beams prior to casting of concrete.

## 5.4 Selection of appropriate pile type to suit ground conditions

For detailed guidance on specific piling methods available and their suitability and application in different ground conditions reference should be made to CIRIA Report PG1.[31] Other publications which provide guidance include:

- Tomlinson and Woodward – *Pile Design and Construction Practice*.[32]
- Fleming et al – *Piling Engineering*.[33]

A summary of issues, advantages and constraints for the main piling methods typically employed for low-rise housing is presented in Appendix D. The common types of piles used are shown on Figure 18. The installation of driven steel and driven pre-cast concrete piles is shown on Figures 19 and 20, and installation of bored piles is shown on Figure 21. Some of the key issues relating to displacement and non-displacement pile types are presented below.

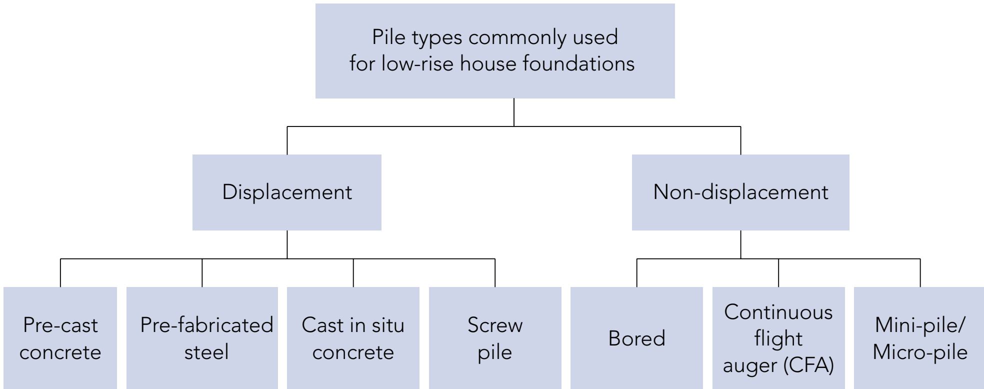



**Figure 18** Pile types commonly used for low rise housing.

### 5.4.1 Displacement piles

Advantages:

- rapid installation in most soil conditions
- prefabricated piles can be inspected prior to driving
- minimal generation of spoil
- indirect evidence of ground conditions and pile capacity may be obtained during driving.

Disadvantages:

- noise and vibration levels may be unacceptable
- ground heave potential
- susceptible to refusal on man-made or natural obstructions
- potential for driving-induced damage for some types of pile, eg pre-cast concrete; can be controlled however by careful choice of pile and driving technique.

### 5.4.2 Non-displacement piles

Advantages:

- limited noise and vibration
- arisings can be used to verify ground conditions.

Disadvantages:

- quality of workmanship and instrumentation influences pile performance
- support to boring required in granular soils, particularly below the water table
- generation of spoil.

**Figure 19** Installation of driven steel piles.

**Figure 20** Installation of driven pre-cast concrete piles.

**Figure 21** Continuous flight auger bored piling.

## 5.5 Pile load capacities

Both displacement and non-displacement piles work by transmitting the loads to a suitable bearing stratum, using a combination of shaft friction and end-bearing resistance. As illustrated in Figure 22, piles are commonly classified as 'friction' or 'end-bearing' piles, depending on how they generate the majority of their load resistance:

- **friction pile**: where the majority of the pile's compressive load carrying capacity is derived from the shaft friction component
- **end-bearing pile**: where the majority of the pile's compressive load carrying capacity is derived from the base resistance component.

Also illustrated in Figure 22 are the fundamental concepts of negative skin friction and heave:

- **negative skin friction**: additional loading on a pile where settlement of compressible strata surrounding the pile induces a down-drag force. This can occur both when ground levels are raised during a development and when fill materials have already been placed prior to the development.
- **heave**: swell behaviour can cause tension loads in the pile, which may cause cracking prior to application of compression loads from the supported structure.

## 5.6 Selection of design parameters

The importance of adequate ground investigation information is discussed in section 4.

The strength and stiffness parameters used for the pile design will have a direct impact on the resulting pile size that is required.

BS EN 1997-1 (Part 1 of Eurocode 7)[11] gives guidance in relation to the selection of characteristic values of geotechnical parameters for design purposes. Characteristic values used for design are required to be cautious estimates of the values affecting the behaviour of the soil or rock, and are to be based on the results and derived values from laboratory or field tests together with experience.

BS EN 1997-2 (Part 2 of Eurocode 7)[26] provides additional guidance relating to the derivation of values of geotechnical parameters and coefficients, together with examples of the application of field test results.

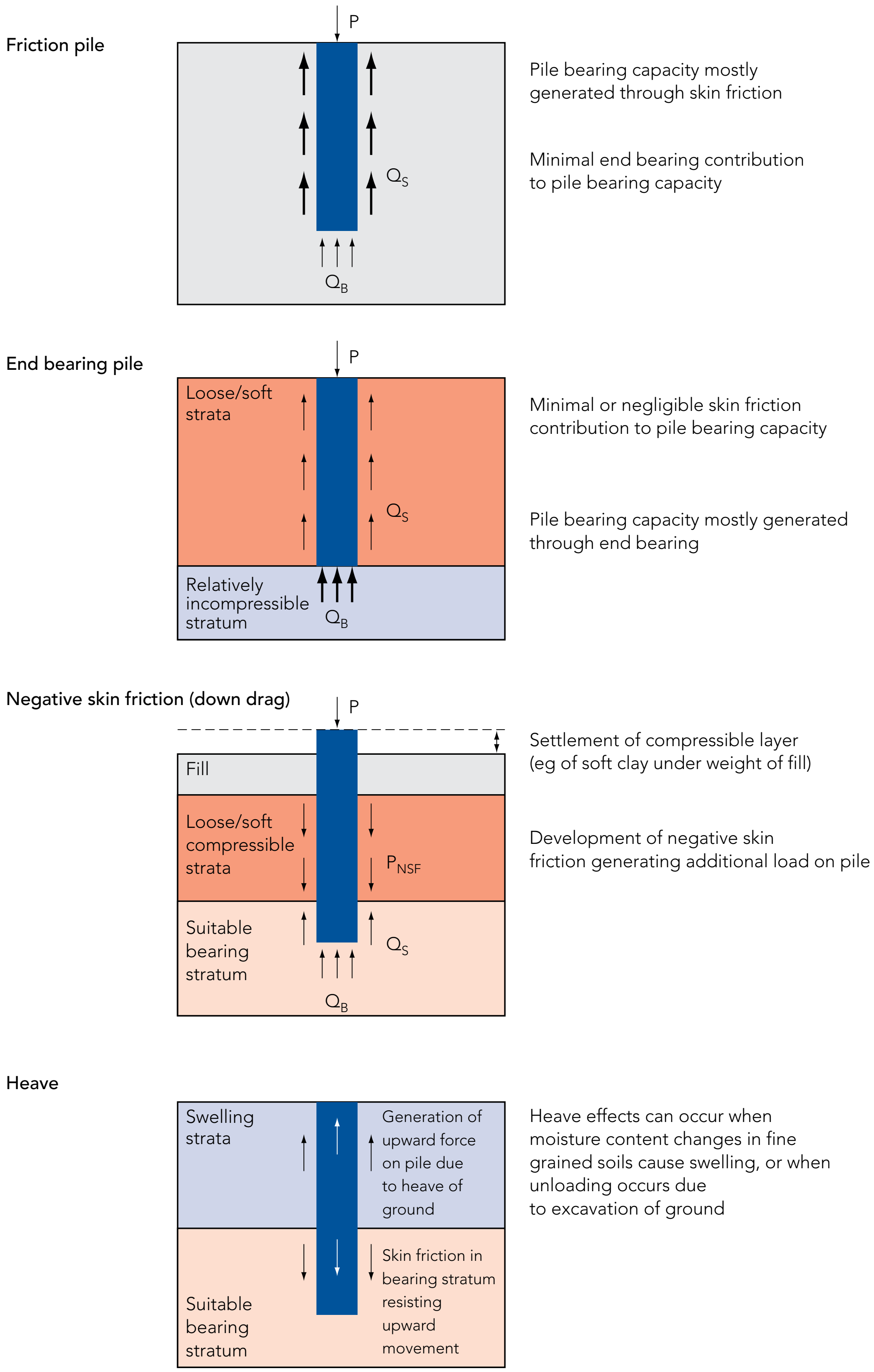
The efficiency of the pile design will depend on the amount of and the quality of the ground investigation information available. Where only limited information is available, this can result in the adoption of more conservative design parameters and hence more conservative design than might otherwise be appropriate to the site conditions.

## 5.7 Design methods

The basic requirements of pile design are that:

- there is an adequate factor of safety against failure
- the pile settlements at working loads are within acceptable limits for the supported structure.

The main codes relating to pile design are BS 8004[34] and BS EN 1997-1 or Eurocode 7. Although these codes differ in the manner in which safety factors are applied, the resulting pile designs should not differ significantly. The main difference in the design codes is that with Eurocode 7 partial factors are applied to characteristic values of loads, soil strength parameters, and pile resistances, compared to BS 8004 where a 'global' factor of safety is used. Both codes support a number of different design approaches, including design by calculation using empirical correlations or analytical methods, design using results of preliminary static load tests, design by dynamic pile-driving formulae, and use of stress wave analysis.




P Pile applied load
$P_{NSF}$ Additional pile load due to negative skin friction
$Q_B$ End bearing resistance
$Q_S$ Skin friction resistance

**Figure 22** Pile load components.

Both codes allow for reductions in safety factors where static load tests are undertaken and the settlement response of the pile can be more accurately defined.

More detailed summaries of the requirements of the BS 8004 and Eurocode 7 approaches to pile design are provided in Appendix F. Some of the key considerations are presented below.

The ICE *Specification for Piling and Embedded Retaining Walls*[30] is generally used as the basis for piling specifications, and also provides useful guidance relating to pile design and testing.

### 5.7.1 Pile-bearing capacity

#### Traditional design approach to BS 8004

Pile design undertaken in accordance with BS 8004 is based on the application of a global factor of safety against ultimate failure to determine acceptable pile working loads. This approach is typically defined by the following relationship:

$$Q_{SAFE} \leq \frac{Q_{su} + Q}{FOS}$$

where:
$Q_{SAFE}$ is the safe working load
$Q_s$ is the pile shaft ultimate capacity
$Q_B$ is the pile base ultimate capacity
FOS is a global factor of safety typically in the range of 2.0–3.0.

Generally, the assumption is made that pile settlements will be within acceptable limits when this approach is used; this is discussed further in section 5.8.

#### Design in accordance with BS EN 1997-1 (Eurocode 7)

BS EN 1997-1 (Eurocode 7) supersedes BS 8004, and as of March 2010 it will be a requirement for pile designs to be in accordance with Eurocode 7.

Eurocode 7 design principles require that both the ultimate limit states (ULS) and serviceability limit states (SLS) are assessed under application of different sets of partial safety factors.

Satisfying the ULS requirements under Eurocode 7 requires compliance of the following relationship:

$$F_{c;d} \leq R_{c;d}$$

where:
$F_{c;d}$ is the design axial compression load on the pile
$R_{c;d}$ is the design compressive resistance of the pile.

For the various approaches that can be adopted, ranging from design by calculation to design based on the results of pile load tests, the UK National Annex presents the recommended values of partial factors to be used. This is discussed further in Appendix F.

### 5.7.2 Pile settlements

It is required by both BS 8004 and Eurocode 7 that pile settlements under working load, and in particular differential settlements, are controlled to be within limits that can be tolerated by the structure.

BS 8004 states that the factor of safety necessary to ensure that settlements are acceptable may in some cases be larger than that required to prevent bearing failure.

Eurocode 7 emphasises the need for an assessment of the anticipated pile settlements to ensure they are within the tolerances of the supported structure. A discussion of the Eurocode 7 approach to ensure that the settlements are within the serviceability limits is presented in section 5.8.4.

## 5.8 Design in practice

### 5.8.1 Reasons for factor of safety

A factor of safety is applied for a number of reasons, including the following:

- to cater for variability in the strength and compressibility of the ground
- to cover uncertainties in the calculation method
- to result in pile designs where typically the total and differential settlements are within acceptable limits.

#### Variability in the ground conditions

Understanding the ground conditions is critical to establishing appropriate design parameters.

Even where the stratigraphy is well-defined there will generally be variability within individual strata. The factor of safety is intended to allow for this variation, but does not allow for unforeseen conditions where the ground may have very different engineering properties to those anticipated. However, based on the results of the questionnaire survey discussed in Appendix A, it appears that often it is incorrectly assumed that the purpose of the factor of safety is to cater for such unforeseen ground conditions.

The factor of safety is not intended to be a replacement for a well-designed ground investigation, for example the potential presence of soft clay within a medium-dense gravel will not necessarily be mitigated by simply applying a larger factor of safety.

#### Uncertainties in the calculation method

Basic calculations of pile load capacity are usually based on empirical relationships, relating pile behaviour to derived soil strength parameters by applying empirical coefficients.

It is generally accepted that calculation methods cannot predict failure loads to any great accuracy. Tomlinson and Woodward[32] suggest that an accuracy of plus/minus 60% of the value determined from full-scale load tests can usually be achieved by calculations, provided that ground conditions are well understood.

### 5.8.2 Settlement considerations

Settlement prediction is not always considered during the pile design for low-rise housing, and it is common practice for the design to rely solely on the application of a 'global' factor of safety, with acceptable settlements implicitly assumed. However, this is not in strict accordance with the code requirements.

To apply a check on pile settlement requires the understanding and specification of acceptable settlement limits. Based on the responses to the survey undertaken during preparation of this guide, it appears that limiting values of settlement or other 'serviceability requirements' are rarely specified for low-rise housing. This may be attributed to a lack of guidance or awareness of how much settlement is acceptable.

Tomlinson and Woodward[32] report that on the basis of UK experience and following the review of extensive pile-testing data, for the smaller diameter piles used for supporting low-rise structures, the settlements will generally be less than 10 mm if a global factor of safety of 2.5 is applied to the ultimate resistance of a pile. This is illustrated in Figure 23, which presents an idealised load versus settlement relationship for a small diameter pile-carrying load predominantly by shaft friction.

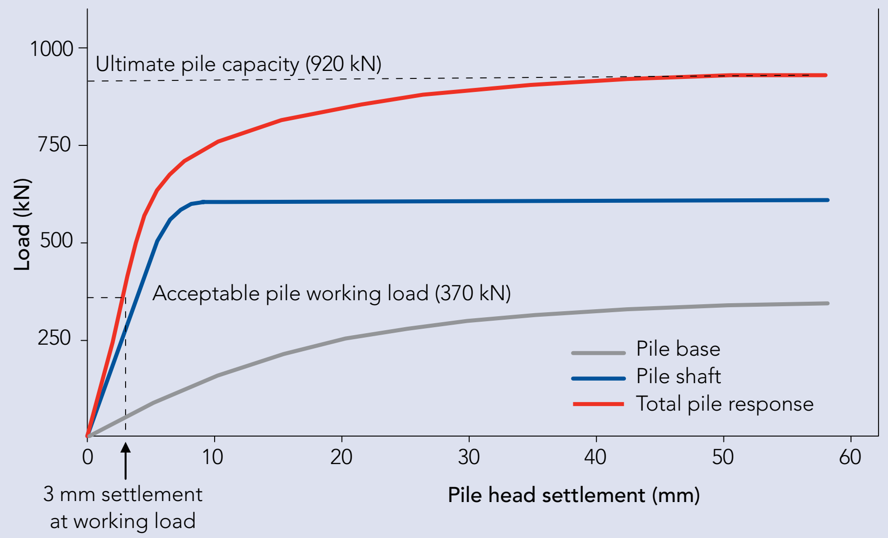



**Figure 23** Pile load versus settlement.

The load settlement behaviour of a pile is a product of the differing response of the shaft and the base components. The pile shaft resistance typically is mobilised at relatively low settlements, which have been reported to be in the region of 0.3–1% of the pile diameter. In most soils it takes significantly greater movements to fully mobilise the base resistance, often more of the order of 10–20% of the pile diameter.[32]

As different degrees of settlement will be required to mobilise the resistance provided by shaft and base of a pile, shaft-controlled piles can often provide support with less settlement than piles that are designed to provide support in end bearing.

In order to control pile settlement under working load conditions, a convenient approach can be to limit the magnitude of the load acting on a pile to less than the ultimate shaft capacity (subject to an adequate global factor of safety in terms of the combined ultimate shaft and ultimate base capacities). This criterion will often be achieved as a matter of course for most piles in cohesive soils. However, in the case of some piles in cohesive soils and the majority of piles in cohesionless soils, the base resistance will be a greater component of the overall pile resistance and will need to be mobilised to a greater degree under working load conditions. In such cases, the designer will need to explicitly assess the pile settlements that are needed to mobilise the required base resistance under working load. This will be dependent on the stiffness of the ground at the pile base. The possibility of disturbing or loosening the ground beneath the pile base during pile installation should be considered, as this will influence the pile base settlement behaviour.

In the case of piles founded on very dense soils or intact rock, the piles can often achieve their maximum carrying capacity with only minimal settlement at the pile base, with the pile head settlement resulting mainly from elastic compression of the pile. In such cases, the load may be carried entirely in end bearing with minimal settlements, and the safe working pile load governed by the structural capacity of the piles.

Fissured, fractured, or weathered rocks will require an understanding of the rock mass characteristics to assess behaviour of piles under load, as closure of fissures or fractures under load can result in greater magnitudes of pile settlement than will occur for intact rock.

In addition to settlements at the interface between the pile and the ground, settlement at the pile head will occur due to elastic compression of the pile under loading. Slender piles typically used for low-rise housing can be subject to quite significant elastic shaft compression, which may result in considerable settlements. The elastic compression will be dependent on the modulus of the pile material, the dimensions of the pile, and the pile loads, and can be readily calculated. Much of the compression/settlement within the pile will occur during construction and before surface finishes are applied.

### 5.8.3 Methods of assessing pile settlements

There are several published methods available for estimation of pile settlement. Guidance on the approach and application of some of the more commonly used methods is given in:

- Tomlinson and Woodward – *Pile Design and Construction Practice*.[32]
- Fleming et al – *Piling Engineering*.[33]

The published methods vary in their approach from simplified empirical estimations to analytical solutions. Several of the available methods require assumptions to be made regarding the relative contributions of the shaft and base resistance to the total pile load carried. Such an assumption is required for the commonly used method outlined in Tomlinson and Woodward,[32] which estimates pile head settlement based on the sum of the elastic shortening of the pile shaft and the compression of the soil beneath the base. A worked example using this approach is included in Appendix G.

The methods presented in Fleming et al[33] involve a more analytical approach to estimating pile settlement. The approach discussed in section 4.2.3 of Fleming et al. provides the basic theory behind several numerical programs which have been developed to calculate pile load settlement behaviour.

### 5.8.4 Requirements of BS EN 1997 (Eurocode 7)

Eurocode 7 generally requires a direct assessment of the serviceability limit state, ie estimation or measurement of pile settlements to ensure they do not exceed permissible structure movements. However, the code accepts that there are instances, particularly where the piles bear on medium to dense soils, where the safety requirements for the ultimate limit state design will be sufficient to prevent a serviceability limit state in the supported structure.

Eurocode 7 allows either static load tests or calculative methods to be used to verify the settlement under loading. Clause 7.6.4.2 states where no load test results are available, the load settlement performance of individual piles should be assessed on the basis of empirically established safe assumptions.

The UK National Annex to Eurocode 7 permits a significant reduction in partial factors when the load settlement response of the pile has been verified by conducting load tests (to 1.5 times the working load) on more than 1% of piles. This can lead to a more efficient pile design. Static load testing is not routinely undertaken for low-rise housing developments, particularly for the smaller scale developments. The potential benefits in relation to the reductions in uncertainty and factors of safety and improvements in the efficiency of the pile designs developed as a result of undertaking such tests should be considered, particularly on larger developments where the potential savings will be more significant.

Where calculation methods are used to determine the ULS, the combination of partial factors used to calculate the design pile resistance are similar in magnitude to a traditional 'global' factor of safety of 2.5–3.0. With this in mind, it could be argued that the traditional approach of assuming acceptable settlements by satisfying the ULS requirements may still be applicable for low-rise house foundations.

## 5.9 Specification of acceptable settlements for low-rise housing

### 5.9.1 Tolerance to differential settlements

The acceptable differential settlements of low-rise housing do not appear to be routinely considered in any explicit way when designing piles. It is generally assumed that the settlements will be acceptable. It does not appear to be common practice to specify limits of differential settlements for design purposes.

The available guidance relating to acceptable differential distortions can however be used to identify limits of differential movements that could be included in piling specifications for low-rise housing. The aim would be to set pile settlement criteria such that the associated movements are lower than those which would lead to the onset of damage in terms of the serviceability of the building.

Based on the literature review for the typical UK house construction, which is discussed in section 2, there is general agreement, which is supported by design codes (including Eurocode 7), that serviceability type damage is unlikely to occur at angular distortions less than the order of 1/500, where sagging occurs, and 1/1000, where hogging occurs.

On the basis of this, the magnitude of acceptable differential settlements can be assessed for a given pile spacing. For typical house layouts, pile spacings will be to the order of 3–5 m, and applying the published limits for acceptable distortions will give the acceptable differential movements between piles shown in Table 3.

Table 3

**Acceptable differential settlements between piles, based on published limits.**

| | Pile spacing (m) | Maximum differential settlement of piles (mm) |
|---|---|---|
| Sagging | 3–5 | 6–10 |
| Hogging | 3–5 | 3–5 |

As discussed in section 2, tolerance to differential settlements will vary depending on the form of construction, housing layout and materials used.

The ground beams and the superstructure will provide bending and shear stiffness, which will lead to redistribution of loads on the piles. This will in practice tend to limit the potential movements of adjacent piles in relation to each other and result in lower levels of differential movements. Quantifying the contribution of ground beam restraint is relatively complex, but, in general terms, the stiffer the ground beams the greater the potential to redistribute the loads and the lower the differential movements that will occur.

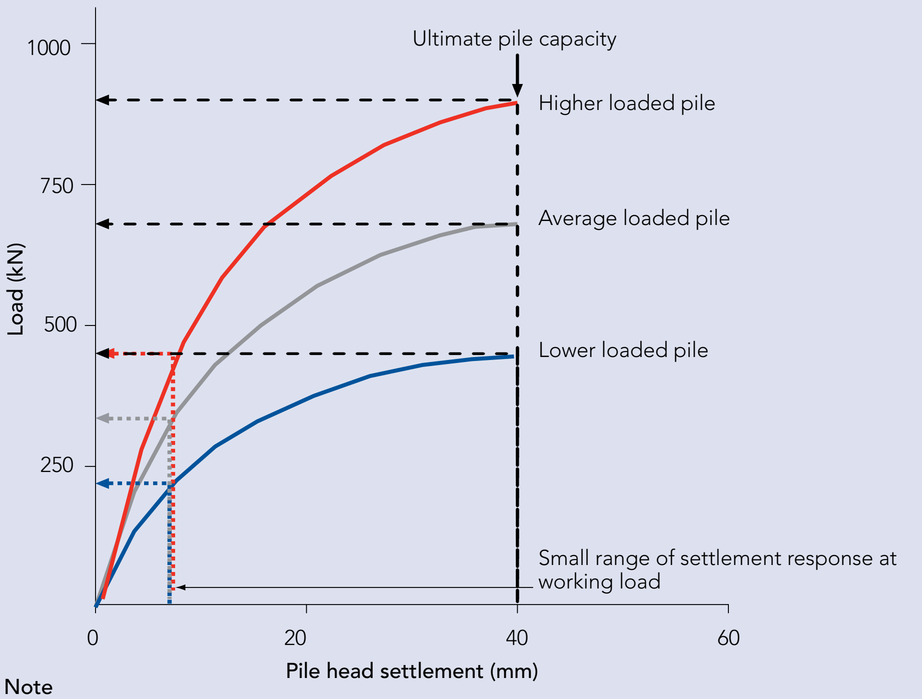
### 5.9.2 Tolerance to total settlements

Although damage to buildings is not usually explicitly linked to total settlements, total settlements of piles need to be controlled to limit potential differential movements. If piles are designed to allow large total settlements, then for differential movements to be within acceptable limits, the pile settlement behaviour would either need to be very well understood, or the structure would need to be stiff enough to redistribute loads extensively.

Where the ground conditions are relatively uniform, the differential settlements between adjacent piles will tend to be small, and relatively large total settlements could occur before differential settlements reach unacceptable limits. This is illustrated in Figure 24 for piles with different applied loads. If the ground conditions are more variable, there will tend to be a greater variation in settlement response between adjacent pile positions. This is illustrated in Figure 25.

Where ground conditions are relatively uniform across the house footprint, and loads are generally of similar magnitude, maximum differential settlements between pile positions are unlikely to exceed 50% of the total settlements, and if differential movements needed for example to be controlled to 6 mm, total pile settlements under working loads in the order of 12 mm would be reasonable for use in piling specifications.

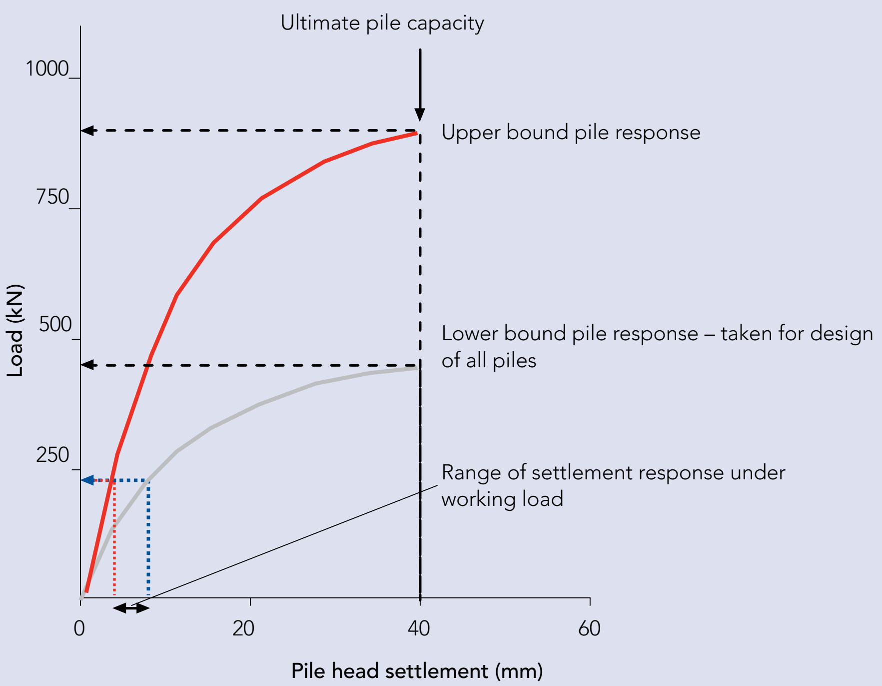
If a large variation in response of the ground conditions is anticipated, it may be necessary to control total settlements more tightly, and, similarly, if pile response were to be more predictable, larger total settlements may be acceptable.




Note

If the capacity of each individual pile could be accurately estimated, and a factor of safety was applied to the ultimate capacity for each, the settlement of the piles is likely to be very similar. Here a factor of safety of 2 has been applied to the ultimate capacity for each individual pile, and piles have been designed to carry different loads.

**Figure 24** Variations in pile settlements – relatively uniform ground.




Note

In practice, a factor of safety will be applied to a 'lower bound' pile capacity, and the settlement response of each pile will vary as a result of the natural variability of the ground. The worst case (ie lower bound) pile response shown here gives 8 mm of settlement under working load, whilst 4 mm of settlement has occurred for stiffer piles (ie upper bound). Here a factor of safety of 2 has been applied to the 'lower bound' pile.

**Figure 25** Variations in pile settlements – variable ground.

### 5.9.3 Tolerance to tilt settlements

Although building foundation design needs to consider the risk of unacceptable tilt settlements, this is generally more of an issue for buildings which are founded on shallow foundations, such as rafts or reinforced strips.

Houses supported on piles will not be susceptible to excessive tilt movements if the piles are designed to adequately control the total and differential settlements.

## 5.10 Structural modifications to redistribute loads

As discussed in section 5.9.1, the ground beams can provide bending and shear stiffness that will lead to redistribution of loads on the piles, which will tend to limit the movements of adjacent piles in relation to each other and potentially result in lower levels of differential movements.

The manner in which the loads become redistributed by the ground beams is dependent on the relative size and span dimensions of the beams and the nature of the structural connection between the pile and the ground beam and the amount of reinforcement that is used.

Stiffer ground beams can generally redistribute loads more effectively, and reduce the differential movements that will act on the superstructure. However, the assessment of interaction between the pile and ground beam behaviour can be complex, and it is unlikely to be worthwhile undertaking such assessment for housing developments, other than in qualitative terms.

## 5.11 Pile testing

Pile testing can be undertaken for a number of reasons, including the validation of pile designs, quality control during construction, and as the basis of the pile design. Pile testing may be a requirement imposed by the warranty provider.

The extent of pile testing undertaken should be assessed not only on the basis of any regulatory requirements that may be imposed, but also on the level of risk or uncertainty posed by the site conditions and the potential for reductions in this uncertainty, allowing reduction in the factors of safety and more efficient designs.

Feedback from the questionnaire which was undertaken to gauge opinion regarding pile testing is included in Appendix A. The routine use of pile load testing does not appear to be commonplace for low-rise housing developments, especially for the smaller developments. There may be opportunities for overall savings to be realised, however, if pile tests are undertaken.

Eurocode 7 design principles allow pile design to be undertaken on the basis of preliminary static load tests carried out in advance of the main phase of development, which can enable more efficient pile designs to be developed as the pile settlement behaviour of the piles can be directly assessed.

As discussed in section 5.8, Eurocode 7 design principles allow significant reductions in partial safety factors when the load settlement response of a pile has been verified by preliminary static load tests of test piles or static load tests on working piles (Fig. 26). This can lead to significant cost savings.

There may also be other benefits of pile testing, particularly of preliminary test piles, which may include confirmation of the variability in ground conditions across the site, the pile buildability, and demonstration that noise and vibration levels will be within acceptable limits in advance of the main piling contract.


**Figure 26** Static load testing of piles using kentledge.

There are a number of different types of pile tests, which provide different levels of information, these include:

- **Preliminary static load tests**: Directly measure the load settlement behaviour of a test pile under loading. Piles are generally loaded to failure and provide a complete load settlement curve.
- **Static load tests on working piles**: Working piles tested to the design verification load plus 50% of the safe working load. This is explained in Appendix H. Results of static load tests allow settlement at working load to be measured.
- **Dynamic load tests**: Measure dynamic load resistance, and can allow predictions to be made of the pile performance under static load. An example of dynamic load test being undertaken using a 1.8 tonne drop hammer is shown in Figure 27. Tests are generally significantly quicker and cheaper than static load tests, but they do not provide a direct measures of pile settlement under loading, and care must be taken in the interpretation of the results (Fig. 27).

A discussion and summary of the advantages, limitations, and applications of the different types of pile test, including a review of the interpretation of test results, and sources of further guidance is presented in Appendix H.

**Figure 27** Dynamic load testing of piles (Image courtesy of PMC pile testing).

## 5.12 Derivation of pile loads

For low-rise housing, it is generally the case that pile loads are calculated based on the assumption that the ground beams between piles act as simply supported beams. In reality, due to its stiffness the building structure itself will redistribute loads, and the loads carried by the ground beams will be further redistributed between piles in a more complex manner as the piles interact with the ground. However, attempting to calculate pile loads based on anything other than simply supported beams is generally not practical for low-rise housing developments. It should be recognised however that actual loads acting on individual piles will vary from the calculated values.

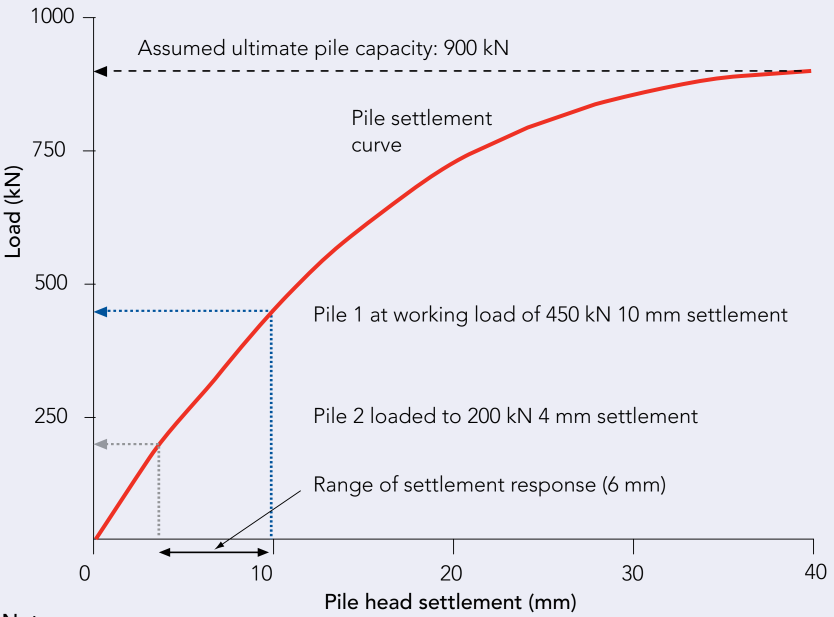
Pile loads should consider both the live and dead building loads, plus any down-drag loading that may be imposed on the piles as a result of settlement of the surrounding ground.

Having calculated loads acting on individual piles, it is generally the case that these will be rationalised for practical design purposes. Typically, pile loads will be rounded up to the nearest 25 or 50 kN by the pile designer. In extreme cases, all piles may be designed to carry the highest pile load, leading to significant over-design in areas of more lightly loaded walls or columns. The building loads may in some cases be rounded up by the structural engineer before being passed on the pile designer.

A more detailed load take-down could lead to improvements in efficiency and savings in pile costs by allowing use of a greater range of pile sizes and/or lengths. In terms of assessing likely differential settlements between piles, an understanding of the actual pile loads is important.

As illustrated in Figure 28 for any given pile size in given ground conditions, higher loads will lead to greater amounts of settlement.

If there is a variation in the pile loads but all the piles are designed on the basis of the highest loaded pile, the piles supporting smaller loads will tend to settle less. This in itself could lead to problems with differential settlements, hence as well as being inefficient in terms of pile sizes used, it is not necessarily conservative to design all piles to the highest load.




Note

The settlement response of each pile will vary as a result of the loading applied, regardless of any differential response cause by natural variability of the ground conditions.

If column loads vary, and all piles are designed to carry the highest column load, there can be a significant variation in settlement response. Assuming a uniform ground response, Pile 1 operating at working load settles by 10 mm, while the more lightly loaded Pile 2 settles by 4 mm.

**Figure 28** Variation in settlement with load for a given pile design.

## 5.13 Summary

### 5.13.1 Understanding ground conditions

The understanding of ground conditions and associated ground hazards is critical to designing effective foundations.

There are many hazards that can have significant impact on both design and construction of piled foundations. If these are not identified and understood, they can result in delays to projects, with impacts on cash flow and increased construction costs and additional costs due to the need for additional investigation and design, remediation and repair or reconstruction costs.

### 5.13.2 Tolerance to foundation movements

In order to design piles, an understanding of the typical requirements of the house in terms of total and differential movements is required.

The assessment of pile behaviour should consider movements under applied building loads, and also from ground-related effects such as subsidence and heave. When assessing pile head settlements, account should be taken of the elastic compression of piles, which may be particularly significant for more slender pile sections. Pile compression/shortening will generally occur during construction as the building load is applied, and will usually be largely complete prior to the application of any surface finishes.

Typical low-rise housing in the UK has a relatively low tolerance to differential movements, as discussed in section 2 of this guide. Acceptable limits of angular distortions, with reference to published studies and recommendations of current design codes, can be used to set acceptable settlement limits for pile design. For typical low-rise housing, where spacing between piles is typically between 3 m and 5 m, piles designed for maximum total settlement of the order of 10 mm under working load can generally be considered acceptable. This will generally limit maximum differential settlements between adjacent piles to the order of 5 mm.

### 5.13.3 Developing efficient designs

To realise the most efficient design to meet the settlement criteria that are specified, the following should be considered:

- The 'design situation' should be carefully reviewed and addressed in the design, requiring consideration of potential ground hazards and the impacts of any changes to site conditions, such as raising of ground levels or changes in groundwater conditions.
- More detailed information about design parameters will reduce uncertainty and invariably lead to less conservative and more efficient design.
- A detailed assessment of loads on individual piles should be undertaken, and where the loads between piles vary, rationalisation of pile sizes and/or lengths should be minimised where possible.
- Pile testing will provide benefits by allowing greater confidence in the understanding of the pile behaviour, allowing reductions in the factors of safety that are applied to the pile design and corresponding reductions in the size and/or length of the piles required.

# 6 Environmental impact of foundation solutions

## 6.1 General

There is increasing emphasis on environmental considerations when planning and implementing housing developments. These include:

- tighter regulation of the development of contaminated sites, with more stringent regulatory frameworks and increasing numbers of developments on more marginal sites which may be affected by contamination
- increased costs and regulation in relation to waste generation and disposal
- greater importance of achieving and demonstrating 'sustainability'
- more stringent enforcement of the regulations relating to noise and vibration that is generated.

### 6.1.1 Key planning considerations and drivers

There are a number of statutory publications and other guidance documents relating to the various 'environmental' concerns that may influence the design and selection of foundations for housing developments. These include:

- Building Regulations and Approved Documents A and C
- Current environmental regulations and legislation
- Code for Sustainable Homes.

The potential implications of not complying with regulations can include criminal prosecution. There may also be significant impacts on process and programme efficiency if compliance with regulations is not considered at an early stage in the development.

The key requirements of relevant aspects of the Approved Documents to the Building Regulations are presented in Appendix B.

The environmental impacts of foundations can be broadly considered in terms of the following:

- contamination
- waste
- vibration, noise and air quality.

This section considers these in turn and includes a discussion of the key regulations, legislation, and guidance documents, details of which are presented in a series of summary tables in Appendix I.

Two other important issues which relate to the sustainability of house foundations include:

- 'embodied carbon' in house foundations
- use of 'geothermal piles' in low-rise housing.

The embodied carbon is a measure of the volume of $CO_2$ emitted to produce an end product, in this case a house foundation. This has been considered in relation to three different types of foundations, and the comparative embodied carbon of these estimated.

The potential for the use of geothermal piles in conjunction with a ground source heat pump to provide a proportion of the heating demands of a house, and the potential application of this technology to meet the higher Code Levels of the Code for Sustainable Homes has also been considered.

## 6.2 Contamination

Foundation construction can lead to the generation of contaminated spoil and potentially the creation of pathways for the migration of contamination that would otherwise not exist. The choice of foundation type and construction procedures, and the feasibility of ground improvement, may be influenced by the need to limit the potential for mobilisation of contamination. The contaminated arisings that can be generated during foundation construction may require treatment and/or disposal as discussed in section 6.3.

The Environment Agency's Model Procedures (CLR11) forms the framework for the assessment of sites of land affected by contamination. Guidance on the practical application of good practice within this framework relevant to the housebuilder is provided in R&D Publication 66,[28] which has been prepared by the Environment Agency and the NHBC.

On all sites, consideration should be given to potential effects of the proposed foundation solution and its construction. The key concerns, which are illustrated in Figure 29, include:

- creation of preferential flow paths, allowing contaminated groundwater and leachates to move downwards through impermeable layers into underlying groundwater or between permeable horizons in a multilayered aquifer
- the breaching of impermeable covers ('caps') by foundation construction or penetrative ground improvement, allowing surface water infiltration into contaminated ground (thus creating leachate) or allowing the escape of landfill or ground gases
- contaminated arisings being brought to the surface by foundation construction, with the risks of subsequent exposure to site workers and residents, and the need for appropriate handling
- the effects of aggressive ground conditions on materials used in foundations – which may affect the structural integrity or performance of the foundation

- movement of contaminated materials downwards into an aquifer during construction/installation of piles or penetrative ground improvement works
- the potential for concrete or grout contamination of groundwater and any nearby surface waters. This is less commonly an issue, but may occur in highly permeable granular materials where groundwater is mobile.

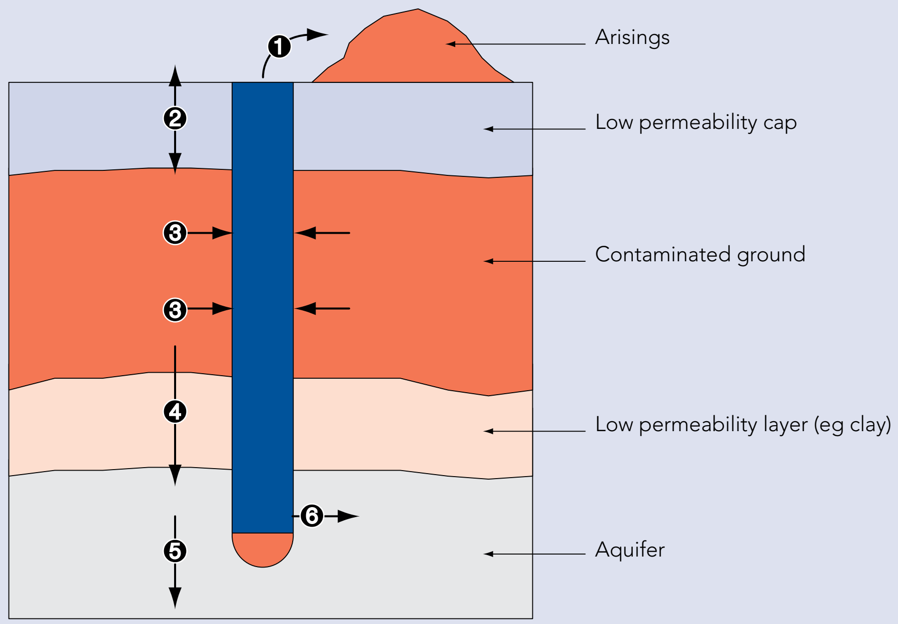
The potential impacts to groundwater are generally greater where the depth of ground treatment or foundations is greater. The Environment Agency publication *Piling and Penetrative Ground Improvement Methods on Land Affected by Contamination*[20] identifies the potential impacts that intrusive ground improvement and piling techniques can have on the environment and includes a recommended risk assessment framework.

A summary of the above guidance is also given in *Piling into Contaminated Sites* by the National Groundwater and Contaminated Land Centre[35], which gives a brief introduction to potential environmental hazards.

Guidance on the risks to groundwater and archaeology is also provided in the Environment Agency special report SC020074/SR.[36]

It is often the case that as long as an appropriate pile type is selected and workmanship and quality control measures are adequate, the potential risk posed by contamination can be acceptably managed. For particularly sensitive sites, it may be necessary to also demonstrate that piling provides the only feasible foundation option.

Of greater concern on sensitive sites is the use of certain penetrative ground improvement methods, such as stone columns, the intent of which can be to provide potential drainage pathways, which, although of benefit in terms of ground improvement, can lead to creation of pathways for contaminants and ground gases.




1. Contaminated arisings
2. Breaching cap allowing gas to escape or surface water to infiltrate potentially increasing leachate in contaminated ground
3. Effects of aggressive ground eg high sulphate and low pH
4. Creation of preferential flow paths (through low permeability layer)
5. Driving of contaminated material downwards into the aquifer (eg during installation of driven piles)
6. Concrete or grout contamination of groundwater or surface water.

**Figure 29** Key concerns in relation to contaminated land – foundation selection and design.

## 6.3 Waste

### 6.3.1 General

Minimisation of the volume of arisings generated should be considered when selecting foundations, particularly where there is little opportunity to reuse arisings within the development site, or where the arisings may be contaminated, requiring disposal at landfill.

Different types of foundations will generate more or less waste. The waste generated as a result of foundation construction can be significant, eg concrete trenchfill foundations can generate significant volumes of waste. In comparison, bored piles will generate a fraction of the volume of arisings, and driven replacement piles will typically create minimal volumes of arisings, with excavation usually limited to the areas of the pile caps.

The cost of waste disposal depends upon the nature of the arisings. Disposal costs will be particularly high on contaminated sites where arisings may require disposal in a licensed hazardous waste facility. Pre-treatment of the waste may also be required prior to disposal and this must be considered.

The UK government has set stringent targets to reduce landfill waste. The Landfill Tax was introduced in 1996 to encourage producers to generate less waste and to use more environmentally friendly methods of disposal.

Significantly reduced waste disposal costs can be achieved if a foundation type which produces limited arisings is adopted.

### 6.3.2 Site waste management plans

The Site Waste Management Plans (SWMPs) Regulations 2008 came into force in England in April 2008. Currently a site waste management plan must be prepared for all new construction projects valued at greater than £300 000, and, at the time of publication, the extension of these regulations to smaller developments was under discussion. In Northern Ireland, Scotland, and Wales, site waste management plans were not compulsory when this guidance was published, although their adoption was being considered.

The SWMP Regulations make it the developer's responsibility to ensure a SWMP is written, followed and updated during the project. Although the plan must be written at the construction design stage, it is a requirement of the regulations to maintain and update the plan throughout the later stages of construction.

Following the procedure of the plans will help to reduce site waste and manage the waste that is produced more effectively as well as helping to ensure compliance with other waste regulations, and even for sites where waste management plans are not compulsory it is recommended that they are used as a means of demonstrating best practice.

A SWMP sets out how building materials, and resulting waste, is to be managed during the project. The SWMP's purpose is to ensure that:

- building materials are managed efficiently
- waste is disposed of legally
- material recycling, reuse and recovery is maximised.

Minimisation of waste can contribute to the reductions in material costs, and also reduction in site traffic, which may be beneficial on sites in urban areas where this can be a particularly sensitive issue. The minimisation of site traffic will also limit haulage costs.

For details and useful templates for preparation of SWMPs, reference can be made to the NHBC Foundation document *Site Waste Management, Guidance and Templates for Effective Site Waste Management*.[37]

Other useful information is also provided by the Waste & Resources Action Programme (WRAP). The Net Waste Tool produced by WRAP, which is freely available on their website, can be used to assist with implementation of SWMPs.

## 6.4 Vibration, noise and air quality

### 6.4.1 Vibration and noise

The noise and vibration sensitivity of the area should be considered when determining the construction of foundations. Local authorities have the right to serve notices on contractors where noise problems arise. If a project is likely to have a significant impact on neighbours as a result of noise or vibration which may be generated, application for a 'prior consent' should be made with the local authority.

Noise and vibration can be of particular concern with piling installation. All piling methods will create noise and to some extent vibration, but issues are likely to be particularly acute with the use of driven piles.

Noise and vibration sensitivity of an area may limit the options available and the use of certain pile types may not be permitted. Any restrictions should be understood early on in the development process.

### 6.4.2 Air quality

Normal good working practice can readily manage problems with dust, ie by use of water sprays during sustained periods of hot and dry weather and the provision of wheelwash facilities to prevent any transfer of mud to roads.

On sites with contamination issues, odours emitted from arisings may also need to be taken into consideration.

Dust occurring as a result of piling or excavations may be of particular concern on some sites, and the minimisation of disturbance may be a key concern, eg if asbestos fibres are present within the made ground at a site.

## 6.5 Embodied carbon in house foundations

The amount of $CO_2$ emitted during construction of a project, measured in tons of $CO_2$, can be used as a measure of a project's environmental impact. The embodied carbon can be used as an absolute quantity, but, more usefully, can be used for comparing alternative design options.

Embodied carbon is associated with a number of factors, including the extraction of raw materials and their processing into construction materials, the transportation of the materials and plant to and from site, and the use of the construction plant during operation on site.

To investigate the amount of embodied carbon associated with different foundation solutions, a detailed assessment has been carried out for the relatively commonly used trenchfill foundations compared with driven and bored pile solutions. These assessments for a typical low-rise terraced house are presented in Appendix J.

The assessment concluded that:

- The majority of the embodied carbon is directly related to the volumes of material used; embodied carbon associated with transportation and installation is significantly less.
- Even shallow trenchfill foundations have greater embodied carbon than piled foundations as a result of the greater volume of concrete used. (Production of cement requires large amounts of energy and so produces a lot of $CO_2$.)
- Bored piles and driven piles have generally similar values of embodied carbon. The potential savings in materials use for driven piles is balanced to some degree by the increased embodied carbon for transportation.
- Given that the primary concern is the embodied carbon related to the volume of materials used, as well as reducing the amount of material required, reuse and recycling of materials are effective ways of reducing the embodied carbon of the foundations.

## 6.6 Use of geothermal piles for low-rise housing

Geothermal piles are defined as load-bearing piles with an in-built closed loop heat exchanger. When used in conjunction with a heat pump, geothermal piles can provide heating or cooling to a building by using the thermal mass of the ground as a heat source, and taking advantage of the high thermal storage capacity of concrete in the piles and the surrounding ground. A detailed study of the use of geothermal piles for low-rise housing is presented in Appendix K. This includes:

- A consideration of the potential for the use of geothermal piles to provide space and water heating, discussing both the performance of the geothermal pile as an exchanger of heat energy, and the implications of the temperature changes in the ground on the performance of the geothermal pile as a load-bearing foundation.
- Details of how the higher Code Levels of the Code for Sustainable Homes in relation to energy and carbon dioxide emissions can be more readily achieved by using geothermal piles, and comparison with biomass boilers and more conventional gas boilers.
- Contractual issues, including design responsibility and appointment of appropriately qualified individuals to provide advice on the potential use of geothermal piles.

The review has concluded that:

- Although there are limited studies, it appears that the load-bearing performance of geothermal piles is not adversely affected by the cyclic heating and cooling of piles, as long as the minimum temperature of the ground is not allowed to fall below critical levels.
- With regards to the Code for Sustainable Homes, Code Level 4 for energy and $CO_2$ emissions is likely to be achievable using geothermal piles. Because of the energy demands of the ground source heat pump (Fig. 30), it is unlikely that Code Levels 5 and 6 could be achieved, unless either the electricity grid is de-carbonised or on-site generation of renewable electrical energy can be used to power the heat pump.
- Geothermal piles confer a small installation cost saving compared with biomass boilers and can be considered to be a feasible option. Even though biomass boilers have been shown to offer slightly greater $CO_2$ savings when compared to an geothermal pile system, the Code Level that can be achieved is broadly equivalent for both systems.
- Care should be taken in the procurement of geothermal pile systems, particularly in assigning design responsibility.

Figure 30 shows the intallation of geothermal piles. The HDPE pipework has been installed with the reinforcement cage. The fluid filled pipe will be connected to the heat exchanger. Further details are provided in Appendix K.

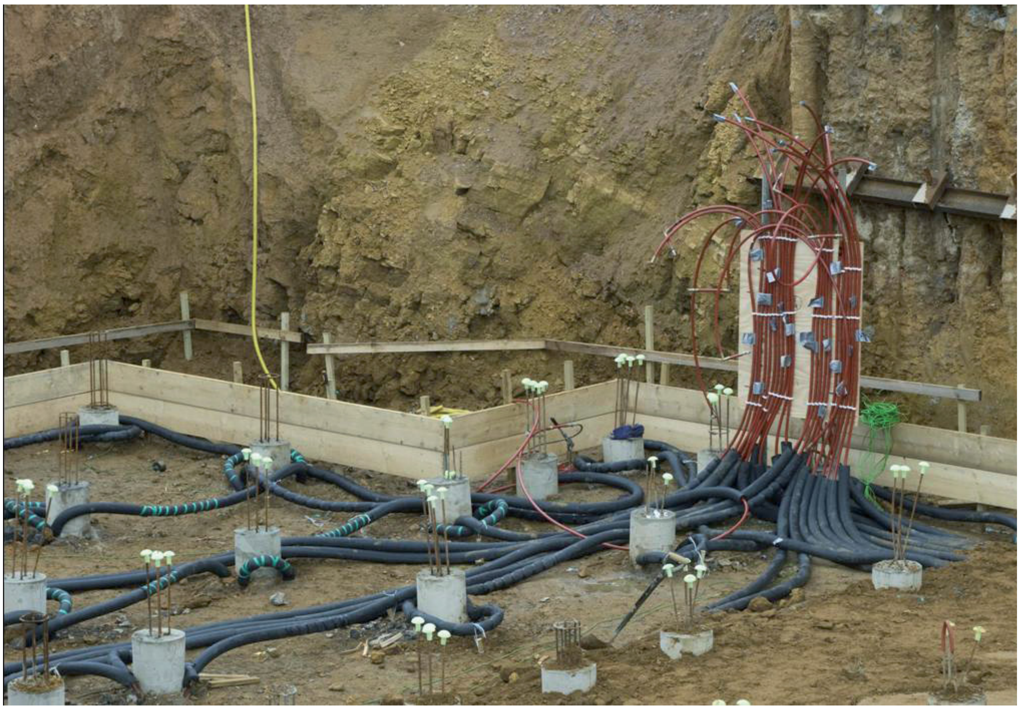


**Figure 30** Geothermal piles with heat exchange pipework.

# 7 Conclusions

## 7.1 Efficient design

The measure of the efficiency of foundation design can be considered in terms of direct and indirect savings. Direct reductions in foundation costs are apparent in terms of the amount of resources/materials used. However, savings can also be made in more indirect ways, for example by preventing foundation failures requiring remedial works, or by reducing waste generated by excavations.

The 'efficiency' of foundation design is difficult to quantify, but instances where efficiency could have been improved include situations where:

- foundations have not met the design requirements
- the type of foundation selected does not provide the best solution
- foundations have performed adequately, but have been 'overdesigned'.

This design guide has considered under these general headings the areas where increased efficiency can be realised. Of key importance are:

- an understanding and recognition of the criteria the building needs to meet in terms of its acceptable performance, and how this relates to the choice of foundation type and the design of the foundation once selected
- the undertaking of a thorough site investigation to identify ground hazards – to avoid 'failure', and to provide appropriate parameters for design – to avoid overdesign.

With increasing emphasis on environmental considerations when implementing housing development, the design guide has also considered the reduction of embodied carbon as a measure of increasing efficiency.

To achieve efficient designs, appropriate advice should be sought from qualified geotechnical specialists at all stages of the development.

## 7.2 Selection of foundations

The selection of the most suitable foundations for a low-rise house will be dependent on a number of factors. These include:

- the tolerance to movements of the supported structure
- the particular ground conditions at the site
- the direct and indirect costs of construction
- the statutory or regulatory requirements that may be imposed.

Selection of the best solution needs to consider the cost of materials and transport, the operational costs and timescales of installation, the costs and timescales of any temporary works that may be required, and the disposal costs of waste arisings. The site constraints and limits that these will impose on operations will impact on the programme and construction cost.

The foundation solution selected may also be influenced by health and safety considerations and environmental regulations associated with noise and vibration and contamination. The potential for creation of contamination pathways in particular should be considered, and this can be of greater concern when using piled foundations as opposed to shallow foundations. However, it is often the case that as long as an appropriate pile type is selected the potential risk posed by contamination can be acceptably managed.

## 7.3 Site investigation

Critical to the development of efficient foundation designs is a well-designed and implemented site investigation, which:

- identifies and quantifies the ground hazards and minimises the uncertainty in the ground
- provides design parameters to avoid excessive over-design of foundations.

Well-timed and phased investigations will allow ground hazards to be effectively targeted. To gain best value from investigations, an initial site appraisal including a desk study and site walkover survey should be completed before the intrusive ground investigation works are undertaken.

The ground risks will generally reduce with increased investment in site investigation. The potential reductions in construction risks and costs should be considered when budgets are set for the site investigation.

Inadequate site investigations can result in the adoption of more conservative design parameters and the need for higher factors of safety as a result of the greater level of uncertainty, increasing the overall cost of the foundations. The results of the survey of current practice indicated that the designers of piled foundations often found the quality of ground investigation information provided to be insufficient to allow them to develop the most efficient designs.

## 7.4 Acceptable foundation movements

A prerequisite to the design of efficient foundations is an understanding of the performance requirements of the buildings and associated access and services connections. Total movements, tilt movements and differential movements should all be considered, although it is differential movements, where parts of the structure move relative to others, which are likely to be the most critical.

If the performance requirements of the supported structure and the load settlement response of the foundation are well-understood, design efficiency can be optimised. The performance requirements for low-rise housing can be investigated based on published studies. Such studies generally have aimed to define and understand the acceptable movements for low-rise structures in relation to the onset of aesthetic, serviceability, and structural damage, and have presented general conclusions and proposed limiting movement criteria.

However, the use of published criteria has to be used with some caution. The acceptable movements of a given house will be dependent on the particular superstructure design and the materials used, and as such are difficult to estimate with any certainty. In general terms, UK low-rise housing has a very low tolerance to differential movements, largely as a result of the brittle nature of the masonry and surface finishes that are typically used. The masonry is likely to be the critical consideration both for houses of traditional load-bearing masonry construction and for those with concrete, steel, or timber frames where masonry panels are used.

The use of less brittle construction materials for the building superstructure, such as softer mortar mixes and less brittle finishes, and the incorporation of movement joints or bed reinforcement in masonry walls could allow greater levels of movement to be accommodated before the onset of damage.

The susceptibility to damage of the building as a result of differential movements can also be reduced by increasing the stiffness of the foundation, which will tend to redistribute the loads and reduce the differential movements which affect the superstructure.

## 7.5 Pile design

### 7.5.1 Key requirements

Foundations should be designed so that:

- there is an adequate factor of safety against foundation failure
- the settlements at working loads are within acceptable limits.

There is much guidance on selection of factors of safety, as embodied in codes of practice, etc., but less guidance on what settlement criteria may be appropriate under working load conditions. Based on the review of published information a reasonable approach for the design of piled foundations supporting low-rise housing would be to limit pile total settlements to the order of 10 mm under working loads. However, this should not be considered as absolute and there are circumstances where this may not be appropriate, for instance where there is significant variation in ground conditions leading to a significant range in pile settlement behaviour.

### 7.5.2 Design assumptions

To enable efficient design it is important that ground conditions are well-understood and that their impact on the design are accounted for. Consideration should be given to existing and potential ground hazards as well as the impacts of any changes which may occur to the site conditions both during construction and within the lifetime of the development. As well as ground movements under applied building loads, other causes of movements linked to ground subsidence or heave mechanisms must be considered in terms of their impact on foundation design.

The selection of representative parameters for ground strength and stiffness will depend on the quality of the site investigation available. The design of piles should focus on estimation of settlements as well as on safe load capacities, and this should be considered when scoping ground investigations.

It is important that accurate pile loads are used if design efficiency is to be achieved; where loads are rounded up by either the structural engineer providing the loading information or the pile designer rationalising the pile loads, designs will tend to be less efficient.

Where the pile and ground beam are designed by different engineers, both designers should be aware of each other's design assumptions so that the designs are consistent. It is important to consider the combined behaviour of the piles and ground beams as the stiffness of the overall system can allow redistribution of loads, which will tend to reduce differential movements.

### 7.5.3 Testing

Pile testing can be undertaken to validate pile designs, for quality control purposes during or following construction, or as a basis for the pile design.

Pile testing using static load tests of both preliminary trial piles and working piles can provide a number of benefits, foremost of which is the understanding of the load settlement response of the piles under working load. Static load tests to failure of preliminary trial piles can form the basis of the pile design, and can allow more efficient designs to be developed. Reduced factors of safety can be adopted if the load settlement response of a pile is verified by load tests on working piles.

Dynamic pile tests can be used to allow predictions to be made of the pile performance under static load. These tests are quicker and cheaper than static load tests, but care must be taken in the interpretation of the results.

### 7.5.4 Pile construction

The pile design must consider the practicalities of construction, including site access and available working space, and the effects of pile construction on adjacent foundations and buildings. The implications of obstructions, unstable ground and groundwater levels on pile installation must be addressed.

## 7.6 Environmental impacts

The design guide has considered environmental impacts in terms of sustainability under various headings. Some of the key conclusions are:

### 7.6.1 Waste

The minimisation of waste generation has important implications in relation to the sustainability of developments, and is likely to become increasingly important in the future. Minimisation of the volume of arisings from foundation construction can be of particular importance where there is little opportunity to reuse arisings, or where the arisings may be contaminated requiring disposal at landfill.

### 7.6.2 Embodied carbon

The amount of $CO_2$ emitted during the construction of foundations provides a measure of environmental impact. A comparative assessment of the embodied carbon of deep trenchfill foundations and piled foundations has demonstrated that the majority of the embodied carbon relates to the volumes of steel and concrete used, and even shallow trenchfill foundations will generally have significantly higher embodied carbon than piled foundations. As well as by reducing the amount of material required, reuse and recycling of materials are effective ways of reducing embodied carbon.

### 7.6.3 Use of geothermal piles for low-rise housing

The exploitation of ground source heat using energy loops incorporated into piled foundations in conjunction with a heat pump can provide heating to the house by using the thermal mass of the ground as a heat source. A study of the potential application of this technology has shown that its use could provide a significant proportion of the heating demands of low-rise housing and help to achieve higher Code Levels of the Code for Sustainable Homes in relation to energy and $CO_2$ emissions.

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# APPENDIX A

# Details of survey of current practice

## A1 Details of survey

A survey of various stakeholders was undertaken to gain feedback on current attitudes and practice and to identify areas of general concern. A series of web-based questionnaires were issued to representatives of the following organisations:

- Association for Consultancy and Engineering (ACE)
- Association of Geotechnical and Geo-environmental Specialists (AGS)
- British Geotechnical Association (BGA)
- Chartered Institute of Building (CIOB)
- Federation of Piling Specialists (FPS)
- Home Builders Federation (HBF)
- Institution of Civil Engineers (ICE)
- Institution of Structural Engineers (IStructE)
- Local Authority Building Control (LABC)
- National House-Building Council (NHBC).

Approximately 450 individuals took part in the surveys, including a cross-section of contractors, consultants, housebuilders, property developers, ground investigation specialists, building control engineers, and regulators.

The questionnaires were split into a number of sections dealing with selection of foundation options, ground investigation and pile design and testing.

Some specific questions were only addressed to specific groups, and others asked more widely to gauge variations in attitudes between different groups of stakeholders.

## A2 Current attitudes

### A2.1 Issues associated with ground investigations

Areas of concern with current practice were identified, including the following:

- Insufficient site investigation information to develop efficient designs. Basic information is often not provided to designers of piled foundations, including locations of boreholes and trial pits, and basic testing information such as in situ SPTs and laboratory strength tests. Investigations frequently were extended to insufficient depth. Table A1 presents details of how often these issues were reported.
- Over two-thirds of pile designers believed that insufficient consideration is usually given to the effects of soil heave or down-drag, leading to either conservative assumptions in the design or no allowance made.
- There was a general concern that appropriately skilled geotechnical advisors are often not appointed to provide advice, or that the scope of appointments was too limited. The brief of consultants employed to scope site investigations and provide advice concerning ground risks does not always account for both the geotechnical and the geo-environmental risks that could affect a development.

**Table A1**

**Frequency of problems with ground investigation**

How frequently do you encounter the following problems with ground investigation used for pile design?

| | Over 50% | 25–50% | 10–25% | Less than 10% | Never |
|---|---|---|---|---|---|
| Investigation to insufficient depth | 33.3% (29) | 32.2% (28) | 25.3% (22) | 9.2% (8) | 0.0% (0) |
| Insufficient laboratory test data | 29.5% (26) | 40.9% (36) | 21.6% (19) | 6.8% (6) | 1.1% (1) |
| Insufficient in situ test data, eg standard penetration testing (SPTs) | 11.4% (10) | 47.7% (42) | 31.8% (28) | 9.1% (8) | 0.0% (0) |
| Exploratory holes not co-ordinated and levelled | 55.1% (49) | 20.2% (18) | 18.0% (16) | 6.7% (6) | 0.0% (0) |
| No exploratory hole location plan | 10.1% (9) | 22.5% (20) | 27.0% (24) | 33.7% (30) | 6.7% (6) |
| Insufficient density of investigation | 17.0% (15) | 39.8% (35) | 37.5% (33) | 5.7% (5) | 0.0% (0) |
| Poor-quality logging | 11.4% (10) | 34.1% (30) | 39.8% (35) | 14.8% (13) | 0.0% (0) |

Note: Numbers in brackets indicate number of responses

### A2.2 How do piles design approaches compare to code requirements?

The current practice for pile design for low-rise housing was assessed with the following conclusions:

- The design approach generally used to determine acceptable pile working loads is the application of an overall factor of safety against ultimate failure. The value of overall factor of safety is generally between 2 and 3. The assumption is typically made that for piles designed on this basis the settlements will be within acceptable tolerances for the buildings carried.
- Predictions of pile settlement are unlikely to be made, and are generally not required since limiting values are rarely specified; it is generally implicitly assumed that settlements will be within acceptable limits.
- Less than 30% of designers said that they currently use Eurocode 7 for pile design for low-rise housing.
- There is a perception that the use of partial factors adopted within the Eurocode 7 approach can confuse and obscure the really critical issues, and is considered by some too complex and overly academic in its approach.
- Further simple guidance on principles of Eurocode 7 design would be useful for the housing sector.

### A2.3 Pile testing

With regard to pile testing, the consensus was that pile tests are costly and time consuming, although they are a useful verification tool and testing should ideally be carried out on all projects:

- It was generally considered that an appropriate amount of static load testing of preliminary piles can lead to savings on larger jobs, but is less significant on small jobs. They do not appear to be very common for low-rise housing developments.
- Dynamic load tests were commonly used. These appear to be seldom correlated to static load tests, and the interpretation of results is generally not well-understood.

### A2.4 Additional guidance for pile design for low-rise housing

When asked about additional guidance that would be useful for the low-rise housing sector, the following comments were made:

- More explicit guidance on the tolerances of houses to differential settlement would be useful, together with guidance concerning generally acceptable levels of settlement, with reference to different types of house construction and their relative sensitivity and tolerance of settlement.
- The methods by which down-drag and heave effects should be calculated and designed for would be helpful.
- Guidance concerning the minimum levels of ground investigation for piled foundations would be useful.
- The appropriate use of geotechnical specialists should be emphasised for all stages of low-rise housing projects, from initial site appraisal to detailed design and construction.
- The benefits of investing in site investigation should be clarified and emphasised.

## A3 Survey of NHBC engineers

Supplementary to the main survey a further questionnaire was prepared and distributed to NHBC engineering and surveying staff. This included a number of questions relating to the following:

- current practice of pile design and procurement
- assessment of suitability of proposed foundations for developments
- adequacy of ground investigation information
- pile testing considerations.

The main responses were:

### A3.1 Common design and construction issues identified

- Negative skin friction (down-drag) is often ignored by pile designers.
- Heave effects often overlooked in areas away from the south-east of the UK where heave is perhaps less commonly an issue.
- Piles often not designed to carry lateral loading and moments because of construction tolerances.

### A3.2 Installation issues identified

- Driven pre-cast piles are relatively brittle, and can break if they are driven too hard where there is high resistance in the ground, or where incorrect methods of cutting down to cut off levels are used.
- Steel tubes of insufficient length were found to be problematic due to the difficulties involved with the splicing or welding on additional sections of steel tube.
- Buckling during driving of driven steel tube piles with very slender aspect can be critical.

### A3.3 Problems with unforeseen ground conditions

- The failure to obtain adequate ground investigation information to the appropriate depth was reported to be a common problem for low-rise housing developments.

# APPENDIX B

# Summary of Building Regulations requirements

## B1 Building Regulations Approved Document A

The Building Regulations Approved Document A[B1] deals with the following requirements contained within the Building Regulations 2000 (as amended by SI 2001/3335, SI 2002/440, SI 2002/2871 and SI 2003/2692):

### B1.1 Loading

"A1 (1) The building shall be constructed so that the combined dead, imposed and wind loads are sustained and transmitted by it to the ground:

a) safely; and

b) without causing such deflection or deformation of any part of the building, or such movement of the ground, as will impair the stability of any part of another building.

(2) In assessing whether a building complies with sub paragraph (1) regard shall be had to the imposed and wind loads to which it is likely to be subjected in the ordinary course of its use for the purpose for which it is intended."

### B1.2 Ground movement

"A building shall be constructed so that ground movement caused by:

a) swelling, shrinkage or freezing of the subsoil; or

b) landslip or subsidence (other than subsidence arising from shrinkage, in so far as the risk can be reasonably foreseen), will not impair the stability of any part of the building."

## B2 Building Regulations Approved Document C

The Building Regulations Approved Document C[B2] deals with the following requirements contained within the Building Regulations 2000 (as amended by SI 2001/3335, SI 2002/440, SI 2002/2871 and SI 2003/2692).

### B2.1 Site preparation and resistance to contaminants and moisture

#### Preparation of site and resistance to contaminants

"C1 (1) The ground to be covered by the building shall be reasonably free from any material that might damage the building or affect its stability, including vegetable matter, topsoil and pre-existing foundations.

(2) Reasonable precautions shall be taken to avoid danger to health and safety caused by contaminants on or in the ground covered, or to be covered by the building and any land associated with the building.

(3) Adequate subsoil drainage shall be provided if it is needed to avoid:

a) the passage of moisture to the interior of the building;

b) damage to the building, including damage through the transport of water borne contaminants to the foundations of the building.

(4) For the purpose of this requirement, 'contaminant' means any substance which is or may become harmful to persons or buildings including substances which are corrosive, explosive, flammable, radioactive or toxic.

### Resistance to moisture

C2 The floors, walls and roof of the building shall adequately protect the building and people who use the building from harmful effects caused by:

a) ground moisture;

b) precipitation and wind driven spray;

c) interstitial and surface condensation; and

d) spillage of water from or associated with sanitary fittings or fixed appliances."

## Appendix B references


B1 ODPM (2004). Building Regulations Approved Document Part A – Structure.

B2 ODPM (2004). The Building Regulations Approved Document Part C – Site preparation and resistance to contaminants and moisture.

# APPENDIX C

# Acceptable movements for low-rise buildings

## C1 Criteria used for damage classification

The classification system widely adopted for low-rise housing is that developed by Burland and Wroth,[C1] which is reproduced in BRE Digest 251.[C2]

The categories of damage considered within these criteria are as follows:

- **Aesthetic**: Damage including but not limited to cracking of internal and external finishes, tiles, etc.
- **Serviceability**: Damage leading to deterioration in weather-tightness, sticking of doors and windows, problems with incoming services, etc.
- **Structural:** Damage requiring major remedial action or causing structural instability.

These three categories are often further sub-categorised in terms of severity of damage as reproduced in Table C1.

Table C1

**Classification of visible damage – extract from BRE Digest 251** *Assessment of Damage in Low Rise Buildings*[C2]

| Category of damage | | Description of typical damage *(ease of repair in italics)* | Approximate crack width |
|---|---|---|---|
| 0 | Aesthetic | Hairline cracks or less than about 0.1 mm width are classed as negligible *(no action required)* | Up to 0.1 mm |
| 1 | | Perhaps isolated slight fracturing in building. Cracks rarely visible in external brickwork *(fine cracks up to 1 mm width can be treated easily using normal decoration)* | Up to 1 mm |
| 2 | Serviceability | Cracks not necessarily visible externally *(cracks can be filled easily. Recurrent cracks can be masked by suitable linings)*<br>Some distortion to doors and windows which may stick slightly *(some external re-pointing may be required to ensure weather-tightness)* | Up to 5 mm |
| 3 | | Doors and windows sticking. Service pipes may fracture. Weather-tightness often impaired *(cracks will require some opening-up and can be patched by a mason. Re-pointing of external brickwork and possibly a small amount of brickwork to be replaced)* | 5–15 mm or several, each up to 3 mm |
| 4 | Structural | Window and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably. Some loss of bearing in beams. Service pipes disrupted *(extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows)* | 15–25 mm depending on number |
| 5 | | Beams losing bearing, walls leaning badly and requiring shoring. Windows broken with distortion. Danger of instability *(requires a major repair, involving partial or complete rebuilding)* | Usually greater than 25 mm, depending on number |

## C2 Previously derived deflection criteria

Numerous studies have been undertaken which have aimed to define acceptable levels of differential movements for structures. These include the following:

- Skempton and MacDonald[C3]
- Meyerhof[C4]
- Polshin and Tokar[C5]
- Bjerrum[C6]
- Burland and Wroth[C1]
- Burland, Broms and de Mello[C10]
- Burland.[C11]

Table C2 presents a summary of the conclusions of these studies, together with guidance given in codes and standards that have been derived from them.

These studies generally define differential movements in terms of either 'angular distortions' or 'deflection ratios' that can be accommodated by structures. The meaning of the terms angular distortion and deflection ratio are defined in Figure C1.

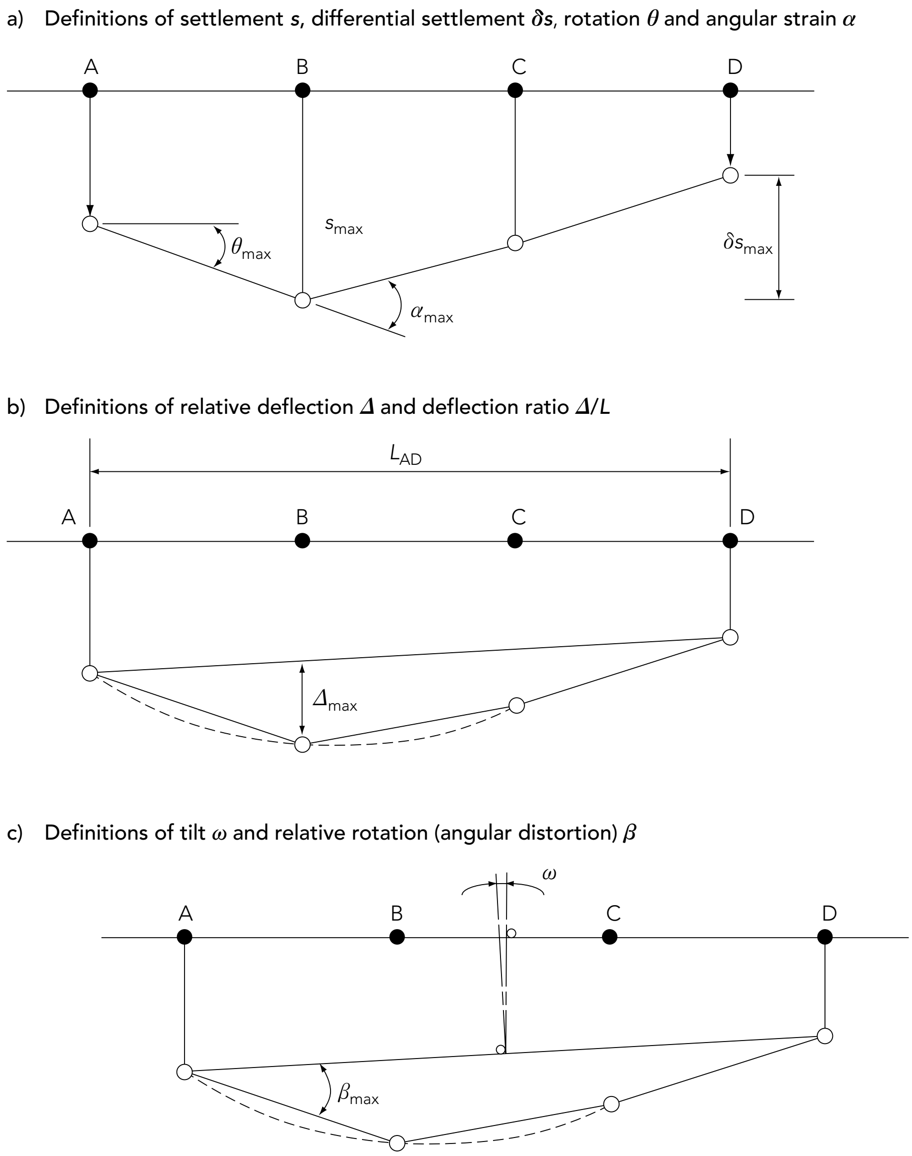



**Figure C1** Definitions of foundation movement (from Eurocode 7).

**Table C2**

**Summary of various deflection and angular distortion criteria**

| Publication | Form of construction | Deflection ratio Δ/L (for onset of visible cracking) | | Angular distortion β | |
|---|---|---|---|---|---|
| | | Sagging | Hogging | Serviceability | Structural |
| Skempton and MacDonald 1956 [C3] | Framed buildings | – | – | 1/300* | 1/150 |
| | Reinforced load-bearing masonry | – | – | 1/300* | 1/150 |
| | Maximum differential settlements should be limited to 25 mm<br>Maximum total settlements should be limited to 40 mm for isolated foundations and 40-65 mm for raft foundations | | | | |
| Meyerhof 1956[C4] | Framed buildings | – | – | 1/500 | 1/250 |
| | Reinforced load-bearing masonry | – | – | 1/500 | 1/250 |
| | Unreinforced load-bearing masonry | 1\2500 | – | – | – |
| Polshin and Tokar 1957[C5] | Framed buildings | – | – | 1/500† | 1/200 |
| | Reinforced load-bearing masonry | – | – | 1/500† | 1/200 |
| | Unreinforced load-bearing masonry | L/H<3; 1/3500 to 1/2500 | – | – | – |
| | | L/H<5; 1/2000 to 1/500 | | | |
| Bjerrum 1963[C6] | Framed buildings | – | – | 1/500 | 1/150 |
| | Reinforced load-bearing masonry | – | – | 1/500 | 1/150 |
| | Unreinforced load-bearing masonry | – | – | – | – |
| Burland and Wroth 1975[C1] | Unreinforced load-bearing masonry | L/H = 1; 1/2500 | L/H = 1; 1/5000 | – | – |
| | | L/H = 5; 1/1250 | L/H = 5; 1/2500 | | |
| Institution of Structural Engineers (1989) [C7] | Framed buildings | | | 1/500 | 1/150 to 1/280 |
| | Unreinforced load-bearing walls | 1/1250 to 1/3500 | 1/2500 to 1/5000 | | |
| Eurocode 7 (see guidance in Appendix H) | | | | 1/500 (sagging) | 1/150 (sagging) |
| | | | | 1/1000 (hogging) | 1/300 (hogging) |
| | Total settlements up to 50 mm often acceptable, larger settlements may be acceptable provided the relative rotations remain within acceptable limits and provided that there are no problems with services connections or tilt | | | | |
| Terzhaghi et al (1996)[C8] | Most buildings can tolerate 20 mm differential settlement between columns; 25 mm total settlement is a safe guide for buildings on isolated pad footings; 50 mm total settlements for raft type foundations | | | | |
| BS 5628 Code of practice for use of masonry[C9] | Final deflections should not exceed 1/250; considering internal finishes – limiting deflections should not exceed 1/500<br>Maximum allowable settlements of 25 mm | | | | |

* Allowable settlements were based on onset of cracking.

† Acceptable angular distortions on end bays of 1/1000 are suggested due to greater susceptibility to the more critical 'hogging' mode of movement.

'Angular distortion' is defined as the differential settlement between two points divided by the distance between them less the tilt of the whole building, and 'deflection ratio' is defined as the ratio of the central deflection (D) and the equivalent beam length (l), and is a measure of curvature of the wall element, as illustrated in Figure C1.

## C2.1 Skempton and MacDonald

Skempton and MacDonald[C3] undertook a review of published literature and case studies with the aim of empirically deriving serviceability criteria with regards to allowable settlements between support positions.

They examined 98 case histories to aim to develop an understanding of the allowable settlements relative to level of building damage in terms of both the allowable total and differential settlements.

A limit of angular distortion of 1/300 was concluded as a serviceability criterion for both load-bearing walls and masonry infill panels in framed buildings.

However, there were limitations to the study, particularly with regard to low-rise housing, insofar as:

- Over half of the structures reviewed exhibited no sign of damage.
- Only a small sub-set of the buildings assessed were low-rise residential.
- There were significant variations in construction methods within the dataset (including houses in different countries, using different codes and materials).
- No consideration was given to the aspect ratio of the affected parts of the structure.
- Damage was categorised in terms of 'cracking' and 'structural damage', and not strictly in terms of the onset of serviceability issues.

The conclusions of Skempton and MacDonald[C3] are presented in terms of angular distortion, which essentially relates to shear deformations, not bending type deformations. This gives it application for framed structures or walls with very small aspect ratios where shear failure is likely to be a controlling mechanism of deformation. Where unreinforced masonry walls are used, application of Skempton and MacDonald's[C3] suggested angular distortion criteria limits may lead to excessive cracking.

## C2.2 Bjerrum, Meyerhof, and Polshin and Tokar

Limits of acceptable angular distortions were also developed by Meyerhof,[C4] Polshin and Tokar[C5] and Bjerrum,[C6] as shown in Table C2. The definition of angular distortion varies between authors, and as Boone[C12] concluded, the rigid body tilt was usually neglected in these earlier studies.

Because of the limitations highlighted above, the other options available, using a more theoretical approach were investigated, as discussed below.

Polshin and Tokar[C5] also considered the case of unreinforced masonry construction, and introduced the idea of limiting tensile strain as a controlling influence of movement tolerances, and established criteria for the deformation in terms of deflection ratios.

## C2.3 Limiting tensile strain approach (Burland and Wroth/Burland)

### Methodology

Burland and Wroth[C1] developed the ideas of Polshin and Tokar,[C5] that the onset of visible cracking in a given material is linked to a limiting tensile strain ($\varepsilon_{lim}$), and developed an approach for its application, where the behaviour of load-bearing masonry walls was idealised as a deep beam, subject to a deformation developed at the beam centre. The 'deflection ratio' was used to define the differential movements, which is directly related to the curvature of the wall, or 'beam element'. The onset of visible cracking in a given material was linked to a limiting tensile strain.

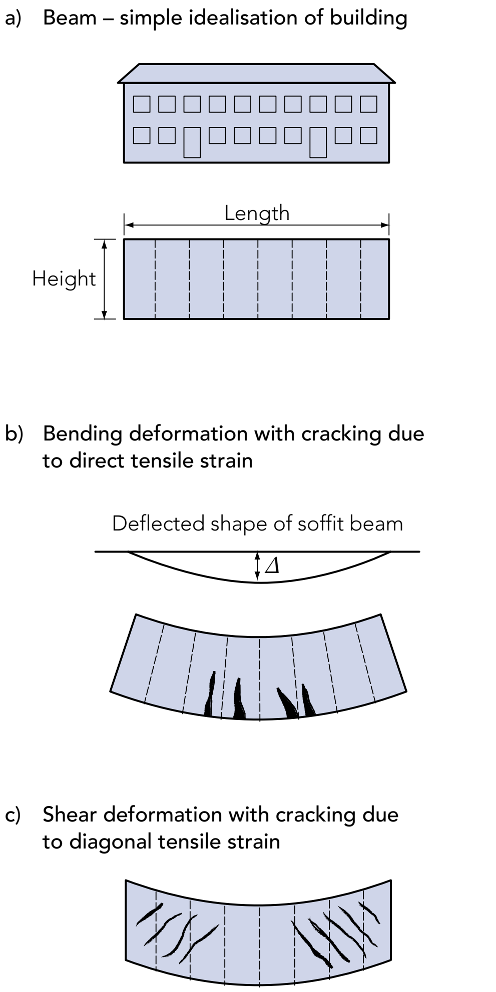



**Figure C2** Cracking of a simple beam in bending and in shear after Burland and Wroth.[C1]

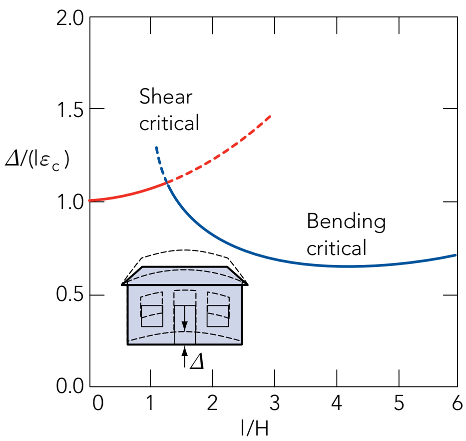



**Figure C3** Threshold of damage (for hogging) of load-bearing walls, with bending neutral axis at the wall base and ratio of E/G = 2.6 (after Burland and Wroth[C1]).

The method assumed that a given structure can be idealised as a simple beam spanning two supports. Under various failure mechanisms, the beam either hogs or sags to the resulting ground profile, and cracking develops both in the extreme fibres due to bending and in the main body of the element due to shear deformation. This is illustrated in Figure C2.

This idealisation assumes that differential movements lead to the development of a sagging or hogging profile in the wall, with cracking developing from the bottom upwards in the case of sagging in Figure C2. This behaviour is reflected in results observed in many studies in this field, and much like a simple beam, the limiting deflection for initial cracking will depend on the span-to-depth relationship of a structure.

If we assume a centrally loaded beam failing in a sagging mechanism, the maximum deflection would occur at the centre of the beam.

## Critical strain values

The approach by Burland and Wroth[C1] developed the equations for the bending of a deep beam and included consideration for a 'critical strain' that relates to the onset of cracking due to deformation. Based on an assumed value of critical strain of 0.03%, they related this to the aspect ratios of the wall elements (or beams). This enabled threshold levels for damage to be developed for a given critical strain value, which showed that shear is the critical mode of deformation for small aspect ratios, with bending becoming critical at larger aspect ratios. This is illustrated in Figure C3.

It was generally concluded based on the analysis, and in agreement with observations of real deformations, that a 'hogging' deformation is more critical than a 'sagging' deformation. The reason for this is that the tensile restraint that acts at the base of a masonry panel built on a reinforced ground beam or foundation is not provided in the hogging mode as the tensile failure occurs at the upper edge of the wall panel.

### Allowance for lateral strain

Burland[C11] developed his earlier critical strain methodology to enable lateral strains to be accounted for, and related deflection ratio and horizontal tensile strains to a variety of damage categories, based on a hogging type failure. This is reproduced in Figure C4.

This inclusion of lateral strains as well as vertical movements has found particular application for cases where tunnelling has been undertaken in proximity to existing structures, where both horizontal and vertical components of ground movements occur.

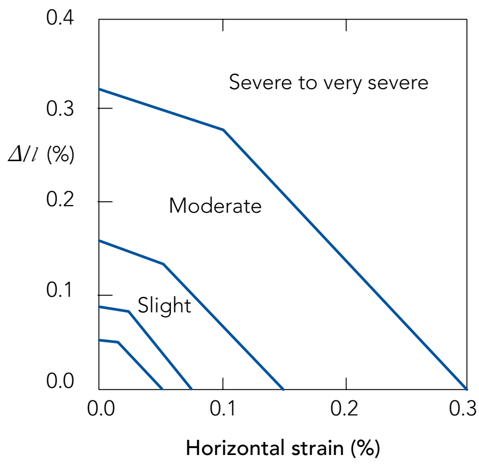



**Figure C4** Relationship of damage category to deflection ratio and horizontal tensile strain for hogging at l/H = 1 (from Burland[C11]).

### Limitations of the method

The limitations of this method include:

- The parameters required to establish the allowable differential settlement for a given wall are extremely difficult to determine; for example, the shear modulus for a load-bearing masonry wall depends on the type of wall, the type of mortar used and the level of workmanship.
- Burland[C11] assumed that the critical strain value as constant at 0.03%. In reality, different types of construction will have different critical strain values.
- The later work discussed above in 'Allowance for lateral strain' was limited to an aspect ratio of l/H = 1, and cannot be used to assess damage associated with other aspect ratios.

## C2.4 Strain superposition – beam approach (Boone)

The strain superposition method proposed by Boone[C12] is a development of Burland's[C11] limiting tensile strain approach. Like the later work of Burland[C11] it models the deformation of a building wall in terms of three basic modes, namely bending, shear and direct extension, as illustrated in Figure C5. Using well-known equations for beam deformation and assumed ratios of elastic and shear modulus values it separately considered these modes of deformation.

Boone assumed that the building deforms to match the ground, and rather than using the idea of 'angular distortion', where discrete points along the wall experience movement, the wall is considered to be uniformly loaded, with the differential settlement acting across the structure as a whole.

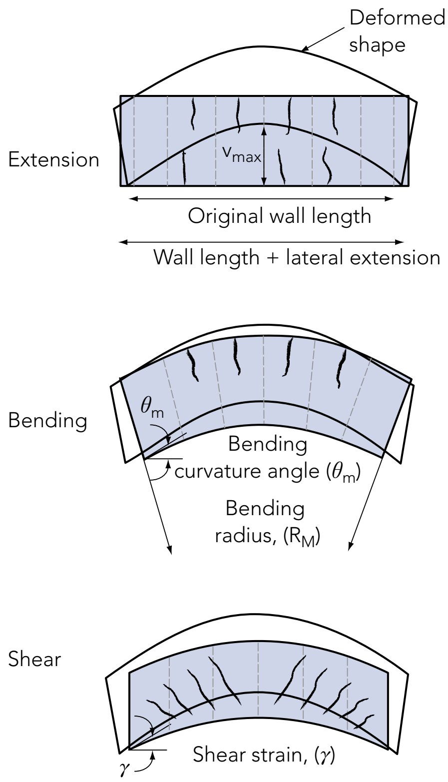



**Figure C5** Deformation modes in a 'hogging-type failure' (from Boone[C12]).

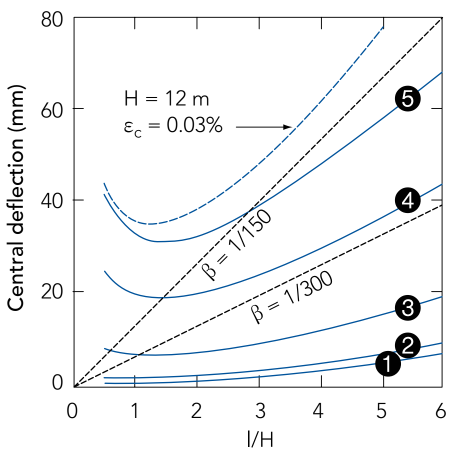
The following three cases were considered:

- load-bearing masonry construction
- steel frame construction
- concrete frame construction.

In common with the earlier work of Polshin and Tokar[C5] and of Burland,[C11] the approach assumes that the wall elements have a certain critical strain value, and, for low-rise housing structures of between one and four storeys, that movements in the ground will induce the same geometry in the structure. Using derived critical strain values for a variety of construction types, the amount of damage that will occur as a result of differential movements between adjacent pile supports can be calculated.

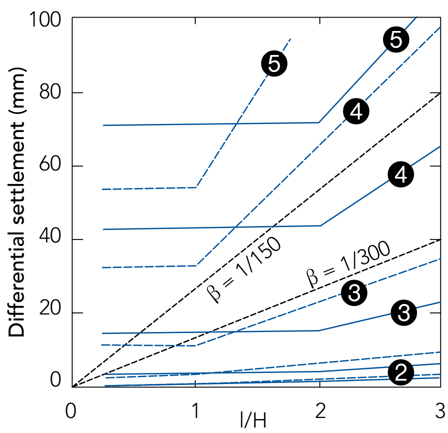
'Critical cracking strains' which have been derived for a variety of construction types are input to the calculation, allowing the strain required to lead to the onset of cracking to be calculated. These values have been obtained from laboratory tests, full-scale wall panel tests and case studies, and are listed by Boone.[C12]

A range of panels with different aspect ratios were analysed to determine the angular distortions required to lead to the onset of serviceability and structural damage in accordance with the damage criteria values given in Table C1, and Boone presented plots of damage thresholds for both load-bearing walls and concrete and steel-framed structures. A range of aspect ratios plotted against central deflection calculated for load-bearing masonry walls are reproduced in Figure C6 and values of differential settlement for concrete and steel-framed walls of different aspect ratios are shown on Figure C7. Plots are presented for each of the five damage categories shown in Table C1. A wall height of 8 m was assumed, corresponding approximately to a three-storey house, and a relatively conservative critical strain value of 0.01%. A plot showing onset of structural damage for a 12-metre-high wall with critical strain value of 0.03% is also shown. The linear 'angular distortion' relationships proposed by Skempton and MacDonald[C3] of 1/150 for structural damage and 1/300 for serviceability are presented on the plots for comparison.




H: height of wall
l/H: aspect ratio
$\beta$: angular distortion
$\varepsilon_c$: critical strain value
(see Table C-1 for definition of damage categories 1-5 shown)

**Figure C6** Damage thresholds for load-bearing walls (from Boone[C12]). H, height of wall; l/H, aspect ratio; $\beta$, angular distortion; $\varepsilon_c$, critical strain value.




H: height of wall
l/H: aspect ratio
$\beta$: angular distortion
(see Table C-1 for definition of damage categories 1-5 shown)

**Figure C7** Damage thresholds developed for infill walls in steel frames (dotted lines) and concrete frames (solid lines) (From Boone[C12]). l/H, aspect ratio, $\beta$, angular distortion.

Based on the relationships developed by Boone,[C12] it is possible to derive acceptable movements for buildings with different aspect ratios, and different values of critical strain. However, there are limitations to the approach, including the following:

- The approach only idealises a two-dimensional panel, and does not consider the modelling of a house in three dimensions.
- Although the approach allows an assessment of the construction materials used and method of construction, the results are very sensitive to the critical strain values used, and only limited critical strain information is currently available.
- Assumptions are required concerning the manner of crack development. It is likely that once a critical strain is reached in a given wall, existing cracks will widen in preference to new cracks forming. Although Boone[C12] suggests on the basis of statistical analysis that the maximum crack size will tend to be two-thirds of the cumulative crack width, this should be viewed with caution, and will be dependent on a number of factors, including the arrangement of openings such as doors and windows and the quality of workmanship.
- It has been assumed that the differential settlements only start after the construction has been fully completed. In reality, a proportion of the total and differential settlements will occur during construction as a result of the dead weight of the primary elements.

## C3 Total settlements and tilt settlements

### C3.1 Charles and Skinner (2004) assessment of acceptable tilt settlements for stiff rafts

Charles and Skinner[C13] discussed the use of a stiff raft foundation to control differential settlements. When there is the potential for differential settlement to occur, foundations of adequate stiffness may control the differential settlements between support positions leading to the development of tilt settlement as the stiff raft structure rotates. If the structure is assumed to deform in pure tilt, with no angular distortion, cracking will not occur. Pure tilt is unlikely to occur in reality, and a combination of components of tilt and reduced differential settlements will develop.

Charles and Skinner[C13] proposed a series of six basic failure mechanisms, including components of both differential settlement and tilt for a section through a typical low-rise building. These are shown on Figure C8.

Although useful as a simple idealisation, there are a number of limitations, including the following:

- No account is made for the three-dimensional settlement behaviour, ie differential movements across the breadth as well as the length of the buildings.
- As building length increases, so too does the complexity of the expected ground deformation profile, with some areas undergoing sagging and some hogging.

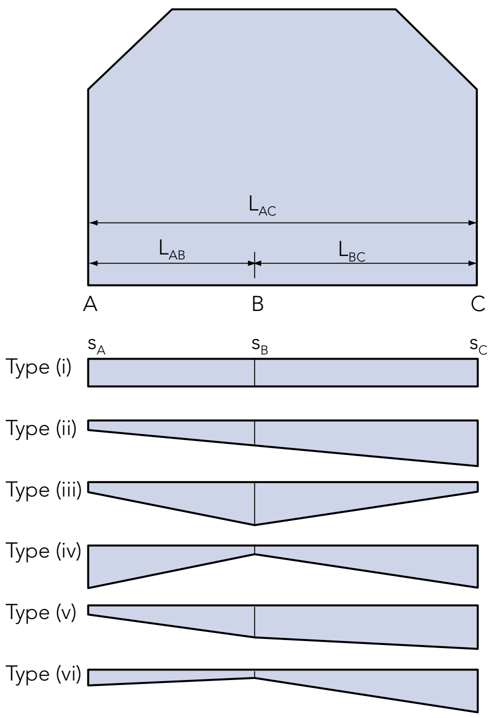



**Figure C8** Basic types of foundation settlement (from BRE Digest 475).[C14]

The definition of angular distortion given by Charles and Skinner[C13] differs from the definitions used by earlier authors, who do not always allow for the component of tilt settlements in their definition of angular distortion. Most authors seem to rely on this parameter to determine levels of damage and yet, due to the inconsistency in definition between them, there are difficulties when trying to combine the findings of various studies.

Charles and Skinner[C13] provide indicative values for acceptable tilt of low-rise housing assuming support on a stiff raft type foundation that undergoes rotation. It was concluded that although dependent on the layout of the building and the perception of the occupiers, tilt generally becomes noticeable at a ratio of between 1/200 and 1/250, although ultimate limits of tilt in the region of 1/50 are given as the point at which the building may be regarded as being in a dangerous condition.

The BRE guidance suggests a design limit value of 1/400, and if tilts are likely to exceed this, ground treatment or deep foundations are recommended.

### C3.2 Consideration of services connections

The critical issues usually considered for buildings are the angular distortions associated with the differential settlements between support positions and the overall tilt of the structure. Total settlements are generally considered to be less important for the structure itself, and large settlements can be acceptable structurally if all of the support positions settle by the same amount.

However, as the total settlements of a building increase relative to the adjacent ground, so too does the potential for damage to the service connections coming into that building.

Although the total settlements of piled low-rise structures are unlikely to be significant if they are adequately designed, the potential for large settlements of adjacent ground, (eg in the case of raised ground over soft compressible materials) must be considered.

More flexible services can accommodate relatively large settlements and it is not considered that they would generally prove critical. More brittle services, for example those associated with drainage, can be dealt with by use of flexible joints between the main structure and the service with which it is connected, eg in the form of a rocker pipe at the structure/manhole interface.

The potential for reduced drainage gradients should be borne in mind where there is a potential for differential settlements to occur between the structure and the adjacent ground. This will be a particular issue if drainage runs are designed with minimum falls and subsequent settlement reduces the falls below acceptable levels.

## C4 Summary and conclusions

The serviceability requirements of low-rise housing will be dependent on the susceptibility to distortions of the various components of the house, including:

- masonry (cracking in brickwork/blockwork panels and load-bearing walls)
- brittle finishes (plaster, external render, tiles and paints)
- windows (sticking of windows within their frames)
- doors (jamming of doors due to distortion)
- flooring systems.

General guidance has been published concerning the acceptable distortions for buildings, which can be used to develop serviceability criteria for low-rise housing. However, there are significant limitations with the published information and current methods available to define the actual tolerance to total, differential, and tilt settlements of low-rise buildings.

The approaches considered have all modelled the problem in terms of a two-dimensional idealisation, and by application of simple structural theories, with reference to either relative geometric movements between support positions or bending of masonry wall elements. The approaches reviewed do not take account of the complex three-dimensional arrangement of building elements, and the uncertain behaviour of the construction materials.

The actual serviceability limits of any given house will be dependent on the way that the various components combine, and hence predicting actual behaviour is very difficult.

More sophisticated approaches to analyse acceptable movements, such as three-dimensional finite element analyses, could in principle be adopted to deliver efficiencies in foundation design, but these are highly unlikely to be considered appropriate for low-rise housing developments.

The use of published guidance to assess generally acceptable limits of building movements and distortions, and application of these to determine values of foundation settlements, can however be of practical use in terms of foundation design.

## Appendix C references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# APPENDIX D

# Review of recorded damage due to subsidence/heave and settlement

## D1 Damage due to subsidence/heave and settlement: case studies

The majority of the literature reviews of damage to houses due to subsidence/settlement issues have been based on claims data, either during or after warranty periods.

A useful discussion of the reasons for subsidence claims for foundations in general, following review of insurance claims data (for houses more than 10 years old), was presented by Driscoll and Skinner[D1] and in *Subsidence Damage of Low Rise Buildings* by IStructE.[D2] Driscoll and Crilly[D3] presented a discussion of the causes of damage leading to claims made during the warranty period.[1]

The most commonly documented causes of damage based on claims data are associated with localised heave or subsidence caused by trees on clay sites, and washout and ground loss issues associated with leaking drains; for example refer to Driscoll and Skinner[D1] and Bonshor and Bonshor.[D4]

However, it must be recognised that the onset of failure caused by the loading effects of buildings (eg differential settlements or excessive settlements due to settlement/consolidation of weak/soft ground) are more likely to manifest themselves either during or shortly after construction, ie often before housing warranties come into effect.[2] Occurrences of these early failures tend to be less well documented, as the remedial works are more likely to be completed during the construction stage without the involvement of insurance companies.

A discussion of some of the issues which have been encountered by NHBC engineers during the construction stages and early in the building's life are discussed in Appendix A.

## Appendix D references

D1 Driscoll R M C, Skinner H. (2007). Foundation for the Built Environment Report 13: Subsidence damage to domestic buildings – a guide to good technical practice. IHS BRE Press, Bracknell.

D2 IStructE (2000). Subsidence damage of low-rise buildings – a guide for professionals and property owners (2nd edition). IStructE, London.

D3 Driscoll R M C, Crilly M S (2000). Foundation for the Built Environment Report 1, Subsidence damage to domestic buildings: Lessons learnt and questions remaining. IHS BRE Press, Bracknell.

D4 Bonshor R B, Bonshor L L (1996). Cracking in buildings. IHS BRE Press, Bracknell.

D5 Atkinson M F (2000). Structural defects reference manual for low-rise buildings. E & FN Spon, London.

# APPENDIX E

# Comparison between pile types

Table E1

| Comparison between displacement and non-displacement pile type† | | | |
|---|---|---|---|
| Pile type | Advantages | Disadvantages | Considerations |
| Displacement piles | | | |
| Driven pre-cast concrete | • Rapid installation, can be economic in most soil conditions<br>• Can be easily lengthened<br>• Concrete can be inspected prior to installation<br>• Pile lengths of up to 30 m are common and greater lengths are achievable with jointed piles<br>• Minimal arisings in comparison to augered piles | • Some noise and vibration is inevitable although reduction measures (shrouded hammers and pre-boring) are possible<br>• Difficulty in driving through ground containing obstructions – will usually require pre-excavation<br>• Generally requires significant headroom for installation<br>• Verticality of pile may be out of tolerance if obstructions are encountered | • Heave of ground, lifting of closely spaced adjacent piles due to pile driving in fine grained soils or of short piles founded on bedrock<br>• Installation effects including vibration and noise as they relate to adjacent structures and occupants<br>• Integrity of pile may be compromised during pile driving<br>• Typical axial load range between 400 and 1200 kN (Fleming et al)[E1] for piles ranging from 200–350 mm section<br>• Some types of segmental pile may not be suitable for tensile loads and/or lateral loads |
| Driven steel tubes | • Robust and light to handle, can tolerate higher driving stresses than concrete (with thick-walled sections)<br>• Less ground displacement and heave than pre-cast concrete piles when driven open ended<br>• Can be installed up to same depths as pre-cast concrete and deeper while withstanding harder driving conditions<br>• Some types can be extended using spigot and socket couplings | • More noise (but can have less vibration) than pre-cast concrete piles<br>• More susceptible to corrosion over long design life<br>• Better able to deal with obstructions than pre-cast concrete but may still require pre-excavation<br>• Generally requires significant headroom for installation<br>• Verticality of pile may be out of tolerance if obstructions are encountered | • Pile head/cap or beam connection detailing<br>• Installation effects including vibration and noise as they relate to adjacent structures and occupants<br>• Typical axial load range between 400 and 700 kN for smaller diameters (180–250 mm outside diameter) |
| Screw piles – steel | • Minimal arisings in comparison to augered piles<br>• Less likely to create contamination pathways<br>• Can be installed to depths of up to 20 m with readily available equipment<br>• Minimal arisings in comparison to augered piles | • More susceptible to corrosion over long design life<br>• Not generally suitable for ground containing obstructions | • Unacceptable heave and shearing can occur in cohesive soils<br>• Typical axial loads up to 300 kN in suitable soil conditions |

| (Table E1 contd.) Comparison between displacement and non-displacement pile type† | | | |
|---|---|---|---|
| Pile type | Advantages | Disadvantages | Considerations |
| Displacement piles (continued) | | | |
| Driven cast in situ concrete | • Readily adjustable in length<br>• Generally less reinforcing required than pre-cast concrete piles<br>• Can be installed to depths up to 30 m | • Noise and vibration (but can be reduced by use of shrouded hammers)<br>• Better able to deal with obstructions than pre-cast concrete but may still require pre-excavation<br>• Time required for installation is generally greater than driven pre-cast concrete or steel tube piles | • Installation effects including vibration and noise as they relate to adjacent structures and occupants<br>• Ground heave<br>• Typical axial load range up to 1500 kN |
| Non-displacement piles | | | |
| Bored cast in situ concrete | • Can be efficient when methods of ground support are not required<br>• Capable of achieving depths greater than 25 m with conventional rigs<br>• Arisings can be examined to verify ground conditions<br>• Limited noise and vibration<br>• Rigs available can be capable of operating in tight and confined spaces – particularly with smaller diameter piles | • Pile behaviour strongly influenced by quality of workmanship<br>• Risk of ground loss which may cause settlement of adjacent structures<br>• Where piling in contaminated land, cost of disposal of arisings must be considered<br>• Measures may be needed to deal with groundwater ingress to borings<br>• Higher costs of mobilisation make this technique better suited to large scale, single visit projects | • Instability of pile borings can occur in granular or soft soil conditions, particularly below the water table, which may affect the integrity and performance of the finished pile. Use of casing or support fluid can overcome this problem<br>• Washout due to mobile ground water can, in some cases, reduce pile section and affect integrity<br>• Loosening or softening of the pile boring if left open can occur which may reduce the shaft capacity<br>• Typical axial load range from 300–1500 kN and greater (CIRIA Report PG1)[E2] |

| (Table E1 contd.) Comparison between displacement and non-displacement pile type† | | | |
|---|---|---|---|
| Pile type | Advantages | Disadvantages | Considerations |
| Non-displacement piles (continued) | | | |
| Continuous flight auger (CFA) | • To a degree, arisings can be examined to verify ground conditions<br>• Limited noise and vibration<br>• Ground support provided by augers, avoiding the need for casings<br>• Can be installed to a maximum depth of around 25 m with standard equipment | • Pile behaviour strongly influenced by quality of workmanship – however instrumentation of rigs can help identify problems during installation<br>• Risk of ground loss (flighting of soft sensitive soils) which may cause settlement of adjacent structures<br>• Where piling in contaminated land, cost of disposal of arisings must be considered | • Instability in borings can occur if auger withdrawal rate is too rapid<br>• Support fluid or casing can be required in loose or water bearing granular soils and soft clays<br>• Loosening of base can occur during augering, which may cause reduced base capacity<br>• Typical axial load range from 300–1500 kN (CIRIA Report PG1)[E2] |
| Minipiles/ micropiles | • Flexibility in drilling techniques using the same rig<br>• Percussive hammer can penetrate bedrock and most obstructions<br>• Small rig sizes available for confined space working<br>• Arisings can be examined to verify ground conditions – but depends on drilling method<br>• Can be installed to depths up to 20 – 25 m and longer | • Noise generation may be significant with percussive hammer<br>• Relatively expensive<br>• In contaminated land, cost of disposal of arisings must be considered | • Installation effects including vibration and noise<br>• Typical axial load range 50–500 kN (CIRIA Report PG1)[E2] |
| † For all pile types installed or constructed through contaminated ground, the potential for creation of pathways for both the migration of contamination to groundwater and ground gases should be considered. | | | |

## Appendix E references


E1 Fleming K, Weltman A, Randolph M, Elson K (2008). Piling engineering (3rd edition). Taylor & Francis, Oxford.

E2 CIRIA (1988). Report PG1 Review of bearing pile types (2nd edition). Construction Industry Research and Information Association, London.

# APPENDIX F

# Summary of design code requirements for pile design

## F1 BS 8004 – British Standard Code of Practice for Foundations

### F1.1 Introduction – Design to BS 8004

BS EN 1997 or Eurocode 7[F1] supersedes BS 8004.[F2] It is evident however from surveys of pile designers undertaken during the preparation of this guidance that pile design using the approach defined in BS 8004 was still commonplace at the time of publication. BS 8004 will cease to be a British Standard in March 2010.

Clause 7.3.1 states that every pile design will have to satisfy three general conditions:

- the factor of safety against failure, both in terms of the fabric of the foundation and of the supporting soil, has to be adequate
- the settlement of the foundation as a whole and in particular differential settlements under working load should not be so large as to affect the serviceability of the structure
- the safety and stability of nearby buildings and services should not be put at risk.

### F1.2 Ultimate bearing capacity

Pile design undertaken in accordance with BS 8004 is defined by use of a 'global' factor of safety applied to prevent against ultimate failure. This approach is typically defined by the following relationship:

$$Q_{SAFE} \leq \frac{Q_{su} + Q_{Bu}}{FOS}$$

where:

$Q_{SAFE}$ is the safe working load
$Q_{SU}$ is the pile ultimate shaft capacity
$Q_{BU}$ is the pile ultimate base capacity
FOS is a global factor of safety (typically in the range of 2.0–3.0 for single piles)

BS 8004 states that the ultimate bearing capacity can be assessed by a number of means, including applying dynamic pile-driving formulae, use of stress wave analysis, on the basis of calculation methods using soil test data, or from a results of preliminary loading test(s). Clause 7.3.8 goes on to state that the ultimate bearing capacity should be obtained from a static load test wherever practicable. BS 8004 also states that where a sufficient number of load tests have been conducted or where it may be justified by local experience, values of factor of safety within the lower end of the suggested range presented above may be used.

The factor of safety should be chosen having regard to the nature of the soil, its variability over the site and the reliability of the method by which the ultimate bearing capacity has been determined.

## F1.3 Serviceability (consideration of settlements)

Generally, the assumption is made when designing piles to BS 8004 that pile settlements will be within acceptable structure tolerances when a sufficiently high global factor of safety is applied to ultimate failure. Thus, in many cases direct verification of the serviceability requirements will not be undertaken.

With regards to the settlement of the pile at working loads, Clause 7.3.8 of BS 8004 states:

> "It is essential that the settlement, and in particular the differential settlement, at working load be not greater than can be tolerated by the structure."

BS 8004 states that the factor of safety necessary to ensure this criterion is achieved may in some cases be larger than that required to prevent bearing failure.

# F2 BS EN 1997-1 (2004) Eurocode 7: geotechnical design

## F2.1 Introduction

The Eurocodes have been developed to standardise design approaches across disciplines. It is intended that Eurocode 7, and the UK National Annex,[F3] will fully replace BS 8004 for geotechnical design by March 2010.

A full review of the requirements of Eurocode 7 is beyond the scope of this guide, and reference should be made to the Eurocode 7 documents and the corresponding UK National Annexes. A number of useful guidance documents are also available, including:

- Frank R, Baudin C, Driscoll R, Kavvadas M, Krebbs Ovesen N, Orr T, Schuppener B (2004). *Designers' guide to EN 1997-1 Eurocode 7: Geotechnical design – General rules*.[F4]
- Driscoll R, Scott P, Powell J (2008). CIRIA C641 – *Eurocode 7, implications for UK practice*.[F5]
- Bond A, Harris A (2008). *Decoding Eurocode 7*.[F6]

A discussion of the Eurocode 7 methodology as it relates to design of piles for low rise houses is given below.

## F2.2 Eurocode 7 design approach

There are a number of methods for designing piled foundations in accordance with Eurocode 7. The different general approaches that the code supports are outlined in Clause 7.4.1 and include design based on:

- the results of static load testing, which have been demonstrated by means of calculations or otherwise to be consistent with previous experience
- calculation methods, either empirical or analytical, whose validity has been demonstrated by static load tests in comparable situations
- results of dynamic load tests, whose validity has been demonstrated by static load tests in comparable situations
- the observed performance of a comparable pile foundation, provided the approach is supported by the results of site investigation and ground testing.

Regardless of the approach used, the overriding requirement of pile design in accordance with Eurocode 7 involves verification of both the ultimate limit state (ULS) and serviceability limit state (SLS). Satisfying these criteria requires compliance of the following relationships:

Ultimate limit state (ULS) $F_{c;d} \leq R_{c;d}$

Serviceability limit state (SLS) $E_d \leq C_d$

where:

$F_{c;d}$ is the design axial compression load on the pile
$R_{c;d}$ is the design compressive resistance of the pile
$E_d$ is the anticipated design pile settlement
$C_d$ is the limiting acceptable pile settlement that the supported structure can tolerate.

In ultimate limit state design calculations, Eurocode 7 requires the use of partial factors applied to actions, material parameters, and pile resistances. The partial factors used in ULS design are presented throughout this Appendix. For serviceability limit state assessment, the value of all partial factors should be 1.0, thus actions and soil properties are unfactored.

## F2.3 Ultimate limit state

### Partial factors for actions

The UK National Annex to Eurocode 7 specifies the use of Design Approach 1, whereby two combinations of partial factors are applied in the calculation of $F_{c;d}$ and $R_{c;d}$; a suitable ULS design will involve both combinations satisfying the above inequality. The partial factors to be applied to permanent and variable actions for each combination are defined in Table F1.

Table F1

Partial factors to be applied to actions for ultimate limit state assessment

| Action type | | | Combination 1 | Combination 2 |
|---|---|---|---|---|
| Permanent actions | Unfavourable | $\gamma_G$ | 1.35 | 1.0 |
| | Favourable | $\gamma_{G,fav}$ | 1.0 | 1.0 |
| Variable actions | Unfavourable | $\gamma_Q$ | 1.5 | 1.3 |
| | Favourable | $\gamma_{Q,fav}$ | 0 | 0 |

### Partial factors for soil parameters

The partial factors to be applied to soil parameters are defined in Table A.NA.4 of the UK National Annex to Eurocode 7. Design Approach 1 requires that a partial factor of 1.0 be applied to all soil parameters used in the calculation of pile resistances. These soil parameters are the characteristic values determined in accordance with the guidance in Eurocode 7.

For the case where a soil parameter is used to determine the magnitude of an unfavourable action (for example negative skin friction) the following partial factors are applied to enhance the effect of the action. However, it is important to note that these factors are not applied to soil parameters in the calculation of the capacity of the pile itself.

Angle of shearing resistance $\gamma_{\varphi'} = 1.25$
Effective cohesion $\gamma_{c'} = 1.25$
Undrained shear strength $\gamma_{cu} = 1.4$
Unconfined strength $\gamma_{qu} = 1.4$

### Partial factors for pile resistances

The design compressive pile resistance ($R_{c;d}$) is calculated using one of the following relationships:

$$R_{c;d} = \frac{R_{s;k}}{\gamma_s} + \frac{R_{b;k}}{\gamma_B} \qquad \text{(F1)}$$

$$R_{c;d} = \frac{R_{c;k}}{\gamma_t} \qquad \text{(F2)}$$

where:

$R_{c;d}$ is the design compressive resistance of the pile
$R_{c;k}$ is the characteristic total compressive resistance of the pile
$R_{s;k}$ is the characteristic value of shaft resistance
$R_{b;k}$ is the characteristic value of base resistance
$\gamma_s$,and $\gamma_B$ are the partial factors for shaft and base resistance respectively, and $\gamma_t$ is the partial factor for the total resistance of the pile.

The relationship F1 can be used when separate load contributions of the shaft and base are understood.

The partial factors to be applied to characteristic pile resistances are defined in A.NA.6, A.NA.7 and A.NA.8 of the UK National Annex to Eurocode 7. Assessment of the partial factor set Combination 1 requires all resistance factors ($\gamma_s$, $\gamma_B$, $\gamma_t$) equal to 1.0. The Combination 2 partial factor set to be applied to pile resistances is presented in Table F2.

**Table F2**

**Partial factors to be applied to pile resistances for ultimate limit state assessment**

| Design verification | $\gamma_s$ | | $\gamma_B$ | | $\gamma_t$ | |
|---|---|---|---|---|---|---|
| | Driven | Bored/ CFA | Driven | Bored/ CFA | Driven | Bored/ CFA |
| No verification of SLS by static load testing | 1.5 | 1.6 | 1.7 | 2.0 | 1.7 | 2.0 |
| Verification of SLS by static load tests on >1% of piles to $1.5Q_{WORKING}$ | 1.3 | 1.4 | 1.5 | 1.7 | 1.5 | 1.7 |

Note: CFA, continuous flight auger; SLS, serviceability limit state.

While Eurocode 7 requires both combinations to be checked during ULS assessment, for a typical pile design associated with low-rise housing, where compressive loading is the main form of loading and the effects of negative skin friction are assumed to be minimal, the Combination 2 partial factor set will generally govern the geotechnical design.

(Note, however, that the structural design of axially loaded piles will generally be governed by Combination 1, which is consistent with the requirements of EN 1990, EN 1992 and EN 1993, and their UK National Annexes).

Eurocode 7 states that the determination of the characteristic compressive resistance ($R_{c;k}$), shaft resistance ($R_{s;k}$) and base resistance ($R_{b;k}$) can be based on the results of static load tests, empirical or analytical calculation methods, dynamic impact tests, pile driving formulae, or wave equation analysis. A description of the design process using static load test results and calculation methods is presented in F2.4 and F2.5 respectively.

Where calculation methods are used to assess characteristic capacities, an additional partial factor (the model factor $\gamma_{Rd}$) is applied to the calculated resistances in order to address the uncertainty in the design and ensure that the characteristic capacities derived from load testing and from calculation are equivalent.

## F2.4 Design by static load test results

The results of static load tests on preliminary piles can be used to design the working piles. The process for determining the characteristic resistance ($R_{c;k}$) from static load tests includes the following:

- Piles are loaded to failure, which is usually the state at which there is significant displacement with negligible increase or decrease in resistance, ie where the load settlement curve flattens off. Where the load settlement plot shows continuous curvature, Eurocode 7 recommends that 'failure' is defined by a pile settlement of 10% of the pile base diameter.
- Results are normalised to account for any variation in pile diameter or length across the dataset of pile tests.
- The pile test results included in any single dataset should relate to similar ground conditions. If there are pile test locations where ground conditions differ significantly they should be removed from the data set and handled as a separate design case.
- The characteristic resistance ($R_{c;k}$) is determined by applying the following expression to the dataset of test results:

$$R_{c;k} = \text{Min}\left\{\frac{(R_{c;m})_{mean}}{\xi_1};\frac{(R_{c;m})_{min}}{\xi_2}\right\}$$

where:

$R_{c;k}$ is the characteristic total compressive resistance
$R_{c;m}$ is the pile load recorded during a static load test at a pile settlement equivalent to 10% of the pile diameter
$\xi_1$,and $\xi_2$ are correlation factors related to the number of piles tested.

Correlation factors are presented in Table A.NA.9 of the UK National Annex to Eurocode 7.[F3] The correlation factors reduce progressively for a greater number of conducted tests. The correlation factors are presented in Table F3.

Table F3

Correlation factors for static load test data

| Number of tested piles | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| $\xi_1$ | 1.55 | 1.47 | 1.42 | 1.38 | 1.35 |
| $\xi_2$ | 1.55 | 1.35 | 1.23 | 1.15 | 1.08 |

Note: The correlation factors in Table F3 can be divided by 1.1 if the supported structure has sufficient stiffness and strength to transfer loads from weak to strong piles, provided that $\xi_1$ is never less than 1.0.

The pile design then proceeds with the general process outlined in section F2.3 above such that:

- Pile actions are factored by the partial factors presented in Table F1 to calculate $F_{c;d}$
- Pile characteristic resistances $R_{c;k}$ are factored by the partial factors presented in Table F2 to calculate the design compressive resistance $R_{c;d}$
- Verification of ULS requirement $F_{c;d} \leq R_{c;d}$.

## F2.5 Design by calculation

Calculation by empirical or analytical methods is also supported by Eurocode 7. Such methods will generally involve calculation of shaft and base capacity using empirical correlations with soil parameters. The soil parameters used will generally be derived from in situ or laboratory test results.

The uncertainty associated with calculative methods results in application of an additional model partial factor ($\gamma_{Rd}$) into the calculation of design compressive resistance ($R_{c;d}$). The magnitude of the model factor is normally 1.4; however, there is an allowance for this to be reduced to 1.2 where a static load test has been conducted on a preliminary pile to verify the unfactored calculated ultimate resistance.

Thus the design by calculation procedure can generally be summarised as follows:

- Pile actions are factored by the partial factors presented in Table F1 to calculate $F_{c;d}$.
- Assessment of the ground investigation data, in situ testing, and laboratory testing data is undertaken to determine 'characteristic' soil strength parameters.
- Various calculation methods are used to calculate characteristic shaft resistance and base resistance (for example unit shaft friction = $\alpha * C_u$).
- Pile characteristic resistances $R_{c;k}$ are factored by the partial factors presented in Table F2 plus an additional model partial factor of 1.4 to calculate the design compressive resistance $R_{c;d}$.
- Verification of ULS requirement $F_{c;d} \leq R_{c;d}$.

Based on the review of current practice, for low-rise housing foundations it is likely that empirical calculation will usually be the favoured method used to determine $R_{c;d}$. However, for larger developments, where it may be feasible to conduct static load tests, the permitted use of lower partial factors may result in a much more efficient design.

The stages for design by calculation are presented in Figure F1.

#### Other methods

Static load testing on preliminary test piles is emphasised in Eurocode 7, although dynamic load testing can be used.

#### Design by pile-driving formulae/wave equation analysis

The estimation of the compressive resistance of the pile using pile-driving formulae is supported by Eurocode 7, which requires that the ground stratification is well-understood, and that the method is validated by static load tests on similar piles in similar ground.

### F2.6 Serviceability limit state

The Eurocode 7 SLS design requires an assessment of the anticipated pile settlements to ensure they are within the tolerances of the supported structure. Partial factors of 1.0 are applied to both actions and material properties for this assessment. Although some designers carry out a specific settlement analysis to satisfy the Eurocode 7 serviceability criteria, it appears on the basis of the survey of current practice that was undertaken when preparing this guidance that this is often not done for pile design for low-rise housing.

Eurocode 7 provides some guidance on design cases where verification of the SLS will generally be achieved by satisfying the ULS. Relevant clauses from Eurocode 7 include:

Clause 7.6.4.1 which notes that when bearing on medium dense to dense soils, safety requirements for ultimate limit state design with lumped factors of safety are normally sufficient to satisfy the serviceability limit state requirements of the supported structure.

Clause 7.6.4.2 which notes that when the pile toe is placed in a medium dense or firm layer overlying rock or very hard soil, the partial safety factors for ultimate limit state design are normally sufficient to prevent a serviceability limit state in the supported structure.


Notes
Partial factors are listed in the UK National Annex to BS EN 1997.[F3]
Load Combinations 1 and 2 to be checked separately.

**Figure F1** Stages of design by calculation for Eurocode 7.

## F2.7 Supervision of pile construction – records and requirements

Eurocode 7 requires that a pile installation plan is prepared, which forms the basis of the piling works. The plan should include details such as the pile types and construction make up, locations, dimensions, construction tolerance, required load carrying capacity, installation sequence, and known constraints such as obstructions.

Construction requirements are presented for driven and bored piles respectively in the following documents:

- BSI (2000). BS EN 12699: *Execution of Special Geotechnical Work – Displacement Piles.*[F7]
- BSI (2000). BS EN 1536: *Execution of Special Geotechnical Work – Bored Piles.*[F8]
- BSI (2006). BS EN 14199: *Execution of Special Geotechnical Work – Micro Piles.*[F9]

## Appendix F references

F1 BSI (2004). BS EN 1997-1 – Eurocode 7, Part 1: General rules. British Standards Institution, London.

F2 BSI (1986). BS 8004. Code of practice for foundations. British Standards Institution, London.

F3 BSI (2004). UK National Annex to BS EN 1997-1:2004. Eurocode 7, Part 1: General rules. British Standards Institution, London.

F4 Frank R, Baudin C, Driscoll R, Kavvadas M, Krebbs Ovesen N, Orr T, Schuppener B (2004). Designers' guide to EN 1997-1 Eurocode 7: Geotechnical design – General rules. Thomas Telford, London.

F5 Driscoll R, Scott P, Powell J (2008). CIRIA C641 - Eurocode 7, implications for UK practice. Construction Industry Research and Information Association, London.

F6 Bond A, Harris A (2008). Decoding Eurocode 7. Taylor & Francis, Oxford.

F7 BSI (2000). BS EN 12699. Execution of special geotechnical work – Displacement piles. British Standards Institution, London.

F8 BSI (2000). BS EN 1536. Execution of special geotechnical work – Bored piles. British Standards Institution, London.

F9 BSI (2006). BS EN 14199. Execution of special geotechnical work – Micro piles. British Standards Institution, London.

## Further reading and guidance documents

### BSI

BSI (1996). BS 8103. Structural design of low-rise buildings. Code of practice for timber floors and roofs for housing.

BS EN 1997-2. Eurocode 7, Part 2: Ground investigation and testing.

BSI (2007). UK National Annex to BS EN 1997-2:2007 – Eurocode 7, Part 2: Ground investigation and testing.

# APPENDIX G

# Worked design examples

## G1 Worked pile design example: friction pile

### G1.1 Low-rise housing foundation

For a three-storey apartment building to be founded on made ground overlying firm to stiff clay, pile foundations have been selected as the favoured foundation solution to limit structural movements to acceptable limits. Based on an assessment of ground conditions, site constraints, construction timescales, and cost, CFA piles have been considered to be the most suitable method of piling at the site. A pile diameter of 450 mm is to be used.

The ground conditions at the site have been investigated using a combination of boreholes (with SPTs) and trial pits. The ground conditions identified at the site comprise:

0.0–2.0 m Made ground: loose, fine to coarse, gravelly sand derived from colliery spoil. SPT results range from N = 2 to 5

2.0–15.0 m Firm to stiff, silty clay. SPT results range from N = 15 at a depth of 2.0 m increasing progressively with depth such that a correlated (using $C_u$ = 5 N) profile of characteristic undrained shear strength of $C_u$ = 75 + 10(z) kPa has been adopted.

Unfactored pile dead and live loads have been provided and comprise:

Dead load (DL) 250 kN per pile

Live load (LL) 100 kN per pile.

Maximum pile settlements of 12 mm ($C_d$) have been specified.

The example in Figure G1 presents the typical pile design procedure to determine the required pile toe level to satisfy the ultimate limit state and ensure the serviceability limit state is satisfied for both Eurocode 7 and BS 8004. The Eurocode 7 calculation adopts Design Approach 1, with partial factors used taken from the UK National Annex.

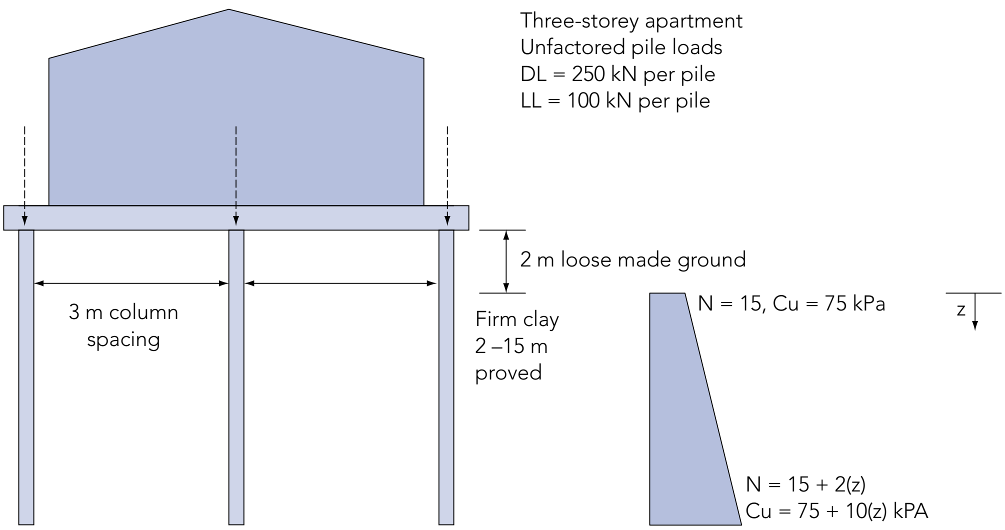



**Figure G1** Schematic section indicating loads, ground conditions and foundation layout.

## G1.2 Design to Eurocode 7

### Ultimate limit state

Design actions ($F_{c;d}$)

Combination 1 $F_{c;d} = \gamma_G$ (DL) + $\gamma_Q$ (LL) = 1.35 * 250 + 1.5 * 100 = 487.5 kN

Combination 2 $F_{c;d} = \gamma_G$ (DL) + $\gamma_Q$ (LL) = 1.00 * 250 + 1.3 * 100 = 380.0 kN

Design resistance ($R_{c;d}$)

$$R_{c;d} = \frac{R_{s;k}}{\gamma_s} + \frac{R_{b;k}}{\gamma_B}$$

**Characteristic shaft resistance ($R_{s;k}$)**

The unit shaft friction can be calculated using empirical calculation methods described by Tomlinson and Woodward.[G1] The contribution of the upper 2 m of made ground has been neglected. An $\alpha$ = 0.5 has been selected and an characteristic $C_u$ value over the shaft assumed. A material partial factor of 1.0 applied for both Combination 1 and Combination 2. A model factor of 1.4 has been applied in the calculation of the Characteristic shaft resistance in accordance with Clause A3.3.2 the UK National Annex. For a 10.5 m long pile:

$$R_{s;k} = \frac{\alpha c_u A_s \gamma_m}{\gamma_{Rd}} = 0.5 * (75 + 10((L-2)/2)) * 3.141 * 0.45 * (L-2)/1.4 = 504.3 \text{ kN}$$

**Characteristic base resistance ($R_{b;k}$)**

The base resistance friction has been calculated using empirical calculation methods described by Tomlinson and Woodward.[G1] An $N_c$ value of 9 has been adopted and a $C_u$ value of 75 + 10(L–2) assumed. A material partial factor of 1.0 is applied for both Combination 1 and Combination 2. A model factor of 1.4 has been applied in the calculation of the characteristic base resistance in accordance with Clause A3.3.2 the UK National Annex. For a 10.5m long pile:

$$R_{b;k} = \frac{N_c c_u A_b \gamma_m}{\gamma_{Rd}} = 9 * (75 + 10(L-2)) * 3.141 * 0.225 * 0.225 * 1.0/1.4 = 163.6 \text{ kN}$$

**Design resistance ($R_{c;d}$)**

Calculating the design resistance to determine the required pile length:

$$R_{c;d} = \frac{R_{s;k}}{\gamma_s} + \frac{R_{b;k}}{\gamma_B}$$

where:

$R_{c;d}$ is the design compressive resistance of the pile
$R_{s;k}$ is the characteristic value of shaft resistance
$R_{b;k}$ is the characteristic value of base resistance
$\gamma_s$ and $\gamma_b$ are the partial factors for shaft and base resistance respectively

The partial factors applied to pile resistance are presented in the UK National Annex Table A.NA.8 and Section A.3.3.2. No pile test has been undertaken.

Combination 1 Combination 2
$\gamma_s$ = 1.0 $\gamma_s$ = 1.6
$\gamma_B$ = 1.0 $\gamma_B$ = 2.0

| Case | $F_{c;d}$ (kN) | $R_{c;d}$ (kN) | | |
|---|---|---|---|---|
| | | L = 10 m | L = 10.5 m | L = 11 m |
| Combination 1 | 487 | 622 | 668 | 713 |
| Combination 2 | 380 | 369 | 397 | 425 |

By inspection of the design actions and resistances in the table above Combination 2 has been shown to be the critical case, requiring a pile length of **10.5 m** to meet the condition of $F_{c;d} \leq R_{c;d}$.

## Serviceability limit state

### Design actions ($F_{c;d}$)

$$F_{c;d} = \gamma_G(DL) + \gamma_Q(LL) = 1.0 * 250 + 1.0 * 100 = 350 \text{ kN}$$

### Design settlement ($E_d$)

An estimate of pile settlement has been undertaken using the method outlined in Tomlinson and Woodward.[G1]

$$E_d = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4}\frac{W_b}{A_b}\frac{B(1-v^2)I_p}{E_b}$$

where:

| | |
|---|---|
| $W_s$ | is the load on the pile shaft |
| $W_b$ | is the load on the pile base |
| L | is the shaft length |
| $A_s$, $A_b$ | are the pile cross-sectional areas of the shaft and base respectively |
| $E_p$ | is the elastic modulus of the pile material |
| B | is the pile diameter/width |
| v | is Poisson's ratio of the soil |
| $I_p$ | is an influence factor related to the ratio of L/B; for L/B >5, $I_p$ = 0.5,[G1] this will generally be the case with piled foundations for low-rise housing |
| $E_b$ | is the deformation modulus of the soil beneath the pile base |

Assessment of pile settlement using this approach requires an assumption to be made regarding the load distribution between the shaft and base. For the ground conditions adopted in this example, it has been assumed that under working load conditions 85% of the pile resistance will be provided by the shaft ($W_s$) and the remaining 15% of the pile resistance will be provided by the base ($W_b$):

| | | |
|---|---|---|
| $W_s$ | = | 0.85*350 = 297.5 kN |
| $W_b$ | = | 0.15*350 = 52.5 kN |
| L | = | 8.5 m |
| $A_s$ | = | $A_b$ = 3.141*0.225*0.225 = 0.159 $m^2$ |
| $E_p$ | = | 12.5*$10^6$ kN/$m^2$ for long-term conditions |
| B | = | pile diameter = 0.45 m |
| v | = | Poisson's ratio = 0.2 |
| $I_p$ | = | 0.5 |

Based on CIRIA report 143[G2] a direct correlation[G3] between SPT 'N' value and drained modulus E' = 0.9 N has been adopted.

E' = $E_b$ = 0.9 N
At pile base N = 15 + 2*(8.5) = 32
$E_b$ = 0.9*32 = 28800 kPa

$$E_d = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4}\frac{W_b}{A_b}\frac{B(1-v^2)I_p}{E_b}$$

$E_d$ = 0.86 + 1.95
$E_d$ = 2.8 mm

This satisfies the serviceability limit state criteria of $E_d \leq C_d$.

### G1.3 Design to BS 8004

The safe pile working load can be calculated from the following expression:

$$Q_{SAFE} \leq \frac{Q_{su} + Q_{Bu}}{FOS} = \text{where FOS} = 2.5$$

The unit shaft friction has been calculated using empirical calculation methods described in Tomlinson and Woodward.[G1] The contribution of the upper 2 m of made ground has been neglected. An $\alpha$ = 0.5 has been selected and average $C_u$ value over the shaft adopted. For a 10.5 m pile:

$$Q_{su} = \alpha c_u A_s = 0.5 * (75 + 10(L2)/2) * 3.14 * 0.45 * (L - 2) = 706 \text{ kN}$$

The base resistance friction has been calculated using empirical calculation methods described by Tomlinson and Woodward.[G1] An $N_c$ value of 9 has been adopted and a $C_u$ value of 75 + 10(L–2) assumed. For a 10.5 m pile:

$$Q_{bu} = N_c c_u A_b = 9 * (75 + 10(L - 2)) * 3.14 * 0.225 * 0.225 = 229 \text{ kN}$$

For pile length:

10.0 m $Q_{SAFE}$ = 650/2.5 + 220/2.5 = 348

10.5 m $Q_{SAFE}$ = 705/2.5 + 229/2.5 = 373

Pile length of 10.5 m would be required for a working load of 350 kN.

The serviceability limit state calculation required by Eurocode 7, and described in the 'servicability limit state' section above should also be undertaken with the BS 8004 design approach.

## G2 Worked pile design example: end-bearing pile

### G2.1 Low-rise housing foundation

Consider the example presented in G1.1 with alternative ground conditions. A three-storey apartment building is to be founded on soft to firm clay overlying medium-dense gravelly sand. Pile foundations have been selected as the favoured foundation solution to limit structural movements to acceptable limits. Based on an assessment of ground conditions, site constraints, construction timescales, and cost, CFA piles have been considered to be the most suitable method of piling. A pile diameter of 450 mm is to be used.

The ground conditions at the site have been investigated by using a combination of boreholes (with SPTs) and trial pits. The ground conditions identified at the site comprise:

| | |
|---|---|
| 0.0–8.0 m | Soft to firm clay: SPT results average N = 5 over the full depth of the stratum. |
| 8.0–15.0 m | Medium dense gravelly sand. SPT results range from 20 to 30 over the depth of the strata, with an average N value of 25. |

A groundwater level of 1.0 m below existing ground surface has been assumed.

Unfactored pile dead and live loads have been provided and comprise:

Dead load (DL) 250 kN per pile.

Live load (LL) 100 kN per pile.

Maximum pile settlements of 12 mm ($C_d$) have been specified.

The example in Figure G2 presents the typical pile design procedure to determine the required pile toe level to satisfy the ultimate limit state and ensure the serviceability limit state is satisfied for both Eurocode 7 and BS 8004. The Eurocode 7 calculation adopts Design Approach 1, with partial factors used taken from the UK National Annex.

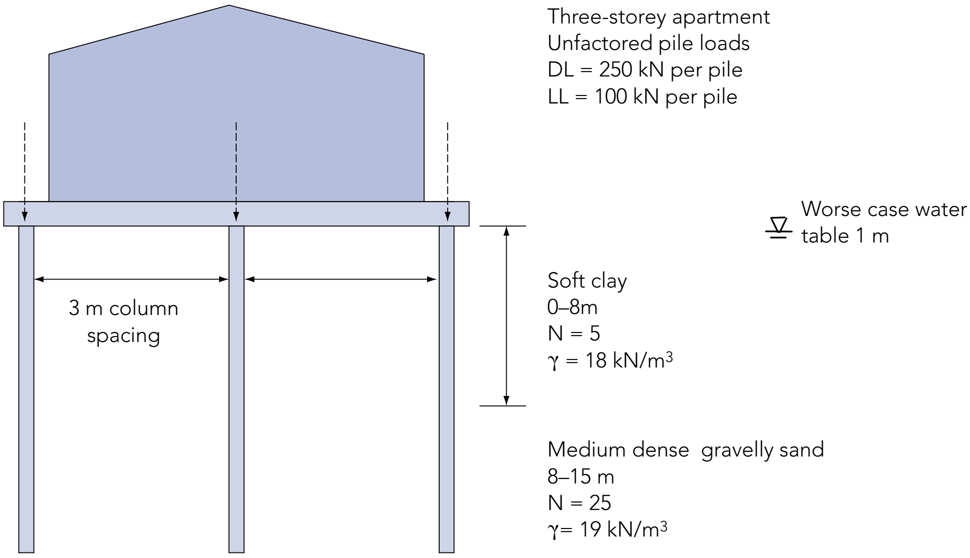



**Figure G2** Schematic section indicating loads, ground conditions and foundation layout.

## G2.2 Design to Eurocode 7

### Ultimate limit state

Design actions ($F_{c;d}$)
Combination 1 = 1.35*250 + 1.5*100 = 487.5 kN
Combination 2 = 1.00*250 + 1.3*100 = 380.0 kN
Design resistance ($R_{c;d}$)

$$R_{c;d} = \frac{R_{s;k}}{\gamma_s} + \frac{R_{b;k}}{\gamma_B}$$

**Characteristic shaft resistance ( $R_{s;k}$) – soft clay**

The unit shaft friction in the soft to firm clay has been calculated using empirical calculation methods described by Tomlinson and Woodward [G1]. An $\alpha$ = 0.5 has been selected and a characteristic $C_u$ value over the shaft assumed. A material partial factor of 1.0 has been applied for both Combination 1 and Combination 2. A model factor of 1.4 has been applied in the calculation of the characteristic shaft resistance in accordance with Clause A3.3.2 the UK National Annex.

$$R_{s;k} = \frac{\alpha c_u A_s \gamma_m}{\gamma_{Rd}} = 0.5 * 25 * 3.141 * 0.45 * (8)/1.4 = 101 \text{ kN}$$

### Characteristic shaft resistance ($R_{s;k}$) – medium dense gravel

The shaft friction in the medium dense gravels has also been calculated using the methods outlined Tomlinson and Woodward [G1]. A k value of 0.85 has been adopted and pile soil interface friction angle of $\delta = \varphi'$. A characteristic or average effective stress ($\sigma'_{av}$) over the relevant shaft increment has been assumed. A material partial factor of 1.0 has been applied for both Combination 1 and Combination 2. A model factor of 1.4 has been applied in the calculation of the characteristic shaft resistance in accordance with Clause A3.3.2 the UK National Annex.

$$R_{s;k} = \frac{k \tan(\delta) \sigma'_{av} A_s \gamma_m}{\gamma_{Rd}}$$

For a 10 m long pile:-

$R_{s;k}$ = 0.85*tan(34)(18*8 + ((L–8)/2)*19–(8 + ((L–8)/2)–1)*9.81)*3.14*0.45*(L–8)*1.0/1.4 = 97.3 kN

### Characteristic base resistance ($R_{b;k}$)

The base resistance has been calculated using the following relationship outlined by Tomlinson and Woodward [G1]. An $N_q$ value of 50 has been adopted based on a friction angle $\varphi'$ = 34° and the bearing capacity factors proposed by Berezantsev [G4] for an assumed pile depth to diameter ratio of approximately 20. A material partial factor of 1.0 has been applied for both Combination 1 and Combination 2. A model factor of 1.4 has been applied in the calculation of the characteristic base resistance in accordance with Clause A3.3.2 the UK National Annex.

$$R_{b;k} = \frac{N_q \sigma'_{base} A_b \gamma_m}{\gamma_{Rd}} = 50 * (8 * 18 + (L - 8) * 19 - (L - 1) * 9.81)$$

$$* 3.14 * 0.225 * 0.225 * 1/1.4 = 532.4 \text{ kN}$$

### Design resistance ($R_{c;d}$)

Calculating the design resistance to determine the required pile length:

$$R_{c;d} = \frac{R_{s;k}}{\gamma_s} + \frac{R_{b;k}}{\gamma_B}$$

where:

$R_{c;d}$ is the design compressive resistance of the pile
$R_{s;k}$ is the characteristic value of shaft resistance
$R_{b;k}$ is the characteristic value of base resistance
$\gamma_s$ and $\gamma_B$ are the partial factors for shaft and base resistance respectively

The partial factors applied to pile resistance are presented in the UK National Annex Table A.NA.8 and Section A.3.3.2. No pile testing has been undertaken.

| Combination 1 | Combination 2 |
|---|---|
| $\gamma_s = 1.0$ | $\gamma_s = 1.6$ |
| $\gamma_B = 1.0$ | $\gamma_B = 2.0$ |

| Case | $F_{c;d}$ (kN) | $R_{c;d}$ (kN) | | |
|---|---|---|---|---|
| | | L = 9.5 m | L = 10 m | L = 10.5 m |
| Combination 1 | 487 | 678 | 731 | 785 |
| Combination 2 | 380 | 361 | 390 | 421 |

By inspection of the design actions and resistances in the table above, Combination 2 has been shown to be the critical case, requiring a pile length of 10.0 m to meet the condition of $F_{c;d} \leq R_{c;d}$.

## Serviceability limit state

### Design actions ($F_{c;d}$)

$F_{c;d} = \gamma_G(DL) + \gamma_Q(LL) = 1.0 * 250 + 1.0 * 100 = 350$ kN

### Design settlement ($E_d$)

An estimate of pile settlement can be undertaken using the method outlined by Tomlinson and Woodward[G1]:

$$E_d = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4}\frac{W_b}{A_b}\frac{B(1 - v^2)I_p}{E_b}$$

where:

| | |
|---|---|
| $W_s$ | is the load on the pile shaft |
| $W_b$ | is the load on the pile base |
| L | is the shaft length |
| $A_s$, $A_b$ | are the pile cross sectional areas of the shaft and base respectively |
| $E_p$ | is the elastic modulus of the pile material |
| B | is the pile diameter/width |
| v | is Poisson's ratio of the soil |
| $I_p$ | is an influence factor related to the ratio of L/B; for L/B >5, $I_p$ = 0.5 (Tomlinson, 2008), this will generally be the case with piled foundations for low-rise housing. |
| $E_b$ | is the deformation modulus of the soil beneath the pile base. |

Assessment of pile settlement using this approach requires an assumption to be made regarding the load distribution between the shaft and base. For the ground conditions adopted in this example, it has been assumed that under working load conditions 30% of the load resistance will be provided by the shaft ($W_s$) and the remaining 70% of the resistance provided by the base ($W_b$):

| | | |
|---|---|---|
| $W_s$ | = | 0.30*350 = 105 kN |
| $W_b$ | = | 0.70*350 = 245 kN |
| L | = | 10.0 m |
| $A_s$ | = | $A_b$ = 3.141*0.225*0.225 = 0.159 $m^2$ |
| $E_p$ | = | $12.5*10^6$ kN/$m^2$ for long-term conditions |
| B | = | pile diameter = 0.45 m |
| v | = | Poisson's ratio = 0.2 |
| $I_p$ | = | 0.5 |

Based on CIRIA report 143 (1995) a direct correlation[G3] between SPT N Value and drained modulus E' = 1.0 N has been adopted.

E' = $E_b$ = 1.0 N
At pile base N = 30
$E_b$ = 30 000 kPa

$$E_d = \frac{(W_s + 2W_b)L}{2A_s E_p} + \frac{\pi}{4}\frac{W_b}{A_b}\frac{B(1 - v^2)I_p}{E_b}$$

$E_d$ = 0.5 + 8.7
$E_d$ = 9.2 mm

This satisfies the serviceability limit state criteria of $E_d \le C_d$

## G2.3 Design to BS 8004

The safe pile working load can be calculated from the following expression:

$$Q_{SAFE} \leq \frac{Q_{su} + Q_{Bu}}{FOS} = \text{where FOS} = 2.5$$

The unit shaft friction in the soft clay strata can be calculated using empirical calculation methods described by Tomlinson and Woodward[G1]. An $\alpha$ = 0.5 has been selected and average $C_u$ value over the shaft has been adopted.

$$Q_{su} = \alpha c_u A_s = 0.5 * 25 * 3.141 * 0.45 * (8) = 141 \text{ kN}$$

The shaft friction in the medium dense gravels is also calculated using the methods outlined by Tomlinson and Woodward [G1]. A k value of 0.85 has been adopted and pile soil interface friction angle of $\delta = \varphi'$. The average effective stress ($\sigma'_{av}$) over the relevant shaft increment has been assumed.

$$Q_{su} = k \tan(\delta) \sigma'_{av} A_s$$

For a 10 m long pile:

$$Q_{su} = 0.85*\tan(34)(18*8 + ((L–8)/2)*19–(8 + ((L–8)/2)–1)*9.81)*3.14*0.45*(L\text{-}8) = 137$$

The base resistance has been calculated using the following relationship outlined in Tomlinson (2008). An $N_q$ value of 50 has been adopted based on a friction angle $\varphi' = 34°$ and the bearing capacity factors proposed by Berezantsev for an assumed pile depth to diameter ratio of approximately 20. For a 10 m long pile:-

$$Q_{bu} = N_q \sigma'_{base} A_b = 50*(8*18 + (L–8)*19–(L–1)*9.81)*3.14*0.225*0.225 = 745 \text{ kN}$$

For pile length:
9.0 m $Q_{SAFE}$ = 206/2.5 + 672/2.5 = 351 kN
9.5 m $Q_{SAFE}$ = 241/2.5 + 709/2.5 = 380 kN
10.0 m $Q_{SAFE}$ = 278/2.5 + 745/2.5 = 409 kN

A pile length of 9.0 m would be required for a working load of 350 kN.

Note that the serviceability calculation undertaken in the section 'Serviceability limit state' should also be conducted for the BS 8004 design approach.

## Appendix G references


G1 Tomlinson M, Woodward J (2007). Pile design and construction (5th edition). Taylor & Francis, Oxford.

G2 Clayton (1995). CIRIA Report 143. The standard penetration test (SPT): methods and use. Construction Industry Research and Information Association, London.

G3 Stroud (1989). The standard penetration test – its application and interpretation. Proceedings of the ICE conference on penetration testing in the UK, Birmingham. Thomas Telford, London.

G4 Berezantsev et al. (1961). Load bearing capacity and deformation of piled foundations. Proceedings of the 5th International Conference, ISSMFE, Paris. Vol. 2, pp. 11-12.

# APPENDIX H

# Pile tests

## H1 General

Pile testing can be undertaken for a number of reasons, including the validation of pile designs, quality control during construction, and as the basis of the pile design.

Eurocode 7 design principles allow pile design to be undertaken on the basis of preliminary static load tests undertaken in advance of the main phase of development, which can enable more efficient pile designs to be developed by providing a better understanding of the pile settlement behaviour.

When design is on the basis of calculation methods, static load tests undertaken on working piles reduce uncertainty and can allow reductions in factors of safety, and subsequent cost savings.

There may also be other benefits of pile testing, particularly preliminary test piles, which may include confirmation of the variability in ground conditions across the site, information on the pile buildability, and demonstration that noise and vibration levels will be within acceptable limits in advance of the main piling contract.

The extent of pile testing undertaken should be assessed not only on the basis of any regulatory requirements that may be imposed, but also on the level of risk or uncertainty posed by the site conditions, together with the potential benefits that may be derived in terms of more efficient designs.

## H2 Static load testing of preliminary trial piles

The confirmation of pile load capacity and assessment of load/settlement behaviour can be made by preliminary static load pile tests (Fig. H1). Such tests are carried out on sacrificial piles in advance of the main piling work, which are usually tested to either the required unfactored ultimate capacity or to failure.

Back analysis of well-executed pile tests can be undertaken eg using the Fleming method,[H1] and under the limit state design principles of Eurocode 7, the application of partial factors to the measured load settlement curve can be used to determine design behaviour of working piles.

For large developments it is worth considering such tests to refine pile designs which otherwise may be based on conservative parameters and higher factors of safety. The benefit of these tests is also dependent on the quality of the ground investigation information available and the degree of uniformity of the ground conditions.

Static load testing of preliminary piles can lead to large savings on larger jobs, but potential efficiency and cost savings will be much less significant for smaller schemes.

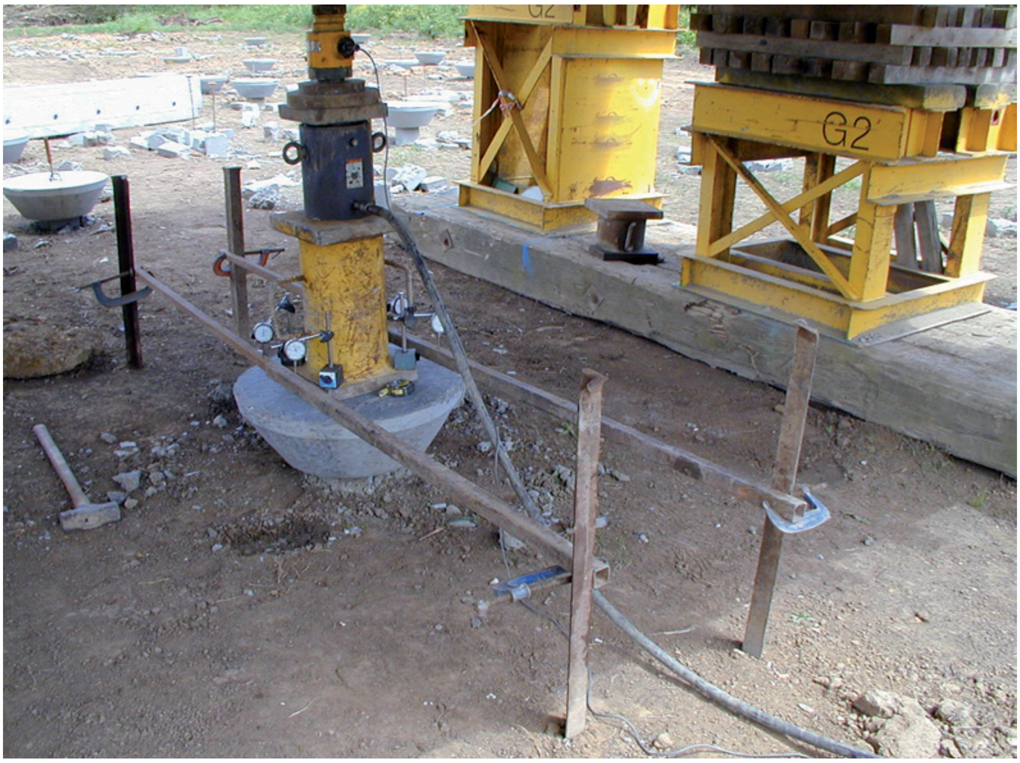



**Figure H1** Static load testing of piles.

## H3 Static load testing of working piles

The maximum load applied is the design verification load (DVL) + 50% of the specified working load (SWL). The design verification load should include allowance for the specified working loads to be supported by the pile together with any other conditions which may increase the settlement of the working piles, including soil induced forces such as down-drag, and if piles are closely spaced, pile group effects.

As well as confirming that the working load capacity (and greater) is achieved, the test confirms that the load/settlement behaviour of the pile is within the specified limits.

**Figure H2** Static load testing using reaction piles.

## H4 Dynamic testing

Dynamic testing methods can allow predictions to be made of the pile performance under static load much more quickly and cheaply than static load tests. Dynamic stress wave data, obtained from instrumentation attached to the pile can be analysed to give estimates of the static pile capacity.

There are various techniques of testing and methods of analysis available using commercially available software, including methods that utilise simple formulae, such as the CASE method of analysis, and more complex iterative methods using commercially available software such as CAPWAP, which involve the matching of stress wave data. The interpretation of the results requires expert judgement, and must be undertaken by a suitably skilled and trained specialist who understands the limitations.

Some of the main considerations and limitations of dynamic test methods include:

- The efficiency of the hammer needs to be known to ensure that the energy transferred to the pile is understood.
- Capacities of driven piles change after installation, this change is typically an increase in capacity due to dissipation of excess pore water pressures generated during installation, although reductions in capacity with time can occur in certain soil types if negative pore water pressures develop during the pile installation. These problems can be addressed by 'restriking' of the piles at some period following installation.
- The hammer blows applied to the pile during the dynamic test are usually insufficient to 'fail' the pile, and the wave data generated will only allow an estimate to be made of the 'mobilised' static resistance – this may be much less that the ultimate static resistance of the pile. Exceptions to this include where dynamic testing is undertaken during installation of driven piles.
- Effects such as 'viscous damping' and 'inertial damping' can strongly influence the pile behaviour in dynamic load tests – with significantly higher dynamic capacities calculated, particularly in cohesive soils. Damping parameters need to be carefully determined to ensure that this is accounted for. Results of static load tests can be used to obtain values for these damping effects.
- Although reliance on the expertise of the dynamic testing organisation and their experience in similar soil types or knowledge from published studies may be adequate if the ground conditions are relatively uniform and well-understood, it is generally advised that if possible at least one static load test is performed to establish values of damping coefficients and 'calibrate' the dynamic analyses.

Used in combination with static testing, dynamic testing undertaken and interpreted by a suitably trained specialist can offer economies and reduce risks by allowing a greater number of piles to be tested.

## H5 Statnamic testing

Although statnamic testing has not been traditionally used for low-rise housing developments, there is potential for its application.

The advantage of statnamic testing is that it allows the testing of piles to obtain similar results to those obtained from static load tests but without excessive reaction systems. A combustion chamber which causes a reaction mass to be accelerated upwards causes a reaction in the pile many times greater than the reaction mass. The load transferred to the pile is monitored by a load cell, and by attaching a laser sensor to the pile head, the pile head displacement can be measured using a laser level. Corrections for viscous damping as discussed above are still required, but the behaviour of the pile is a much closer approximation to that of a static load test.


**Figure H3** Example of a crawler mounted 1 MN statnamic test system.

## Appendix H references

H1 Fleming W G K (1992). A new method for single pile settlement prediction and analysis. Geotechnique 42, No. 3, pp. 411-425.

H2 The Institution of Civil Engineers (2007). Specification for piling and embedded retaining walls (2nd edition). Thomas Telford Ltd, London.

H3 Tomlinson M, Woodward J (2007). Pile design and construction (5th edition). Taylor & Francis, Oxford.

H4 Fleming K, Weltman A, Randolph M, Elson K (2008). Piling engineering (3rd edition). Taylor & Francis, Oxford.

H5 BSI. BS EN ISO 22477-1. Pile load test – static axially loaded compression test (not yet published).

H6 ISSMFE Subcommittee on field and laboratory testing, axial pile loading test, suggested method. ASTM Journal, June 1985, pp. 79-90.

## Appendix H further reading and guidance

For further detailed guidance relating to the load testing of piles, it is recommended that reference is made to the FPS *Load Testing Handbook*, which is available as a free download on the website of the Federation of Piling Specialists (www.fps.org.uk).

# APPENDIX I

## Planning considerations and key environmental legislation

**Table I1**

**Building Regulations and planning**

| Legislation, orders and regulations | Key guidance documents | Key requirements | Further information |
|---|---|---|---|
| Building Regulations 2000 (SI 2000/2531) for England and Wales (and subsequent amendments)<br>Approved document A | • NHBC Standards Parts 4 and 5 (2008)<br>• Site Investigation Steering Group (1993). *Site investigation in construction series.* Thomas Telford Ltd., London<br>• *AGS guidelines for good practice in site investigation* (www.ags.org.uk) | • Foundations to be designed to ensure no impairment of the stability of the structure<br>• Assessment of ground conditions by commissioning a suitable site investigation | • BS 5930 *Code of Practice for Site Investigation*<br>• Eurocode 7: DD ENV 1997-2:2000 |
| Approved document C | • In addition to documents listed above:<br>– NHBC Report No. 10627-R01 (04) (January 2007). Evaluation of Development Proposals on Sites where Methane and Carbon Dioxide are Present<br>– BRE 211 (2007). *Radon Guidance on Protective Measures for New Dwellings*<br>– Annex A of Approved Document C provides guidance for the assessment of land affected by contaminants. Relevant standards and guidance documents are also listed | • Assessment of contaminated land – risks to people, buildings, and services and remedial measures<br>• Assessment of methane and other ground gases and remedial measures<br>• Assessment of radon gas<br>• Ensure appropriate site investigation/characterisation undertaken | • CIRIA Report 665 (2007). *Assessing Risks Posed by Hazardous Ground Gases to Buildings*<br>• Environment Agency (2004). *Guidance for Monitoring Landfill Gas Surface Emissions – (for flux box testing)*<br>• BSI (2007). BS 8485. *Code of Practice for the Characterisation and Remediation from Ground Gas in Affected Developments* |
| Town and Country Planning Act (1990) (OPSI 1990a) and related planning guidance<br>Part IIA of the Environmental Protection Act (Defra 2006a, WAG 2006a and Scottish Executive 2006a)<br>The Contaminated Land (England) Regulations 2006 | • *Guidance for the Safe Development of Housing on Land Affected by Contamination* (2008). R&D, p66, volume 1. Published by NHBC/EA<br>• *Guidance on the Legal Definition of Contaminated Land* (Defra, July 2008) | • Assessment of contamination – human health and controlled waters | • Environment Agency (2006) *A Guide for Developers Practical Advice in adding Value to Your Site: Environmental*<br>• BSI (2001). BS10175. *Investigation of Potentially Contaminated Sites – Code of Practice*<br>• CIRIA (2001). 552: *Contaminated Land Risk Assessment - A Good Practice Guide*<br>• Environment Agency (2005). *UK approach for Evaluating Human Health Risks from Petroleum Hydrocarbons in Soils*<br>• Environment Agency DEFRA (2004) CLR 11: *Model Procedures for the Management of Land Contamination*<br>• Environment Agency (2008). *Updated Technical Background to the CLEA Model*<br>• Environment Agency (2008). *Human Health Toxicological Assessment of Contamination in Soil*<br>• www.defra.gov.uk/environment/land/contaminated/legislation.htm |

Table I2

Waste

| Legislation, orders and regulations | Key guidance documents | Key requirements | Further information |
|---|---|---|---|
| Section 34 of the Environmental Protection Act 1990 | • See regulations below | • Imposes duty of care on any person who imports, produces, carries, keeps, treats or disposes of controlled waste or, as a broker, has control of such waste | • www.environment-agency.gov.uk |
| Environmental Protection (Duty of Care) Regulations 1991 | • Environment Agency. (1996 [revision in consultation]) *Waste Management – the Duty of Care – a Code of Practice* | • These regulations impose requirements for documentation/record keeping of activities involving waste on any person who is subject to the duty of care | • Environment Agency position statement - Definition of waste: development industry code of practice (www.environment-agency.gov.uk) |
| Environmental Permitting (EP) Regulations (6 April 2008) – replaces the Pollution Prevention Control and Waste Management Licensing Regulations<br><br>Hazardous Waste Regulations, 2005<br><br>Waste Framework Directive accompanies the Environmental Permitting Regulations | • DEFRA (2009). *Environmental Permitting Guidance. The Waste Framework Directive, for the Environmental Permitting* (England and Wales) Regulations 2007 | • Adopt a waste management hierarchy approach, reuse on site, reuse off site if possible or pre-treat prior to disposal. Material must have Waste Acceptance Criteria (WAC) testing to determine acceptance at landfill site<br>• Seek opportunities for reuse off site (under an exemption), then investigate disposal options | • www.environment-agency.gov.uk |
| Landfill (England and Wales) (Amendment) Regulations 2004.<br><br>Hazardous Waste Directive (Council Directive 91/689/EEC). | • Environment Agency (2003). *Technical Guidance WM2 Hazardous Waste – Interpretation of the definition and classification of hazardous waste*<br>• Environment Agency (April 2005). *Guidance on sampling and testing of waste to meet landfill waste acceptance procedures* – Version 1 | • Characterisation of waste to determine appropriate disposal<br>• Assessment of waste to determine whether or not it meets the definition of hazardous waste | • Environment Agency (July 2004) Framework for the classification of contaminated soils as hazardous waste<br>• Environment Agency (April 2006). Using the List of Waste to Code Waste for Waste Transfer Notes, PPC Permits and Waste Management Licences, England and Wales |
| Site Waste Management Plans (SWMP) Regulations 2008 came into force in England on 6th April 2008 | • NHBC Foundation (2008). NF8 *Site Waste Management, Guidance and Templates for Effective Site Waste Management*<br>• CIRIA C536 (2001). *Demonstrating Waste Minimisation Benefits in Construction*<br>• CIRIA (1997). Special publication 133: *Waste Minimisation in Construction – Site Guide* | • A SWMP sets out how building materials, and resulting waste, should be managed during the project | • www.wrap.org.uk<br>• Defra (2008). Non-Statutory Guidance for Site Waste Management Plans<br>• NetRegs. A Simple Guide to Site Waste Management Plans. For up to date guidance see www.netreg.gov.uk |

**Table I3**

**Contamination**

| Legislation, orders and regulations | Key guidance documents | Key requirements | Further information |
|---|---|---|---|
| Environmental Protection Act 1990: Part IIA | • Environment Agency (2004). CLR11: *Model Procedures Form a Framework for the Assessment of Sites of Land Affected by Contamination* | • Assessment of land to determine whether it is in such a condition that significant harm is being caused or there is a significant possibility of such harm being caused; or pollution of controlled waters is being, or is likely to be, caused | • Environment Agency and NHBC (2008). R&D publication 66: *Guidance for the Safe Development of Housing on Land Affected by Contamination* provides guidance on the practical application of good practice within this framework<br>• Environment Agency (2001) *Piling and Penetrative Ground Improvement Methods on Land Affected by Contamination: Guidance on Pollution Prevention* |
| The Groundwater Directive 2006/118/EC<br>'Daughter Directive' of the EU Water Framework Directive (WFD) which has been developed specifically to address the protection of groundwater against pollution and deterioration. | • Environment Agency (2001). *Piling and Penetrative Ground Improvement Methods on Land Affected by Contamination: Guidance on Pollution Prevention*<br>• Environment Agency. CLR11: *Model Procedures Form A Framework for the Assessment of Sites of Land Affected by Contamination* | • It must be ensured that activities do not pose a threat to groundwater from List 1 and List 2 substances (required under Part 2 of the EPA, 1990). It must be ensured that construction of foundations does not cause pollution of controlled waters, including groundwater | • www.environment-agency.gov.uk<br>• www.netregs.gov.uk<br>• The following legislation is also referred to in the Groundwater Directive 2006/118/EC:<br>• Water Resources Act 1991<br>• Pollution Prevention and Control (England and Wales) Regulations 2000<br>• Part II of the Control of Pollution Act 1974 (Scotland)<br>• Groundwater Regulations 1998 |

**Table I4**

**Air quality and dust**

| Legislation and orders | Key guidance documents | Key requirements | Further information |
|---|---|---|---|
| The Control of Pollution Act 1974<br>Part 1B of The Environmental Protection Act 1990<br>The Clean Air Act 1993 | • Contact your local authority environmental health officer. Guidance on prescribed processes covered is available | • All reasonable steps should be taken to avoid problems with dust, eg by damping down sand, aggregate or dusty concrete surfaces with water sprays when conditions become dry<br>• Wheel wash facilities should be provided at site exits to prevent any transfer of mud to roads | • www.netregs.gov.uk |

Table I4

**Noise and vibration**

| Legislation, orders and regulations | Key guidance documents | Key requirements | Further information |
|---|---|---|---|
| Section 80 of the Environmental Protection Act 1990 | • BSI (2009). BS 5228. *Code of Practice for Noise and Vibration Control on Construction and Open Sites Part 1, Noise; and Part 2, Vibration*<br>• Other useful codes:<br>– BSI (2008). BS 6472.1. *Guide to Evaluation of Human Exposure to Vibration In Buildings. Vibration Sources Other than Buildings*<br>– BSI (1993). BS 7385: Part 2. *Evaluation and Measurement for Vibrations in Buildings*<br>– BRE (1995). Digest 353: *Damage to Structures from Ground-borne Vibration* (deals specifically with damage to structures from ground borne vibration) | • Local authorities have the right to serve notices on contractors where noise problems arise | • www.netregs.gov.uk |
| Section 61 of the Control of Pollution Act 1974 | | • If a project is likely to have a significant impact on neighbours from noise or vibration, application for a 'prior consent' should be made with the local authority | • Local authorities have the right to serve notices on contractors where noise problems arise |
| The Control of Noise (Codes of Practice for Construction and Open Sites) (England) Order 2002<br>(*Parallel Orders approving the same guidance came into force in Scotland on 29 March 2002, in Wales on 1 August 2002 and in Northern Ireland on 1 November 2002)<br>Works plant and equipment must also comply with the Noise at Work Regulations 1989 in order to protect on-site personnel | | • The noise sensitivity of the area should be considered when determining the method of piling to be used<br>• No legislation for noise level exposure of the general public due to construction activities, although local authorities adopt their own levels of acceptable noise<br>• A method statement including the type of plant to be used and the proposed noise control methods – sound power levels of the plant is required<br>• Measurement of the vibration generated during the works may be required<br>• Calculations of LAeq and peak levels at specified buildings, as requested<br>• The contractor should be able to demonstrate that best practicable methods are being used to minimise vibration | • Community relations are of the uppermost importance<br>• Agree minimum distance from receptor (not practical in most UK sites)<br>• Enclose the piling hammer and pile with a sound absorbent box (ie Hoesch noise abatement tower)<br>• Use of a hush piling rig<br>• Tomlinson (2008). *Pile design and construction practice* – 4th edition) page 72 gives chart of noise levels<br>• An inspection of adjacent properties by a building surveyor prior to commencement of piling should be considered<br>• The contractor should reach an agreement with local residents/occupiers of commercial properties about the times when piling occurs, as it would not be reasonable for disturbance to occur throughout the entire working day<br>• Where there are concerns about the potential damage that may be caused by vibration to properties it is expected that the contractor will liaise directly with the complainants |

Note: The intention of Table I1 to I5 is to highlight some of the key considerations which should be addressed. The list is not intended to be comprehensive and is subject to amendment.
The regulatory environment is constantly changing, and developers should ensure that they obtain advice from a suitably qualified professional who is familiar with the requirements.
The UK accepts harmonised standards which are dual numbered British Standards, issued by the British Standards Institution (BSI) International Standards Organisation (ISO) and the European Committee for Standardisation (CEN).
These tables do not consider health and safety matters relating to building operations.

# APPENDIX J

# Comparison of embodied energy and carbon emissions: calculations and supporting information

## J1 Introduction

This Appendix aims to address environmental impact by means of a simple comparative exercise, using the environmental impact indicator of embodied carbon. Embodied carbon can be defined as the amount of $CO_2$ emitted in the creation of a given end product, in this case the foundations for a house.

Comparison has been made of three foundation options for a typical UK terraced house, relative to different ground conditions. The assessment considers construction of trenchfill foundations and of bored and driven piles.

## J2 Embodied carbon methodology

### J2.1 General

The embodied carbon for construction of a foundation is an indicator of its environmental impact. The embodied carbon can be used as an absolute quantity, but, more usefully, can be used for comparing alternative design options in the process of optimising a design solution. The embodied carbon calculation in this Appendix does not take into account the whole-life environmental impact of the structure, but only the part of it up to and including its construction, as described in Figure J1. The embodied carbon excludes any $CO_2$ emitted during or at the end of the operational life of the project, including any maintenance work, modification or demobilisation at the end of its life.

### J2.2 Embodied carbon calculation methodology

The calculation of the embodied carbon in a structure comprises three components, as schematically represented in Figure J2:

- **Materials**: $CO_2$ emitted for the production of the construction materials; this includes the extraction of raw materials and their processing into construction materials.
- **Transportation**: $CO_2$ emitted during transportation of material and plant to and from site.
- **Installation**: $CO_2$ emitted from construction plant during operation on site.

The servicing and maintenance of plant are assumed to be negligible and are not included in the calculation. Similarly, transportation of labour to and from site is excluded.

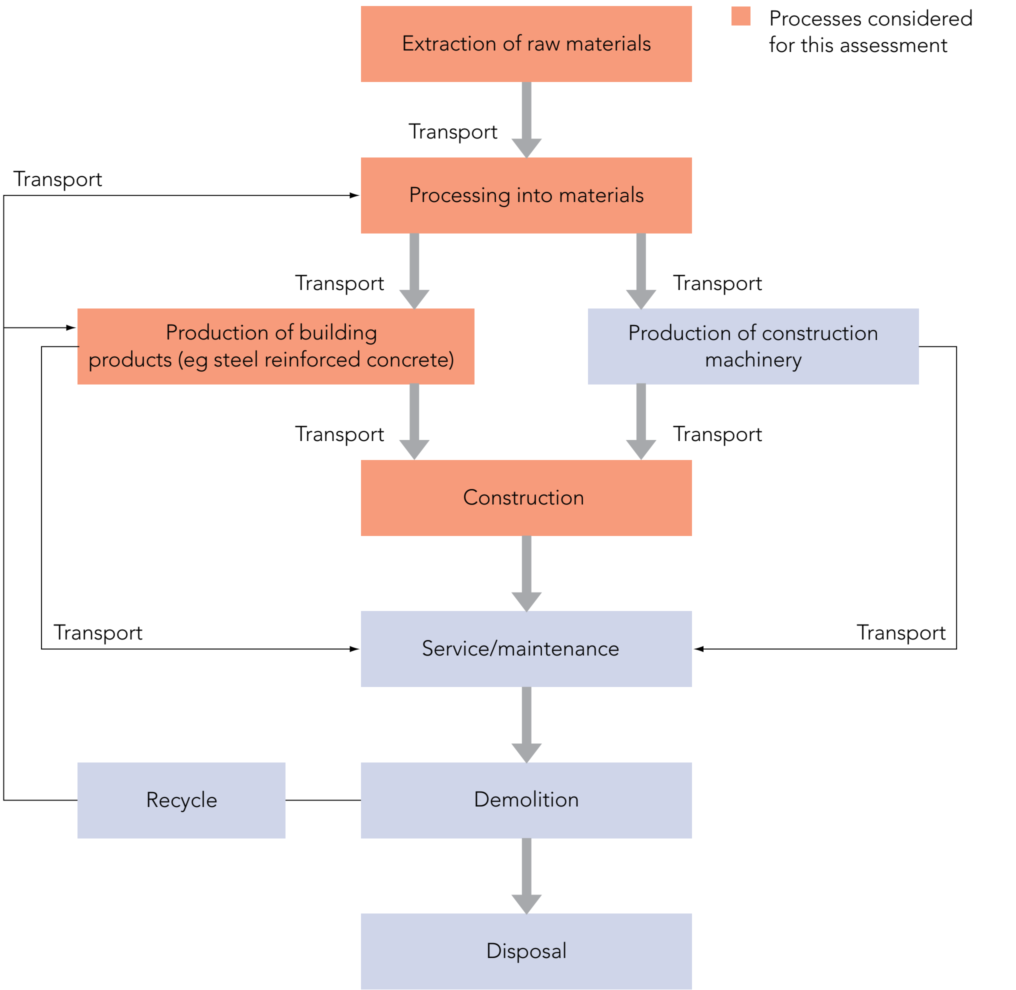
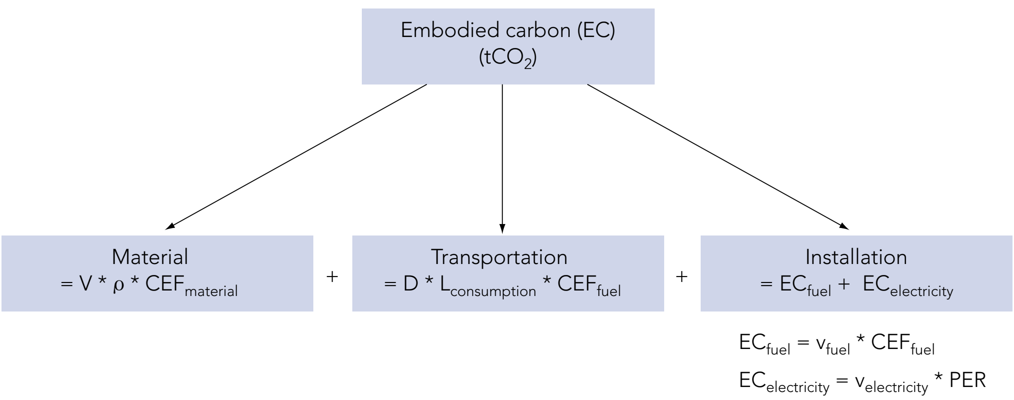



**Figure J1** Process flowchart for construction.




where:

| | |
|---|---|
| CEF | Carbon emission factor [kg $CO_2$/kg or $CO_2$/L) |
| V | Material quantity ($m^3$ from bill of quantities) |
| $\rho$ | Material density ($t/m^3$) |
| D | Distance travelled by transportation vehicle (miles or km) |
| $L_{consumption}$ | Fuel consumption rate (L/km) from DMRB[J1] |
| $v_{fuel}$ | Fuel consumed from plant during installation (L) |
| $v_{electricity}$ | Electricity consumed from electrical plant during installation (kWh) |

**Figure J2** Components of the carbon calculation. DMRB, *Design Manual for Roads and Bridges.*[J1]

## J2.3 Material component: carbon emission factors

The calculation of embodied carbon involves the use of published carbon emission factor (CEF) values for each of the materials involved in the final product. This is defined as the amount of embodied carbon emitted in the production of a given construction material from the point of resource extraction to the end product; its units are kg $CO_2$/kg or kg $CO_2$/litre for liquids. Research into CEF values has been active since 1979, from both the public and private domains. A wide range of values is often quoted for certain materials. The variation in value can be due to the different types of material in question, as well as the assumptions and study boundaries drawn for the CEF evaluations. As the Inventory of Carbon and Energy (ICE) states,

> "The nature of this work and the problems outlined made selection of a single value difficult and in fact a range of data would be far simpler to select but less useful to apply in calculations."

CEF values adopted for this study come from the ICE of the University of Bath,[J2] where they summarised values from international publications and deduced a set of recommended values for the most commonly used in the UK building materials.

Table J1 lists the material CEF values adopted in the present work with their respective sources.

**Table J1**

Carbon emission factors for the materials used in this study [J2]

| Material | Density (t/m³) | CEF (kg$CO_2$/kg) | Reference |
|---|---|---|---|
| Concrete in situ (kg) | 2.20 | 0.209 | Hammond & Jones (2008) |
| Concrete pre-cast (kg) | 2.20 | 0.215 | Hammond & Jones (2008) |
| Steel (bar) (kg) | 7.86 | 1.770 | Hammond & Jones (2008) |
| Backfill (soil) (kg) | 2.00 | 0.023 | Hammond & Jones (2008) |
| Brick (kg) | 2.00 | 0.220 | Hammond & Jones (2008) |

## J2.4 Carbon emissions from transportation and installation

### General

Embodied carbon associated with the transportation of items to and from site and during construction on site is directly linked to the amount of fuel and/or energy consumed during these activities. The calculation does not include any estimate of vehicle and plant depreciation (wear and tear during use on the project) or emissions resulting from the production of the vehicle or plant itself.

### Carbon emission factors of fuel

The emission factors used for diesel and electricity consumption are listed in Table J2.

**Table J2**

Carbon emission factors (CEFs) for fuel and electricity [J3]

| | CEF |
|---|---|
| Diesel (L) | 2.620 kg$CO_2$/litre |
| Electricity (kWh) | 0.537 kg$CO_2$/kWh |

### Transportation vehicles – DMRB approach

The amount of fuel consumed during transportation to and from site is carried out in accordance with the methodology described in the DMRB (Highways Agency)[J1] approach: seven categories of vehicle are distinguished.

For the calculation of the carbon emissions due to transportation of materials and plant:

$EC_{transportation} = V_{fuel} \times CEF_{fuel}$

where:
$V_{fuel}$ (L) = fuel consumed during transportation of material and plant
$CEF_{fuel}$ ($tCO_2/L$) = carbon emissions factor for fuel

Fuel consumption is calculated based on the type of transportation vehicle and average speed assumed, using the method described in the HA DMRB[J1] as detailed in Figure J3.

### Installation emissions

The amount of fuel and/or electricity consumed by the machinery and equipment during construction on site is quantified using information mainly based on Japanese guidelines [J4] in the absence of UK-specific information. The consumption rates used in this work are summarised in Table J3.

Table J3

Consumption rates for construction plant used in the calculations

| Construction plant | Production rate | Consumption rate | |
|---|---|---|---|
| | | (kW/h) | (l/day) |
| Pile driver: crawler base machine tripod support type 21–33 m 100–110 t | 0.3 days/pile | – | 10.5 |
| Rough terrain crane hydraulic jib type, load 25 t | 0.2 days/pile | – | 120 |
| Truck mixer and agitator truck: 3.0–3.2 $m^3$ | 3 hours/truck | – | 57 |
| Reverse circulation drill: air lift/pump suction type, max diameter 3000 mm max depth 200 m | 0.2 days/pile | 23 | – |
| Back-hoe: crawler type 0.8 $m^3$ | 300 $m^3$/day | – | 23 |

# J3 Calculation of $CO_2$ for house foundations

## J3.1 Assumptions made in the calculations

The present calculation is generic, making a number of simplifying assumptions relevant to transportation distances, type of construction plant used, and general construction practices. Such assumptions influence the $CO_2$ value calculated and are only acceptable for the purposes of comparing different types of foundations. For an accurate emissions value, the input parameters are site-specific and should be carefully chosen.

Table J4 lists out the transportation distances assumed in the calculations.

Table J4

Transportation distances used in the calculations

| Input parameter | Distance assumed (miles) |
|---|---|
| Distance of plant yard to site | 20 |
| Distance of landfill from site | 50 |
| Distance for ready mix concrete | 50 |
| Distance for steel transport | 50 |
| Distance for pre-cast units | 138 |

Note Excluded from the calculations: energy consumed to manufacture plant, transport of labour to and from site (very site specific), depreciation/maintenance during operational life and end of life decommissioning.

Calculation of the vehicle duel consumption using the methodology in Highways Agency. *Design Manual for Roads and Bridges*. Vol. 11 Environmental Assessment, section 3 Assessment Techniques. Part 1, Air quality: Annex B: Vehicle emissions.

$L = a + b.v + c.v^2 + d.v^3$

Where: L (l/km) = fuel consumption

v (km/hr) = average link speed

a, b, c, d = parameters defined for each vehicle category

**Table B11 Fuel Consumption Coefficients (Reference Year 2002)**

| Vehicle category | Parameters | | | |
|---|---|---|---|---|
| | a | b | c | d |
| Diesel_car | 0.1408661 | -0.0028522 | 2.86706E-05 | -6.93E-08 |
| Diesel_LGV | 0.1863759 | -0.0026805 | 1.17153E-05 | 8.23E-08 |
| OGV1 | 0.7683375 | -0.022573 | 0.000317658 | -1.354E-06 |
| OGV2 | 1.0244316 | -0.0302181 | 0.000442855 | -2.006E-06 |
| Petrol_car | 0.1880476 | -0.0043795 | 5.06795E-05 | -1.691E-07 |
| Petrol_LGV | 0.2524615 | -0.00487 | 4.42438E-05 | -7.53E-08 |
| PSV | 0.6346687 | -0.0189897 | 0.000274313 | -1.216E-06 |

**Table B12 Assumed Vehicle Fuel Efficiency Improvements**

| Vehicle category | 2002-2003 (actual) (%) | 2003-2004 (actual) (%) | 2004-2005 (%) | 2005-2010 (% pa) | 2010-2015 (% pa) | 2015-2020 (% pa) |
|---|---|---|---|---|---|---|
| Diesel_car | -1.18 | -1.19 | -1.21 | -1.22 | -1.35 | -1.24 |
| Diesel_LGV | 0.97 | -1.4 | -1.78 | -1.49 | 0 | 0 |
| OGV1 | 0.46 | 0 | 0 | -1.23 | 0 | 0 |
| OGV2 | -0.17 | 0 | 0 | -1.23 | 0 | 0 |
| Petrol_car | -0.74 | -0.75 | -0.76 | -0.85 | -1.35 | -1.48 |
| Petrol_LGV | -1.22 | -1.56 | -1.11 | -1.33 | 0 | 0 |
| PSV | 0 | 0 | 0 | 0 | 0 | 0 |

| **Vehicle category** | **Highways Agency. *Design Manual for Roads and Bridges. Vol. 12, Traffic Appraisal*** |
|---|---|
| car = | including taxis, estate cars and light vans with rear windows. |
| LGV = | Light Goods Vehicle<br>All goods vehicles up to 3.5 tonnes design gross vehicle weight. This includes light vans and light goods vehicles and equates approximately to up to 30 cwt unladen weight (in practice, vehicles with 2 axles fitted with a total of 4 tyres). |
| OGV = | Other Goods Vehicle<br>All goods vehicles over 3.5 tonnes design gross vehicle weight (in practice, vehicles with 2 axles fitted with more than 4 tyres, and vehicles with more than 2 axles. This equates approximately to greater than 30 cwt unladen weight). |
| OGV1 = | Vehicles less than 25 tonnes design gross vehicle weight identified as less than 4 axles. |
| OGV2 = | Vehicles greater than 25 tonnes design gross vehicle weight identified as 4 axles or more. |
| PSV = | Public Service Vehicle<br>Buses and coaches including works buses but not mini-buses excluding caravans or other types of car and trailer. |

Based on the above for an

average speed of v = 40 mph

(2007 efficiency values)

| Vehicle category | Fuel L (l/mile) | $CO_2$ emitted (kg/mile) |
|---|---|---|
| Diesel_car | 0.092 | 0.246 |
| Diesel_LGV | 0.137 | 0.369 |
| OGV1 | 0.437 | 1.176 |
| OGV2 | 0.610 | 1.640 |
| Petrol_car | 0.114 | 0.305 |
| Petrol_LGV | 0.163 | 0.437 |
| PSV | 0.362 | 0.972 |

**Figure J3** *Design Manual for Roads and Bridges* methodology for fuel consumption.[J1]

## J3.2 Ground conditions

The choice of foundation types and their geometry is dependent on the ground conditions at the site. The effect of the ground conditions on the foundations is considered by comparison of seven different types of ground profiles, presented in Table J5. Two distinct types of founding stratum have been assumed:

- stiff clay with a design undrained shear strength $c_u = 75 + 10z$ kN/m² (z = depth below top of stratum)
- sand/gravel with a design SPT N value of 25.

Four different thicknesses of ground unsuitable for load-bearing (eg made ground, very soft alluvial/organic clays, etc.) have been specified, as shown in Table J5. Generally, deep trench foundations are unlikely to be used for depths greater than 2.5 m, ie not for profiles 5 and 6. Trenches for profiles 3 and 4 are included in the calculations for comparison, although they would be unusual in practice.

Profile 0 (ground bearing clay from surface) has been introduced as the ground condition traditionally most favourable for selecting trench foundations.

Table J5

Definition of reference ground profiles*

| Thickness of unsuitable material (m) | Underlying material | |
|---|---|---|
| | Clay ($c_u = 75 + 10z$ kN/m²)† | Sand/gravel (SPT N = 25) |
| 0.0 | Profile 0 | – |
| 1.0 | Profile 1 | Profile 2 |
| 2.5 | Profile 3 | Profile 4 |
| 4.0 | Profile 5 | Profile 6 |

* Groundwater assumed 1 m below ground surface for all cases
† z, depth in metres below surface of clay
SPT: standard penetration testing.

## J3.3 Superstructure

A typical terraced house building was established, with floor areas and range of foundation loads as tabulated in Table J6.

Table J6

Summary of superstructure loads

| Type of house | Footprint plan area (m²) | Load/pile (kN) | |
|---|---|---|---|
| | | Maximum | Minimum |
| Terraced | 195 | 350 | 225 |

The carbon calculation comparison is made for three different foundation options (trench fill, bored piles, driven piles – see Fig. J6) for the typical terraced house option of Table J6. The following assumptions have been made for the calculations:

- calculations for carbon emissions exclude base slab
- base slab is assumed to be suspended for all foundation options
- groundwater is assumed 1 m below ground surface
- pumping for dewatering is not included in the calculations
- no overbreak is assumed for the excavations
- 2.5% material (concrete and steel) waste assumed for all cases.

Figures J4 and J5 show the layout of the typical terraced house foundations.

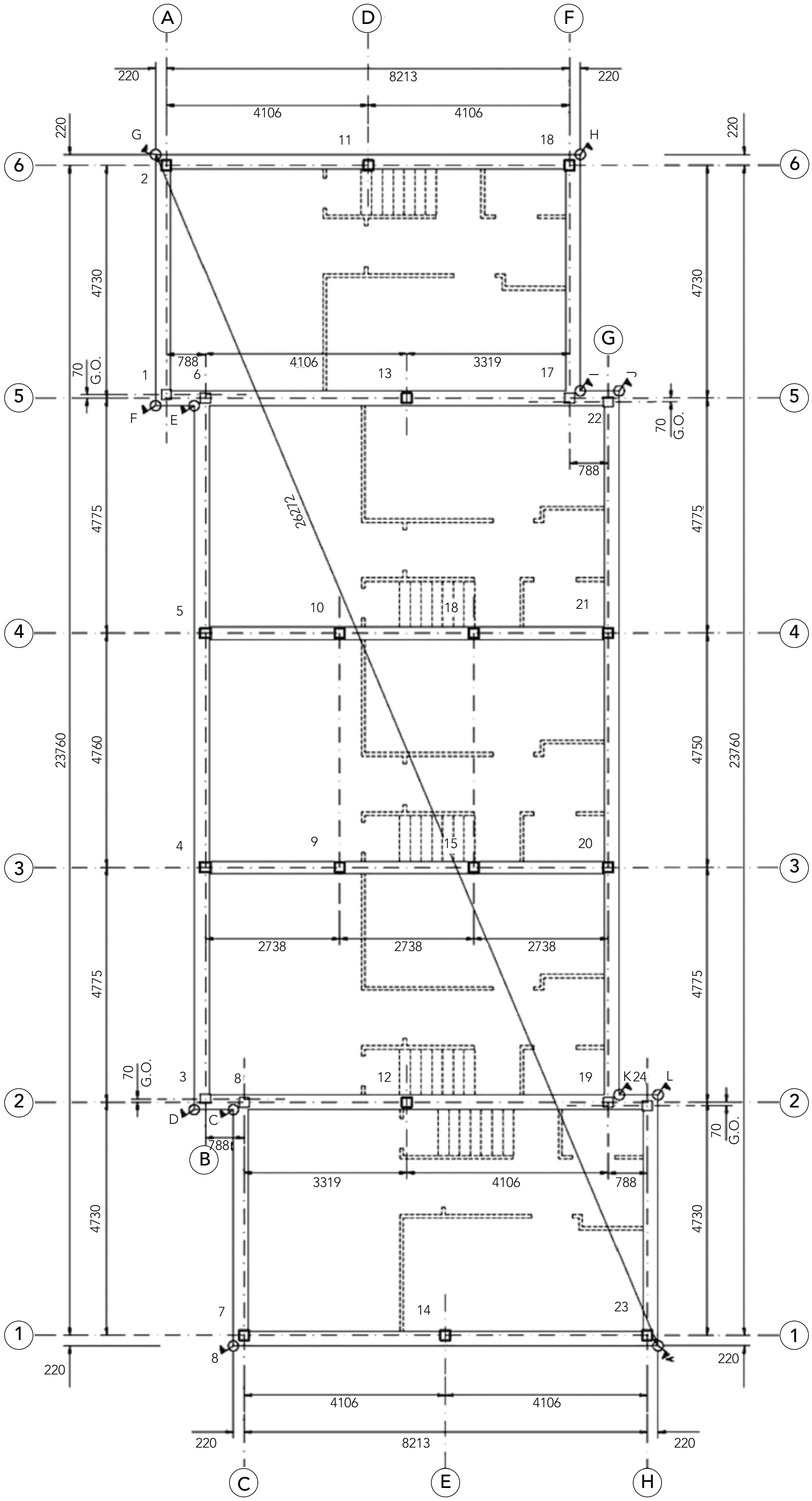


**Figure J4** Plan layout for typical terraced house. (Courtesy of Roger Bullivant)

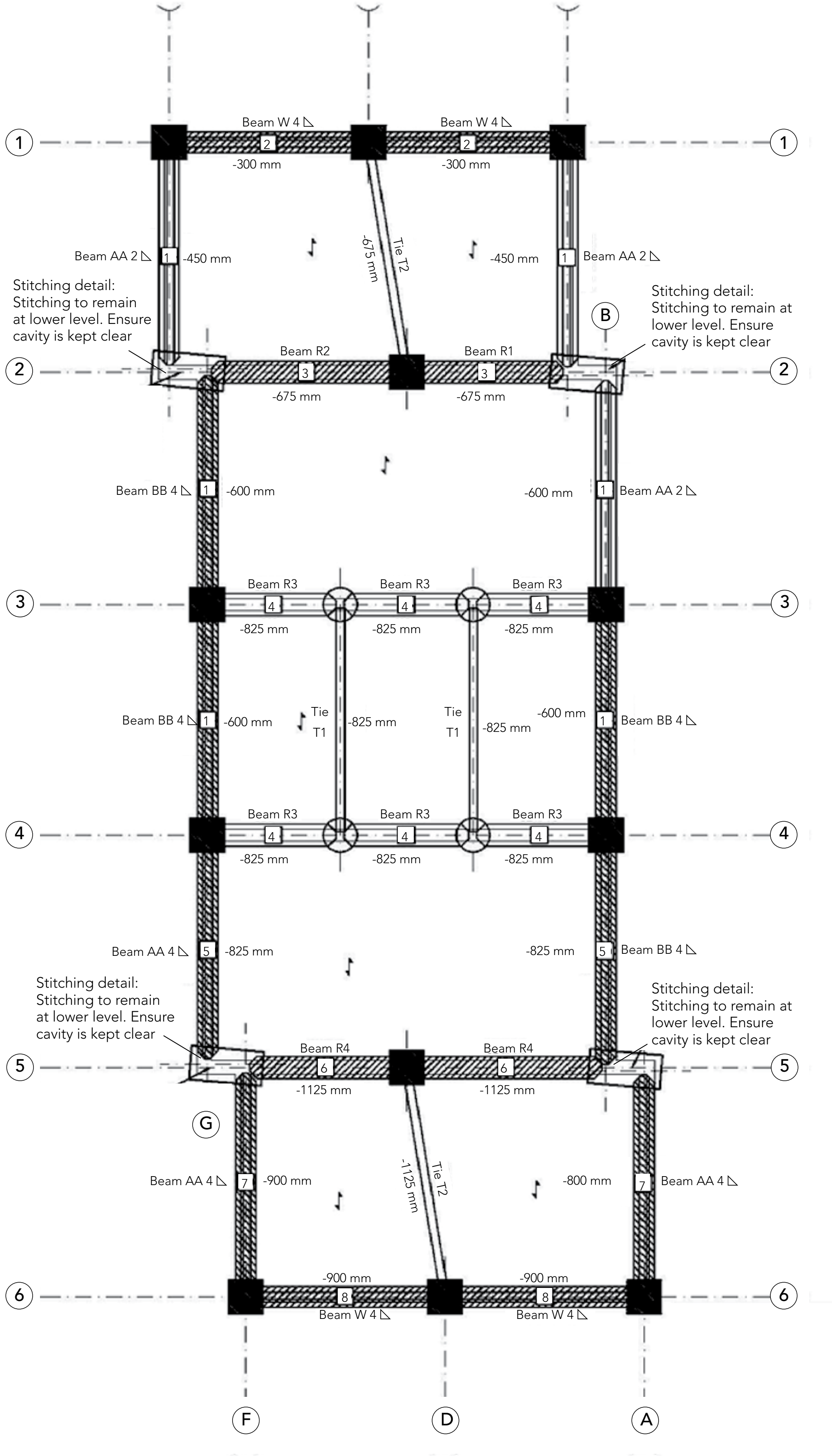



**Figure J5** Beam locations for typical terraced house. (Courtesy of Roger Bullivant)

## J3.4 Foundation geometry for terraced house

For the terraced house described above, three different types of foundation are examined:

- bored cast in situ concrete pile
- driven pre-cast concrete piles
- trenchfill.

Figure J6 compares the three foundation options and the reference line to which $CO_2$ is calculated (indicated by red hatched line in Figure J6). The two piled options include emissions from the ground beams construction. Table J7 summarises the foundation geometries derived for the seven different ground conditions. Table J8 summarises the geometries of the ground beams used with the piling options.

**Table J7**

**Foundation geometry for terraced house option**

| Made ground thickness (m) | | 0 | 1 | 2.5 | 4 | 1 | 2.5 | 4 |
|---|---|---|---|---|---|---|---|---|
| Founding stratum | | Clay $c_u = 75 + 10z$ (kN/m$^2$) | | | | Sand SPT N = 25 | | |
| Ground condition | | GC 0 | GC 1 | GC 3 | GC 5 | GC 2 | GC 4 | GC 6 |
| Bored piles | Pile length (m) | 11.3 | 12.3 | 13.8 | 15.3 | 12.6 | 12.8 | 13.1 |
| | Pile diameter (m) | 0.3 | | | | | | |
| | No. | 24 | | | | | | |
| | Pile cap height (m) | 0.38 | | | | | | |
| | Pile cap width (m) | 0.9 | | | | | | |
| Driven piles | Pile length (m) | – | 13.5 | 15 | 16.5 | 14.5 | 14.5 | 15 |
| | Pile size – square section (m) | 0.2 | | | | | | |
| | No. | 24 | | | | | | |
| | Pile cap height (m) | 0.38 | | | | | | |
| | Pile cap width (m) | 0.6 | | | | | | |
| Deep trench | Trench width B (m) | 0.75 | 0.75 | 0.75 | N/A | 0.6 | 0.6 | N/A |
| | Trench depth D (m) | 1 | 2 | 3 | N/A | 2 | 3 | N/A |
| | Length (m) | 97 | | | | | | |
| | Brick block width (m) | 0.45 | | | | | | |
| | Brick block height (m) | 0.225 | | | | | | |
| | Excavation to top of trench (m$^3$) | 26.1 | | | | | | |

**Table J8**

**Ground beam geometry (present only in the piling options)**

| Beam reference | Depth (m) | Width (m) | Main reinforcement | Shear reinforcement | Span (m) | No. of beams |
|---|---|---|---|---|---|---|
| R1-3 | 0.450 | 0.45 | 4B20 | 8 mm links @200 mm c/c | 3.3 | 1 |
| R1-6 | 0.450 | 0.45 | 4B20 | 8 mm links @200 mm c/c | 3.3 | 1 |
| R2-3 | 0.450 | 0.45 | 4B25 | 10 mm links @200 mm c/c | 4.1 | 1 |
| R3-4 | 0.375 | 0.45 | 4B20 | 8 mm links @200 mm c/c | 2.7 | 6 |
| R4-6 | 0.450 | 0.45 | 4B25 | 10 mm links @200 mm c/c | 4.1 | 1 |

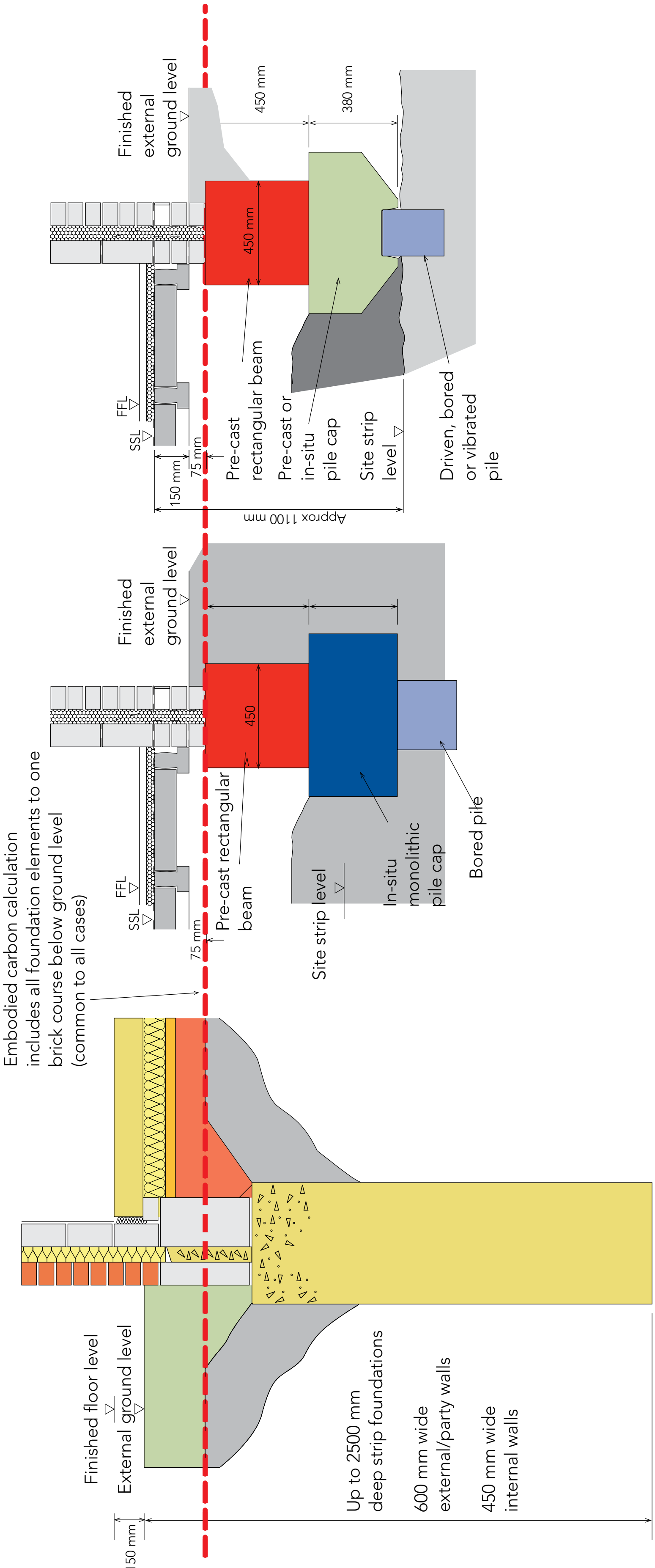



**Figure J6** Foundation types considered.

## J4 Results

The emissions from the three foundation options of bored and driven piles and deep trenches are presented in the tables J9 to J11. Figure J7 shows a bar chart of the results.

**Table J9**

**Embodied carbon results for the bored piled foundation option**

| Ground condition | Embodied carbon [t $CO_2$] | | | | | | |
|---|---|---|---|---|---|---|---|
| | GC 0 | GC 1 | GC 3 | GC 5 | GC 2 | GC 4 | GC 6 |
| Materials | 12.90 | 13.50 | 14.40 | 15.30 | 13.70 | 13.80 | 14.00 |
| Transportation | 1.90 | 2.00 | 2.00 | 2.10 | 2.00 | 2.00 | 2.00 |
| Installation | 4.57 | 4.60 | 4.60 | 4.63 | 4.58 | 4.58 | 4.60 |
| Concrete | 10.40 | 10.90 | 11.70 | 12.60 | 11.10 | 11.20 | 11.40 |
| Steel | 2.70 | 2.80 | 3.00 | 3.20 | 2.90 | 2.90 | 2.90 |
| Excavation | 5.36 | 5.39 | 5.40 | 5.43 | 5.39 | 5.39 | 5.39 |
| Fill | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |

**Table J10**

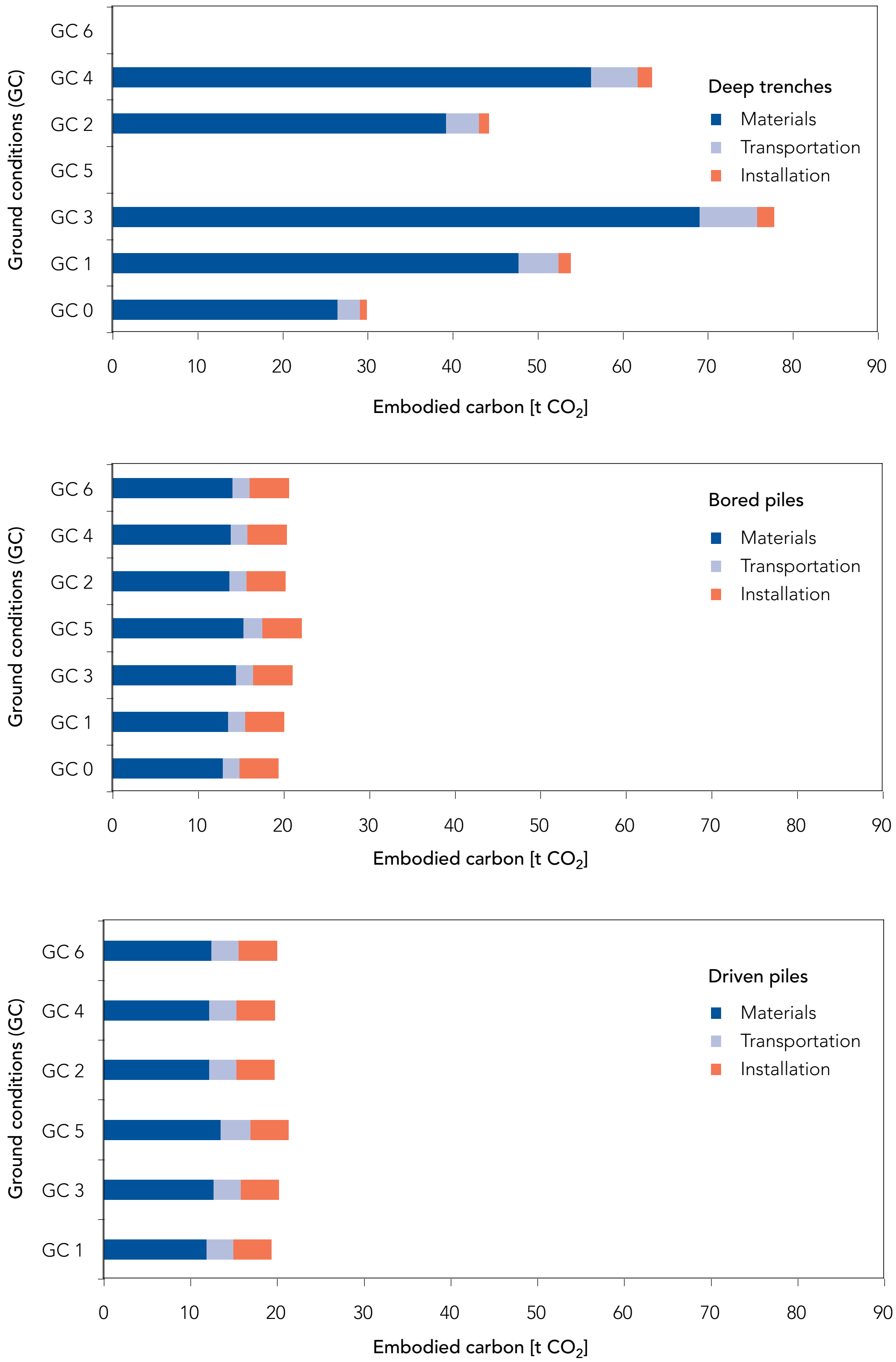
**Embodied carbon results for the driven piled foundation option**

| Ground condition | Embodied carbon [t $CO_2$] | | | | | | |
|---|---|---|---|---|---|---|---|
| | GC 0 | GC 1 | GC 3 | GC 5 | GC 2 | GC 4 | GC 6 |
| Materials | Not calculated | 11.8 | 12.7 | 13.5 | 12.2 | 12.2 | 12.4 |
| Transportation | | 3.1 | 3.1 | 3.4 | 3.1 | 3.1 | 3.1 |
| Installation | | 4.4 | 4.4 | 4.4 | 4.4 | 4.4 | 4.4 |
| Concrete | | 11.6 | 12.4 | 13.4 | 12.1 | 12.2 | 12.4 |
| Steel | | 2.0 | 2.1 | 2.2 | 1.9 | 1.9 | 1.9 |
| Excavation | | 5.1 | 5.1 | 5.1 | 5.1 | 5.1 | 5.1 |
| Fill | | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 |

**Table J11**

**Embodied carbon results for the trenchfill foundation option**

| Ground condition | Embodied carbon [t $CO_2$] | | | | | | |
|---|---|---|---|---|---|---|---|
| | GC 0 | GC 1 | GC 3 | GC 5 | GC 2 | GC 4 | GC 6 |
| Materials | 26.4 | 47.7 | 69.0 | N/A | 39.2 | 56.2 | N/A |
| Transportation | 2.6 | 4.7 | 6.7 | | 3.9 | 5.5 | |
| Installation | 0.8 | 1.4 | 2.0 | | 1.2 | 1.6 | |
| Concrete | 23.3 | 46.5 | 69.7 | | 37.2 | 55.8 | |
| Brick | 4.5 | 4.5 | 4.5 | | 4.5 | 4.5 | |
| Excavation | 1.2 | 1.9 | 2.6 | | 1.6 | 2.2 | |
| Fill | 0.9 | 0.9 | 0.9 | | 0.9 | 0.9 | |




| Ground condition | GC 0 | GC 1 | GC 3 | GC 5 | GC 2 | GC 4 | GC 6 |
|---|---|---|---|---|---|---|---|
| Made ground thickness (m) | 0 | 1 | 2.5 | 4 | 1 | 2.5 | 4 |
| Founding stratum | Clay | | | | Sand | | |
| Properties | cu = 75 + 10z (kN/$m^2$) | | | | SPT N = 25 | | |

**Figure J7** Carbon emission results.

**Table J12**

Comparison of total $CO_2$ emitted for the three foundation options

| Ground profile | | Embodied $CO_2$ (t) | | |
|---|---|---|---|---|
| | | Trenchfill | Bored piles | Pre-cast driven piles |
| Clay | GC 0 | 29.9 | 19.4 | Not calculated |
| | GC 1 | 53.8 | 20.1 | 19.3 |
| | GC 3 | 77.7 | 21.0 | 20.1 |
| | GC 5 | Not calculated | 22.1 | 21.2 |
| Sand | GC 2 | 44.2 | 20.3 | 19.6 |
| | GC 4 | 63.4 | 20.4 | 19.7 |
| | GC 6 | Not calculated | 20.6 | 19.9 |

## J5 Conclusions

The results of the study show that:

- The majority of $CO_2$ is locked in the material; the emissions contribution from the transportation and installation is significantly smaller.
- Construction of deep trenches emits more carbon even for the most favourable ground conditions GC 0 (clay from surface). This is a direct result of greater volume of concrete used in this option compared to the piling solutions.
- Bored piles and driven piles have similar emissions. The numbers may vary depending on specific site conditions and contractor's practices.

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# APPENDIX K

# Discussion of the use of geothermal piles for low-rise housing

## K1 Introduction

This Appendix discusses the use of geothermal piles to provide space and hot water heat energy for low-rise domestic housing. Geothermal piles are defined as load-bearing piles with an inbuilt closed-loop heat exchanger.

The following sections provide an introduction to the concepts of geothermal energy and geothermal piles. The general issues relating to the performance of geothermal piles as ground heat exchangers and as load-bearing foundations and a summary of the various contractual issues are presented, specifically that of design responsibility, competence in construction, and system commissioning.

The rationale for incorporating geothermal pile systems into domestic housing is provided by an introduction to the Code for Sustainable Homes.[K1] A discussion is presented describing how geothermal pile systems may help to meet the standard requirements of high Code Levels, and a comparison is made with data for biomass boilers and conventional gas boilers. A comparison of the potential financial cost of an geothermal pile system for low-rise domestic housing is also briefly compared with that of biomass boilers and conventional gas boilers.

### K1.1 Geothermal energy and geothermal piles

The ground can be used, in conjunction with a ground source heat pump (GSHP), to provide heating or cooling to a building. The thermal mass of the ground is used as either a heat sink (when cooling a building) or heat source (when heating a building). For more detail on the theory of ground-sourced energy and the design of geothermal systems, refer to the introductory text by Banks.[K2]

Geothermal energy exchange systems have been increasingly incorporated into building foundations since the 1980s. These energy foundations, as they are known, take advantage of the high thermal storage capacity of concrete in the pile and the surrounding ground. Concrete is an ideal energy-absorbing medium because it has a high thermal conductivity and thermal capacity. A summary of the varying types of energy foundations that have been developed for buildings, including geothermal piles, principally throughout mainland Europe is provided by Brandl.[K3]

In geothermal piles, one or more HDPE (high density polyethylene) pipes of between 20 and 32 mm in diameter are looped within the concrete in each pile. The fluid which circulates within these pipes typically comprises water, an anti-freeze solution, or a saline solution.

The issues concerning the performance of geothermal piles can be separated into two broad categories. The first category deals with the performance of the geothermal pile as an exchanger of heat energy. The second considers the performance of the geothermal pile as a load-bearing foundation.

## K2 Heat exchanger performance

### K2.1 General design principles

Banks[K2] has summarised that the performance of a closed-loop heat exchanging system will depend upon:

- thermal ground conditions (ie the thermal conductivity, specific heat capacity and initial ground temperature)
- the heating demand (ie the operational pattern of the scheme and the complexity of the heating and cooling loads)
- spacing of the geothermal piles.

Banks identifies the basic principles which underpin the design of closed-loop ground energy systems.

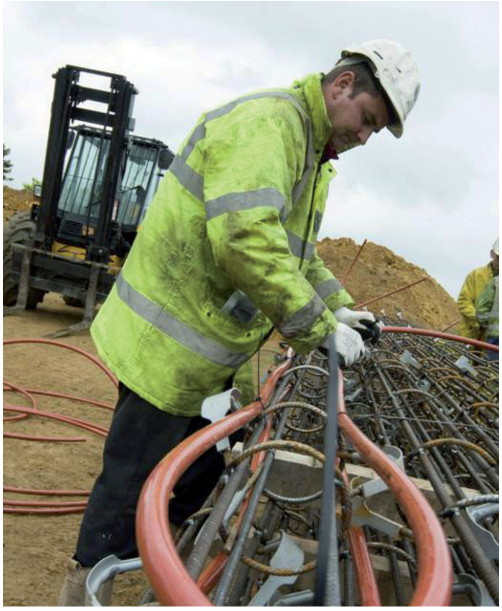

**Figure K1** HDPE pipework being fitted to the reinforcement cage for a pile.

For energy exchange efficiency, long and small diameter piles are generally preferred since longer piles provide a larger thermal mass of soil below the house with which the pile can exchange heat energy. It is possible that some of the house piles will not be used as geothermal piles. Ultimately, the number of piles that are incorporated into the geothermal pile system will depend on the pile spacing and the heat energy demands of the house.

### K2.2 Ground conditions

Some knowledge of underground thermal properties is necessary for the effective design of the geothermal pile heat exchangers. The most important parameters are the initial temperature and thermal conductivity of the ground. Although these are site specific, for smaller systems, general rules of thumb may be used to assign these parameters.

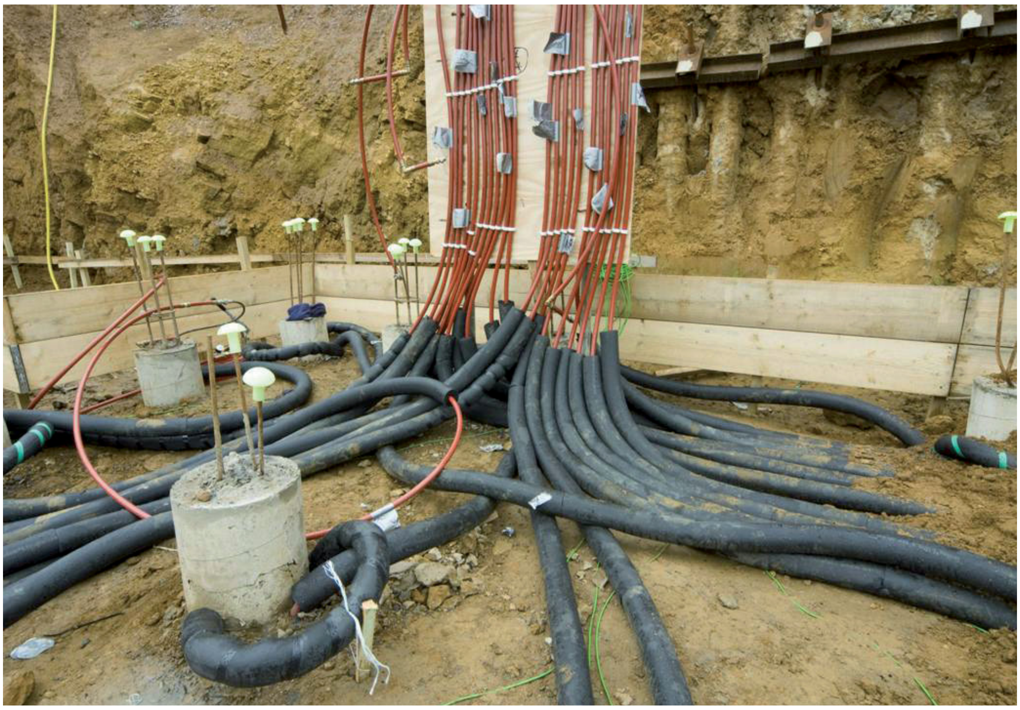

**Figure K2** Following installation of the reinforcement cage and pipework and the casting of the pile.

Conventional ground investigations, which are carried out to assess the ground properties for the pile designs, can be extended to include geothermal sampling and testing, including:

- laboratory test procedures for determining ground thermal properties such as those described by Clarke et al.[K4]
- thermal response tests which may provide more cost-effective and technically proficient designs for larger geothermal pile heat exchanger schemes. Recommended test specifications, originally listed by Kavanaugh,[K5, K6] are summarised by the American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE).[K7]

## K3 Load-bearing performance

### K3.1 European geothermal pile design experience

There is extensive experience of geothermal pile performance within Europe over the last 20 years.[K3] This provides observational evidence that the load-bearing performance of energy foundations, including geothermal piles, is not adversely affected by the heating and cooling cycles imposed upon them by GSHP systems.

One particular precautionary principle, however, is that the pile must not be allowed to freeze. If the coolant is circulated at temperatures below freezing point, then it will be necessary to demonstrate that the freezing front does not reach the soil interface. It is recommended that geothermal pile fluid circulation temperatures range from ambient ground temperatures down to no less than 2°C.

### K3.2 Concrete stresses and strains

Trials have shown that changes in axial stress in the pile remain within the acceptable stress limit for concrete[K8] due to the combined effect of axial load and the contraction/expansion effect caused by pile cooling/heating.

The trials also derived a coefficient of unrestrained thermal expansion $8.5 \times 10^{-6}$ m/m/°C). Using this figure, an increase of 10°C in a pile 13 m long would cause an unrestrained pile expansion of about 1 mm.

Pile movements due to temperature changes should be considered in addition to pile movements due to the loading in assessing acceptable limits for foundation design.

### K3.3 Shaft friction

The thermal contraction and expansion of the pile may also mobilise shaft friction between the soil and pile. The results from trials show that for the relatively short house foundation geothermal piles and the normal operational temperature ranges of the pile, the cyclic shaft friction strains are within the elastic range.

### K3.4 Long-term consolidation effects

Cyclic cooling and heating of the pile and the surrounding ground can cause thermal contraction and expansion of the soil volume and lead to changes in horizontal total stress in the ground. Increases in total stress could in turn lead to increases in pore water pressures in clay soils. Pore water may also dissipate over a number of years, reducing the volume of the clay around the piles, and reducing the horizontal effective stress in the ground in the long term. Reductions in the horizontal effective stress could lead to reductions in the pile shaft friction, or the soil volume reduction could lead to negative skin friction effects in normally consolidated soils.[K9]

Research is currently underway to investigate these issues, and UK trials are ongoing to examine the potential impact of these long-term consolidation effects. However, over the last 20 years there have been no recorded evidence of reduced pile capacity in European case studies.[K3]

# K4 Contractual issues

## K4.1 Design responsibility

The design phase of geothermal pile system commissioning requires a thorough site survey and characterisation, accurate load modelling, and reasonable assurance that the design chosen meets the design intent.

For geothermal foundations contractual issues may be more complicated than for closed-loop borehole systems. The load-bearing performance of the geothermal pile needs to be considered as well as the performance of the pile as a heat exchanger. This problem may be mitigated by having the same contractor take responsibility for both the pile capacity and the geothermal system design.

The temperature changes in the pile and ground should be assessed by the geothermal system designer. This information should be provided to the pile designer so that he can assess the effect of the temperature on the pile capacity and concrete stresses. The pile capacity allowing for thermal effects should remain within the conventional factors of safety for pile design.

The temperature of the circulating fluid can be monitored at the heat pump outflow and return pipes to ensure that the temperatures remain within the limits provided by the geothermal system designer. Additional instrumentation can be provided in the piles to monitor specific pile behaviour.

## K4.2 Competence in construction

Certified training courses and accreditations for the design and construction of ground source energy systems are not yet available in the UK.

There are several areas of demonstrable competence relevant to the design and installation of ground-sourced heat pump systems which are relevant to geothermal pile systems. A summary of the competences and list of possible staff qualifications that could be sought are provided in Clause 5 and Appendix A, respectively, of the Microgeneration Installation Standard: MIS 3005.[K10]

## K4.3 System commissioning

ASHRAE[K7] provides information on the various tasks and participants involved in the commissioning of ground source heat pump systems. These could be applied to geothermal pile systems. The roles for the participants include the checking of numerous system functions, and witnessing of those checks. The participants with responsibility may include the architect/engineer, contractor, manufacturer, commissioning authority, or owner. A contractor will also be required to test, adjust, and balance the system.

# K5 Code for sustainable homes

## K5.1 Introduction to the Code

The Code for Sustainable Homes was launched in December 2006 with the publication of 'Code for Sustainable Homes: a step change in sustainable home building practice' (since withdrawn) and came into effect in April 2007. The latest version was released in February 2008[K11] and sets out the assessment process and the performance standards required by homes. It is designed to provide a single national standard to cover aspects of sustainable design and construction of homes.

The Code measures the sustainability of a home against nine design categories, rating the 'whole home' as a complete package. Points are scored against each of these categories to generate a total credit score. The design categories are:

- energy and $CO_2$ emissions
- pollution
- water
- health and wellbeing
- materials
- management
- surface water run-off
- ecology
- waste.

As well as an overall points score, minimum standards at each level of the code are required specifically for the categories of energy and $CO_2$ emissions and water.

The energy and $CO_2$ emissions targets will be imposed via the Building Regulations, with interim steps defined by Code Levels 3, 4, 5 and 6, which seek defined improvements on Part L 2006 Building Regulations. Credits are awarded based on the percentage improvement in the dwelling emission rate (DER), (estimated $CO_2$ emissions in kg per $m^2$ per annum arising from energy use for heating, hot water, and lighting for the actual dwelling), over the target emission rate (TER) (the maximum emission rate permitted by Building Regulations), for the dwelling where DER and TER are as defined in AD L1A 2006 edition of the Building Regulations.[K12]

Table K1 sets out the improvement of DER over TER and the corresponding level of the Code.

**Table K1**

**Criteria for the Code Levels for energy and carbon dioxide emissions, summarised from the Code for Sustainable Homes[K13]**

| Code Level | Percentage improvement of DER over TER |
|---|---|
| 3 | ≥ 25% |
| 4 | ≥ 44% |
| 5 | ≥ 100% |
| 6 | Zero carbon home |

DER, dwelling emission rate; TER, target emission rate.

It is mandatory for all new homes to have a rating against the Code. A 25% improvement on Part L Building Regulations (Code Level 3) will be required in England by 2010 and by 44% (Code Level 4) in 2013. There is no time frame for the implementation of Code Level 5, so this will not be discussed further.

Current government targets mean that new homes in England will be required to meet the requirements of Code Level 6 of the Code for Sustainable Homes by 2016. Code Level 6 is defined as a zero carbon home. There are additional requirements to achieve this but it equates to an approximate 145% improvement on TER. A zero carbon home is one that generates as much power as it uses over the course of a year and therefore has net zero $CO_2$ emissions.

Similar targets are also being established for Wales, Scotland, and Northern Ireland.

The following section discusses the design issues relating to the use of geothermal piles to help achieve these higher Code Levels.

### K5.2 Achieving the Code Levels using geothermal piles

With regards to achieving the Code Levels as defined in the Code for Sustainable Homes, the energy which is transferred from the ground to the building is considered renewable. The electricity used to transfer energy from the ground to the building (ie the electricity used to drive the circulation pumps, heat exchangers, etc.) must be considered as non-renewable unless the electricity is from a renewable source.

Modelling work has been undertaken to calculate whether geothermal piles, installed for a mid-terrace low-rise domestic dwelling, could offer a method of providing sustainable heating and hot water to the home. Based on the results of this modelling, which are presented in Table K2, it can be concluded that depending on the standard of insulation specified, as defined by use of different heat loss parameters (HLPs), the use of geothermal piles could enable houses to comply with the Code for Sustainable Homes Levels 3 or 4.

The $CO_2$ emission savings of the geothermal pile system were compared against a conventional gas boiler and a biomass boiler. The results for terraced houses with different HLPs are summarised in Table K2.

The government has commissioned analysis of various carbon compliance options.[K7] This showed that ground-sourced heat pumps in a typical mid-terrace house insulated to advanced practice energy efficiency (APEE) and best practice energy efficiency (BPEE) could achieve 46% and 29% carbon reduction respectively versus Part L Building Regulations. The 25% improvement on Part L Building Regulations (Code Level 3), which is required in 2010, could be achieved by ground source energy foundations for both insulated property types. The 44% improvement (Code Level 4), which is required in 2013, could be achieved by ground source energy only for the APEE-insulated property type.

It should be noted that the heat pump $CO_2$ savings reported here are based on the results of the 2006 version of the UK Government's Standard Assessment Procedure (SAP) for the energy rating of dwellings. This currently provides a beneficial weighting for electrically fuelled heating, such as that provided by heat pumps. The 2010 revision to the Building Regulations may reduce or remove this weighting, which will significantly reduce the notional $CO_2$ savings possible.

**Table K2**

**$CO_2$ savings compared with the Part L 2006 target emission rates (determined using the SAP methodology)**

| | House 1 (HLP = 1.05 $W/m^2 K$) | House 2 (HLP = 0.8 $W/m^2 K$) |
|---|---|---|
| Condensing gas boiler | 12% (Code 2) | 28% (Code 3) |
| Geothermal piles and heat pump with immersion top up | 35% (Code 3) | 42% (Code 3) |
| Geothermal piles and heat pump without immersion top up | 48% (Code 4) | 55% (Code 4) |
| Biomass boiler | 70% (Code 4) | 72% (Code 4) |

## K6 Cost comparison

The relative cost of installing a ground source heating system has been assessed for a mid-terrace building. The following cases have been considered:

- an geothermal pile system comprising a heat pump, between three and four geothermal piles, and underfloor heating
- a gas boiler with radiators
- a gas boiler with underfloor heating
- a biomass boiler with radiators.

A summary of the findings are provided in Table K3. The cost analysis shows that, for both housing types, although geothermal piles are significantly more expensive to install than a high-efficiency condensing gas boiler solutions, they may prove cheaper to install than biomass boiler solutions.

Table K3

Average heating system installation costs determined for a typical mid-terrace house

| | Gas boiler + radiators (£) | Gas boiler + underfloor heating (£) | Ground source heat pump + geothermal piles + underfloor heating (£) | Biomass boiler + radiators (£) |
|---|---|---|---|---|
| House 1 (HLP = 1.05) | 3500 | 3400 | 12 100 | 14 900 |
| House 2 (HLP = 0.8) | 3400 | 3200 | 11 400 | 14 900 |

In a separate government commissioned study,[K13] an analysis of various carbon compliance options for a range of onsite technology solutions that could be applied in different development and dwelling types was undertaken. The results for a typical mid-terrace house showed that the cost of installation for a closed-loop ground source energy system (not geothermal piles) was £12 430 and £10 550 in houses constructed to APEE and BPEE insulation standards, respectively. The cost of biomass boilers was more expensive, at £13 600 and £17 180 respectively.

## K7 Conclusions

Care should be taken in the design of geothermal pile systems, particularly in assigning design responsibility. European experience has shown that the load-bearing performance of geothermal piles is not adversely affected by the cyclic heating and cooling of piles, as long as they are operated within a normal temperature range (eg from ambient ground temperatures down to 2°C). As a precaution, it is advised that freezing of the pile should be avoided.

With regard to the Code for Sustainable Homes, Code Level 4 for energy and $CO_2$ emissions is almost certainly achievable using geothermal piles. It seems unlikely that Code Levels 5 and 6 could be achieved by geothermal pile systems unless either the electricity grid is decarbonised (ie the production of electricity to supply the national grid releases zero $CO_2$ into the atmosphere) or on-site generation of renewable electrical energy is used to power the geothermal pile heat pump.

The cost of installation of an geothermal pile heating system may prove to be more cost effective than a biomass boiler system.

Although biomass boilers may be able to offer slightly greater $CO_2$ savings against the Part L Building Regulation requirements than geothermal piles with ground source heat pumps, the actual Code Levels that can be achieved by installation are broadly equivalent.

The 2010 revision to the Building Regulations may reduce or remove the beneficial weighting for electrically fuelled heating, such as that provided by heat pumps, which would significantly reduce the notional $CO_2$ savings possible using this technology.

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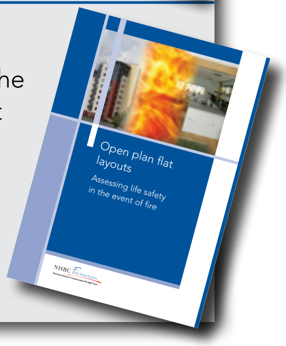
# NHBC Foundation publications

Water efficiency in new homes This guide, specifically intended for the smaller builder, provides an introduction to water efficiency. It outlines the standards being encouraged by the Code for Sustainable Homes, the Building Regulations and the Water Efficiency Calculator for New Dwellings. The technologies used to achieve water efficiency – ranging from simple tap flow restrictors all the way through to greywater recycling systems – are described, together with some key issues associated with each. ***NF20*** October 2009


Open plan flat layouts – assessing life safety in the event of fire This research report is the result of a study examining the options for satisfying the requirements of the Building Regulations. It addresses layout, size, travel distances, enhanced detection options and sprinkler use. In addition it addresses the human implications, including the various reactions, wake up and response times from people occupying the building. ***NF19*** August 2009




Indoor air quality in highly energy efficient homes – a review
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Zero carbon homes – an introductory guide for housebuilders
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# Efficient design of piled foundations for low-rise housing

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A review was undertaken during the preparation of this guidance to gain knowledge relating to current practice and to identify areas of general concern in relation to the selection of foundation types, ground investigation practice, and design and construction methodologies adopted. A brief review of the surveys and the main conclusions is presented in Appendix A.

Further details to illustrate or expand on the information discussed in the main body of the text are provided in a series of appendices.

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NF21
Published by IHS BRE Press on behalf of the NHBC Foundation
February 2010
ISBN 978-1-84806-106-4