

Code of practice for
Foundations —
(Formerly CP 2004)

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- Concrete Society
- Department of the Environment, Housing and Construction Industries
- Department of the Environment, Property Services Agency
- Department of Transport, Highways
- Federation of Civil Engineering Contractors
- Health and Safety Executive
- Institution of Civil Engineers
- Institution of Structural Engineers
- Royal Institute of British Architects
- Co-opted members

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Publications referred to	Inside back cover

Foreword

This British Standard, having been prepared under the direction of the Civil Engineering and Building Structures Standards Committee, supersedes CP 2004:1972 which is withdrawn.

The original Civil Engineering Code of Practice No. 4 “Foundations” was prepared by a committee convened by the Institution of Civil Engineers on behalf of the Codes of Practice Committee for Civil Engineering, Public Works, Building and Construction Work, under the aegis of the former Ministry of Works, for publication in the Civil Engineering and Public Works Series.

In 1949 the responsibility for the preparation and issue of Codes of Practice of the Civil Engineering and Public Works Series was handed over to the following four professional Engineering Institutions: the Institution of Civil Engineers, the Institution of Municipal Engineers, the Institution of Water Engineers and the Institution of Structural Engineers, and the arrangements for the preparation and publication of these codes were made by the Civil Engineering Codes of Practice Joint Committee constituted by these institutions. The revised Code of Practice No. 4 “Foundations” was published by the Institution of Civil Engineers in 1954 for the Joint Committee. The responsibility of the Joint Committee passed in 1961 to the British Standards Institution.

The knowledge gained from the structural behaviour of high-alumina cement concrete during 1973 and 1974 has considerably altered the understanding of the behaviour and sensitivity of the use of this material in structures. As a result of both research and experience there is not, at this time, the consensus of opinion necessary for agreement on recommendations for the use of this material.

Consequently, an engineer wishing to specify concrete made with high-alumina cement has to rely entirely on his own engineering judgement, based on his own experience in the use of this particular cement without guidance from this code of practice.

There has always been some controversy in the matter of rock classification. While this code has been updated it is arguable as to whether it goes far enough in respect of allowable bearing capacities. In consequence of this, appendix D has been introduced in order to expand this subject.

SI units are used throughout this code. However, having regard to the many countries in Europe and the rest of the world where the code may be read, and where metric units or imperial units are in use, the committee considered it essential to include imperial units and/or other metric units alongside SI units wherever necessary to facilitate the widest understanding of the code.

Attention is drawn to the Health and Safety at Work etc. Act which came into force in 1974 and the associated Construction Regulations, made under the Factories Act 1961. The health and safety of persons engaged in carrying out the works to which this code refers is the concern of everyone associated with the design and construction of the works. Additional references are made to the various publications that complement the Construction Regulations and that, in some cases, offer an explanatory text pertaining to statutory requirements. These documents are essential reading to effect the necessary attitude of mind which ensures safety in construction operations. These publications and the Statutory Instruments are given in appendix D.

This revision of CP 2004:1972 has been prepared by practising foundation engineers. It attempts to take into account consensus opinions on the often wide range of solutions to the day to day problems that face those having to take decisions on the foundation works. Ground being an anisotropic medium needs a solution which in practice is more often art than science. This aspect is stressed to attract the attention of the user of the code to take nothing for granted and to work from commonsense principles to provide a data base upon which he can consider the most appropriate and cost effective solution for the satisfactory completion of the Works.

The revision does not attempt, nor is it the purpose of the code, to be a manual of foundation engineering but provides sources of information on some of the ways and means whereby work in the ground can take advantage of the scope of available alternatives. There are several standards which have relevance to foundation work and careful regard should be given by the user to the cross-references in this code to ensure that his search is adequate for his purposes.

Although this revision is based on the application of established practice, technological advances in design and construction methods relevant to each section should be assessed by the user to ensure that the latest proven methods are given adequate consideration.

The code has been redrafted with the help of many professional engineers from academia, research and industry.

The full list of organizations who have taken part in the work of the Technical Committee is given on the back cover. The Chairman of the committee is Mr M W Leonard and the following have made a substantial contribution in the drafting of the code:

Mr R W Cooke
Mr D W Divall
Mr C K Haswell
Mr A L Little
Mr D J Palmer
Mr J Paterson
Mr M J Tomlinson

This revision has given particular emphasis to safety in foundation works and a new section covering safety precautions has been introduced. In particular, the revision omits the allowance previously made to the adoption of increased stresses in temporary works. Particular acknowledgement is made to the inspectors of the Health and Safety Executive for their assistance in preparing this section.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to x, pages 1 to 186, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

Section 1. General

1.1 Scope

This British Standard code of practice gives recommendations for the design and construction of foundations for the normal range of buildings and engineering structures.

Section two covers the general principles of design; sections three, four, five and seven are concerned with more detailed considerations of the design and installation of the main types of foundations.

Sections six, eight and nine relate to site operations and construction processes involved in foundation engineering and section ten describes the factors affecting the durability of the various materials used in foundation structures. Section eleven covers safety precautions. The standard does not cover foundations for special structures.

NOTE 1 For non-industrial structures of not more than four storeys, reference should also be made to CP 101.

NOTE 2 Detailed information for the design and construction of foundations for reciprocating machinery is given in CP 2012-1.

NOTE 3 The titles of the publications referred to in this standard are listed on the inside back cover.

1.2 Definitions

For the purposes of this British Standard the following definitions apply.

1.2.1

adjoining owner

the freeholder, owner, lessee, tenant or authorized occupier of an adjoining property

1.2.2

air lock

a chamber used in compressed-air work, having one door to the open air and another to the space under compressed air. It may be filled with compressed air or reduced to atmospheric pressure to enable men and/or materials to be passed to or from the compressed-air space while maintaining pressure in the latter

1.2.3

allowable load

the load that may be safely applied to a pile after taking into account its ultimate bearing capacity, negative friction, pile spacing, overall bearing capacity of the ground below the piles and allowable settlement

1.2.4

allowable net bearing pressure

the maximum allowable net loading intensity at the base of the foundation, taking into account the ultimate bearing capacity, the amount and kind of settlement expected and the ability of the structure to accommodate this settlement

NOTE It is a combined function both of the site conditions (including all construction in the vicinity) and the characteristics of the proposed structure.

1.2.5

anvil

the part of a power-operated hammer which receives the blow of the ram and transmits it to the pile

1.2.6

barrettes

a system of piling involving the excavation of large rectangular panels under bentonite

NOTE The techniques used in their construction are very similar to those used for diaphragm walling.

1.2.7

bearing pile

a pile driven or formed in the ground for transmitting the weight of a structure to the soil by the resistance developed at the pile point or base and by friction along its surface

NOTE If the pile supports the load mainly by the resistance developed at its point or base, it is referred to as an end-bearing pile; if mainly by friction along its surface, as a friction pile.

1.2.8

bentonite suspension/slurry

a mixture of bentonite and water

1.2.9

boiling

the phenomenon of the soil being forced up from the base of an excavation by the upward flow of water or air, or of the soil being forced up from the toe of an embankment due to the pressure of water retained by the embankment

NOTE This phenomenon is sometimes called blowing.

1.2.10

bored cast-in-place pile

a pile formed with or without a casing by excavating or boring a hole in the ground and subsequently filling it with plain or reinforced concrete

NOTE Another form of construction is precast concrete sections that are grouted in place.

1.2.11

box pile

a pile made of rolled steel sections welded together to form a hollow pile

1.2.12

bracing

the internal system of walings, struts, and other members which enable a cofferdam to resist external pressures

1.2.13**building owner**

a person having the right to construct, demolish or modify a building, plant or other structure by virtue of legal ownership or other legal authority

1.2.14**caisson**

a structure that is sunk through ground or water for the purpose of excavating and placing work at the prescribed depth and which subsequently becomes an integral part of the permanent work

1.2.15**box caisson**

a caisson which is closed at the bottom but open to the atmosphere at the top

1.2.16**compressed air caisson**

a caisson with a working chamber in which the air is maintained above atmospheric pressure to prevent the entry of water and ground into the excavation

1.2.17**open caisson**

a caisson open both at the top and at the bottom

1.2.18**cased pile**

a pile formed with a permanent shell or casing filled with concrete

1.2.19**clay cutter**

a percussion boring tool used in clay soils

1.2.20**cleat**

a bracket or joining piece fixed to prevent movement of a strut or waling

1.2.21**cofferdam**

a structure, usually temporary, built for the purpose of excluding water or soil sufficiently to permit construction to proceed without excessive pumping and to support the surrounding ground

NOTE A land cofferdam is one constructed from a land surface; a water cofferdam is one constructed in open water.

1.2.22**cellular cofferdam**

a cofferdam consisting of a series of filled cells of circular or other shape in plan

1.2.23**double-wall cofferdam**

a cofferdam enclosed by a wall consisting of two parallel lines of sheeting tied together, with filling between them and which is usually self-supporting against external pressure

1.2.24**contiguous bored piling**

a system of retaining wall construction in which bored piles are installed in single or double rows so that they are touching or in close proximity to each other

1.2.25**crib**

a layer of timber or steel in either one or two directions placed beneath a column, shore, post, sill or soleplate for the purpose of spreading the load

NOTE This is also called a grillage.

1.2.26**cut-off level**

the prescribed elevation at which the top of a pile is cut. This may be above or below ground level

1.2.27**cylinder**

an alternative name for an open caisson or monolith of cylindrical form

1.2.28**dead shore**

a vertical strut supporting a horizontal member (see headtree) which is carrying the wall or other load from above

1.2.29**depth of cut-off**

the depth reached by the sheet piling or cofferdam walls below excavation level

1.2.30**diaphragm wall**

a system of construction in which a bentonite slurry is employed to maintain a pressure head and stabilize a trench being formed with boring or grabbing equipment. When the required depth has been reached the trench is concreted from the bottom upward, displacing the slurry for removal

1.2.31**displacement piles**

piles formed by displacement of the soil or ground through which they are driven

1.2.32**dolly**

a cushion of hardwood or other material placed on top of the driving cap or helmet to receive the blows of the hammer

1.2.33**downdrag (negative skin friction)**

a downwards frictional force applied to the shaft of a pile caused by the consolidation of compressible strata, e.g. under recently placed fill

NOTE Downdrag has the effect of adding load to the pile and reducing the factor of safety.

1.2.34**driven cast-in-place pile**

a pile installed by driving a permanent or temporary casing, and filling the hole so formed with plain or reinforced concrete

1.2.35**driven pile**

a pile driven into the ground by the blows of a hammer or by a vibrator

1.2.36**driving cap**

a temporary steel cap placed on top of a steel pile to distribute the blow over the cross section and to minimize damage to the pile head during driving

1.2.37**driving shoe**

a shoe or point fitted to the tip of a pile or driving tube to facilitate its penetration

1.2.38**drop hammer**

a weight, usually of metal, raised by a winch and allowed to fall under gravity

1.2.39**drop or stroke**

the distance which the hammer or ram is allowed to fall

1.2.40**enlarged base**

an enlargement of the base area of a pile, formed either

- a) with a base larger than the shaft of a pre-formed pile, or
- b) in situ, by driving a plug of concrete into the surrounding ground, or
- c) in situ, by undercutting (under-reaming) the soil at the base of a bored pile.

1.2.41**factor of safety**

the ratio of the ultimate bearing capacity to the intensity of the applied bearing pressure or the ratio of the ultimate load to the applied load

1.2.42**fan**

a temporary overhead protective screen projecting from a structure and erected for the purpose of protecting persons from falling material

1.2.43**flight auger**

an open screw-type rotary boring tool with a helically arranged cutting edge

NOTE Spoil is removed by spinning off once clear of the bore.

1.2.44**flying shore**

a strut or series of struts in the same vertical plane framed together to provide a horizontal restraint between two walls to prevent lateral movement of one or both walls towards each other. No part of a flying shore takes a bearing on the ground

1.2.45**follower**

a removable extension that transmits the hammer blows to the pile when the pile head is to be driven down below the leaders and out of reach of the hammer

NOTE This is also called a long dolly.

1.2.46**foundation**

that part of the structure designed and constructed to be in direct contact with and transmitting loads to the ground

1.2.47**gross loading intensity**

the intensity of vertical loading on the ground at the base level of a foundation due to all loads above that level

1.2.48**ground beam**

a beam in a substructure transmitting load(s) to a pile, pad or other foundation

1.2.49**double-acting hammer**

a hammer operated by steam, compressed air or internal combustion, the energy of its blows being derived mainly from the source of motive power and not from gravity

1.2.50**single-acting hammer**

a hammer raised by steam, compressed air, or internal combustion and allowed to fall under gravity

1.2.51**headtree**

a horizontal member placed immediately on the heads of dead shores

1.2.52**heave**

the upward displacement of ground as a result of operations within it

1.2.53**helmet**

a temporary steel cap placed on top of a precast concrete pile to minimize damage to the pile head during driving

1.2.54**h-pile**

a steel wide-flange column or other section often rolled with a uniform thickness in web and flange

1.2.55**jacked pile**

A pile, usually in short sections, which is forced into place by jacking it against a reaction such as the weight of the structure

NOTE A jacked pile is commonly used in underpinning work.

1.2.56**jetting**

the use of a sufficient quantity of water under pressure at the point of a driven pile as an aid to pile penetration

NOTE When difficult driving is encountered the jet pipe may be formed in the pile shaft or used separately alongside the pile.

1.2.57**kelly bar**

the sliding shaft on a boring rig which transmits the driving force or torque to the boring tool from a driven rotary table

1.2.58**kentledge**

material used to add temporary loading to a structure, e.g. to the top of caissons to assist in sinking or as dead weight in a loading test

1.2.59**lacing**

a system of ties connecting one setting of walings or struts to another

1.2.60**leaders**

steel channels or tubes used for locating and guiding the hammer and pile during driving, either forming part of a pile frame, or suspended from the jib of a mobile crane or supported on a mobile base or standing on the ground secured by guy ropes

1.2.61**leg**

a support in an underpinning forming part of the permanent work

1.2.62**monolith**

an open caisson of heavy mass concrete or masonry construction, containing one or more wells for excavation

1.2.63**mudding-in**

the technique of stirring bentonite powder and water using an auger into granular soil to facilitate the installation of temporary casing

1.2.64**needle**

a member inserted into or through a wall to give it temporary or permanent support

NOTE A needle used in conjunction with a flying or raking shore consists of a short member passing through the vertical wallpiece and into the wall to form an abutment to the shore.

1.2.65**needle piles**

small diameter, cast-in-place, reinforced concrete piles formed by drilling through existing foundations and the surrounding soil, or slender, tubular steel tube or bars installed by driving, both used in underpinning operations

1.2.66**net loading intensity**

the increase or decrease in intensity of vertical loading at the base level of a foundation due to excavation and/or the weight of the new structure including earthworks (if any). Usually it is the difference between the gross loading intensity before building operations are commenced and the gross loading intensity after the structure is complete and fully loaded

1.2.67**packing**

a pad of resilient material contained between the helmet and the top of a reinforced concrete pile to minimize damage to the pile head during driving

1.2.68**pad foundation**

an isolated foundation to spread a concentrated load

1.2.69**penetration**

the total depth below external ground level reached by a caisson or the sheet piles of a cofferdam

1.2.70**permanent set**

the net distance by which a driven pile penetrates into the ground at each blow of the hammer

1.2.71**pile cap**

a concrete block cast on the head of a pile or a group of piles to transmit the load from the structure to the pile or group of piles

1.2.72**pile frame**

a movable steel or timber structure for driving piles in the correct position and alignment by a hammer operating in the leaders of the frame

1.2.73**piled raft**

a foundation formed of piles and a raft acting together

NOTE It is generally used where swelling forces applied by the soil to a raft at the base of an excavation need to be resisted or where the settlement of an unpiled raft is expected to be excessive.

1.2.74**pipe pile**

a pile consisting of a length of steel pipe, driven either open-ended or with a shoe

1.2.75**poling board**

a flat member supported by walings in contact with the ground and supporting the face or sides of an excavation, and usually 1 m to 1.5 m long

1.2.76**preboring**

removal, by boring, of soil that may cause heave during pile driving. The pile is placed in the prebored hole and driven as usual

1.2.77**precast pile**

a reinforced or prestressed concrete pile cast before driving

1.2.78**preliminary pile**

a pile installed before the commencement of the main piling works for the purpose of establishing the suitability of the chosen type of pile and for confirming the design, dimensions and bearing capacity (see also test pile)

1.2.79**presumed bearing value**

the net loading intensity considered appropriate to the particular type of ground for preliminary design purposes

NOTE Values for various types of ground are usually given in the form of a table. The particular value is based either on experience or on calculation from laboratory strength tests or field loading tests using a factor of safety against bearing capacity failure.

1.2.80**proof load**

a load applied to a selected working pile to confirm that it is suitable for the load at the settlement specified

NOTE A proof load should not normally exceed 150 % of the working load on the pile.

1.2.81**puncheons**

vertical struts transmitting the weight of the bracing to the excavated ground surface inside a cofferdam or to a cleat, or vertical struts, transmitting the weight of a bracing frame to the frame below

1.2.82**raft foundation**

a foundation continuous in two directions, usually covering an area equal to or greater than the base area of the structure

1.2.83**raking pile**

a pile installed at an inclination to the vertical

NOTE This is sometimes called a batter pile.

1.2.84**raking shore**

an inclined strut or series of struts in the same vertical plane placed against a wall to restrain it from lateral movement. Its upper end abuts against a needle and its lower end bears upon a soleplate, usually in contact with the ground, the whole of the series of struts being braced or laced together

1.2.85**ram**

the rising and falling part of the hammer which delivers the blow

1.2.86**rock socket**

the portion of the pile boring which penetrates into a hard formation beneath less competent overburden

1.2.87**runner**

a vertical member supported by walings used to support the sides or face of an excavation and progressively driven or lowered as the excavation proceeds, its lower end being kept below the bottom of the excavation

1.2.88**screw pile**

a pile consisting essentially of a shaft provided with a helix or helices at its base and screwed into the ground

1.2.89**secant piling**

a system of retaining wall construction normally providing better watertightness than contiguous bored piling in which alternate piles are bored and concreted at closer spacings than contiguous piling. The intermediate piles are then bored between the piles previously installed and a groove chiselled down the shafts of the earlier piles so that, when the concrete is poured, a fully interlocked wall is obtained

1.2.90**segmental piles**

precast concrete shaft units which can be formed into bearing piles of any desired lengths by means of special couplings

1.2.91**sheet pile**

one of a row of piles driven or formed in the ground adjacent to one another in a continuous wall, each generally provided with a connecting joint or interlock usually designed to resist mainly lateral forces and to reduce seepage; it may be vertical or at an inclination

1.2.92**shell (1)**

a percussion boring tool with a clack valve at its lower end used in water-bearing granular soils

1.2.93**shell (2)**

a preformed tubular element in steel or concrete that is driven into the ground and subsequently filled with concrete

1.2.94**sill or soleplate**

a member, usually in contact with the ground, placed under the foot of a shore for the purpose of distributing the load

1.2.95**skin friction**

the frictional resistance of the surrounding soil on the surface of cofferdam or caisson walls, and pile shafts

1.2.96**slip layer**

a layer (normally a bituminous coating) applied to the shaft that will shear under the downward force of the settling soil thereby reducing additional load being shed onto the pile

1.2.97**soldier**

a vertical member supporting horizontal poling boards or walings

1.2.98**strip foundation**

a foundation providing a continuous longitudinal ground bearing

1.2.99**strut**

a structural member in compression, e.g. supporting the walings of a cofferdam

1.2.100**substructure**

that part of any structure (including building, road, runway or earthwork) which is below natural or artificial ground level. In a bridge this includes piers and abutments (and wing walls), whether below ground level or not, which support the superstructure

1.2.101**tension pile**

a pile that is designed to resist a tensile force

1.2.102**test pile**

a pile to which a load is applied to determine the load/settlement characteristics of the pile and the surrounding ground (see also preliminary pile)

1.2.103**tie**

a structural member in tension

1.2.104**tremie pipe**

a pipe formed of a number of lengths with quick-release couplings by which concrete can be placed in a controlled manner through slurry or water at the bottom of a trench or borehole

1.2.105**ultimate bearing capacity**

the value of the gross loading intensity for a particular foundation at which the resistance of the soil to displacement of the foundation is fully mobilized

1.2.106**waling**

a horizontal beam resisting the load from a cofferdam wall against external pressure

1.2.107**wall piece**

a vertical member (usually timber) placed in direct contact with a wall to distribute the thrust from a shore or shores

1.2.108**working load**

the load which the pile is designed to carry (see also "allowable load")

1.2.109**working pile**

one of the piles forming the foundation of the structure

1.2.110**colloidal solution**

a suspension in which the particles do not settle because they respond to molecular movement in the fluid

1.2.111**deflocculants**

agents to promote the dispersion of flocs, or solid particles which are coalesced into larger groups

1.2.112**elastomers**

a material with rubber-like properties, ease of deformation and rapid and complete recovery

1.2.113**emulsion**

a dispersion of discrete particles or droplets of one material in a continuous phase or another material in which it is not soluble

1.2.114**emulsion breaker**

an agent which promotes coalescence of small droplets and separation of the liquids in an emulsion

1.2.115**epoxide (epoxy)**

a thermosetting plastics in which a chain is formed by reactions of a polyhydric alcohol and epichlorhydrin

1.2.116**filler**

a finely divided solid which, while taking no part in the chemical reaction in a system, modifies its flow properties and subsequent mechanical behaviour

1.2.117**phenoplastic**

a thermosetting plastics based on phenols (e.g. resorcinol)

1.2.118**polysaccharides**

carbohydrates which are akin to starches and sugars and are found naturally as alginates

1.2.119**polyester**

a thermosetting plastics with a long chain formed from the reaction of an organic acid and an alcohol

1.2.120**reactants**

active constituents in a system in which chemical reactions are taking place

1.2.121**suspensions**

finely divided solid matter dispersed in a liquid

1.2.122**water soluble precondensates**

water soluble constituents of a condensation polymer in which the condensation process is largely arrested and which, when a suitable catalyst is added, is completed

1.2.123**blowing down**

the means of causing a pneumatic caisson to sink by reducing the air pressure in the working chamber thereby increasing the effective weight, which acts to overcome skin friction resistance on the caisson walls and resistance in end bearing at the cutting edge

Section 2. Design of foundations

2.1 General

NOTE This section deals with the principles of, and makes recommendations applying generally to, the design of foundations of all types.

2.1.1 Characteristics

The characteristics of soils and rocks used in this code are given in Table 1 and 2.2.2.3.

NOTE Further information can be obtained from BS 5930.

2.1.2 Ground movement

2.1.2.1 General. Foundation design should ensure that foundation movements are within limits that can be tolerated by the proposed structure without impairing its functions. Since the capacity of structures to accommodate movements varies, the design of the structure should be interrelated to its foundation (Burland et al. 1977).

In addition to knowledge of the engineering properties of the underlying rocks or soils, as well as groundwater conditions, foundation design requires an understanding of the local geology and history of the site (see 2.2.1.2).

Foundation movements occur from the application or removal of load or ground movement independent of load (see 2.1.2.4) and, as well as affecting the structure, may affect external connections such as pipes or drains and the structure's relationships to adjacent structures (Burland and Wroth 1975).

For simple structures, especially those not sensitive to movement, simple design methods are usually adequate.

2.1.2.2 The foundation and the ground as an interrelated system. Errors are likely to occur if undue attention is given to refinement in calculation or to a particular aspect of foundation design at the expense of maintaining an overall understanding of the behaviour of the foundation and supporting ground. This understanding includes the mechanism and behaviour of the ground and the way in which the different parts of the ground, the foundation structure and water (whether free to flow or observed as pore water pressure) form an interdependent system (Burland 1975).

In addition to considering the ground and foundation together as a system, it is helpful to have a rough picture of how the individual elements of the ground respond to changes in direct or shear stresses, the volume, the water content and the strength. The way in which elements of ground act as a system is best perceived in the light of the surrounding stratigraphy, the geological history, appropriate tests on samples and the principle of effective stress. The response of each system to changes of load or movement or water content change is likely to be time-dependent to a greater or lesser degree.

It should be remembered that the ground, before excavation begins, may already be subject to movement and pressure changes, though these changes may be small in amount and taking place slowly. Such changes may be due to nearby construction works, ground consolidating under its own weight, changes in groundwater level or the consolidation of previously placed fill (see 2.1.2.4). The construction method is an important factor influencing movements and loads during and following the works.

The engineer, when considering the likely effects on the structure, should endeavour to assess, if only roughly, the kind of movement or pressure change or water pressure change which could be expected in the ground, and the orders of magnitude of such changes. Detailed design and calculations should only proceed in the light of this general understanding of how the ground is likely to respond to the excavation and foundation works.

At this stage consideration should be given to whether the foundation or superstructure is in any respect subject to structural sensitivity. A system or structure is structurally sensitive when it responds disproportionately to forces or environmental effects (Armitage and Bratchell 1976).

Where detailed design and calculations depend on, or result in predictions of the amount of earth movement, the engineer may usefully refer to the records of earth movements which have taken place and also to publications such as Lambe (1973). Satisfactory correlations have been obtained between theoretical and observed movements using computer-based finite element methods, and a variable elastic modulus. Before using these calculations the designer should consider whether the ground data are sufficiently accurate and representative to justify such accuracy of calculation, and whether the accuracy of the results is useful. These considerations should take account of the proposed method of construction.

Careful consideration should be given to monitoring the movements of the structure and the ground by establishing a system of measurement during and after construction to give assurance that design assumptions are being adequately realized.

2.1.2.3 Movements due to application of load

2.1.2.3.1 General. Application of an increased load through the foundations of a structure results in deformation of the ground and settlement. Settlements are divided into immediate settlement, which takes place as load is applied, and long-term settlement, which may continue for some time after application of load. In practice it is difficult to separate the two because some of the long-term settlements may occur quite rapidly in the early stages of foundation loading. One type of long-term settlement on soils is known as consolidation settlement and is due to expulsion of air and water from voids in the soil, resulting in volume reduction. In clay soils, this process may take years to reach an equilibrium condition, after which no further settlement occurs. Long-term creep or secondary settlement may also occur especially in peaty or organic soils.

If the factor of safety is inadequate, overstressing of the ground results in plastic deformation of the ground below the foundation, leading to movement of the foundation greater than the settlements previously discussed. In extreme cases shear failure will occur (see **2.1.2.3.2**).

2.1.2.3.2 Shear deformation and failure

2.1.2.3.2.1 Bearing capacity. In order to guard against the possibility of shear failure or substantial shear deformation, the foundation pressures used in design should have an adequate factor of safety when compared with the ultimate bearing capacity for the foundation.

With uniform ground conditions, the ultimate bearing capacity will increase with the depth of embedment of the foundation in the ground. This increase is associated not only with the effects of confinement of the ground and the increased overburden pressure at foundation level, but also with shear forces that can be mobilized between the sides of the foundation and the ground. Where foundations are deep, as in the case of deep basements, piles, cylinders or caissons, the contribution to the ultimate bearing capacity by the shear forces on the sides may be large.

Methods for estimating the allowable net bearing pressure for foundations are outlined in **2.2.2**. In Table 1 presumed bearing values for foundations on various types of ground are given.

It should, however, be realized that existence of an adequate factor of safety against shear failure will not necessarily ensure that foundation settlements will be sufficiently small. In particular, the allowable bearing pressure for a large foundation on granular soil may have to be much smaller than the ultimate bearing capacity divided by a conventional factor of safety of 2 or 3.

Loading tests on an individual test plate, footing, pile, pier or caisson may give the ultimate bearing capacity of the soil stressed by the single structural element at the particular locality tested, but the results may not be representative of the bearing characteristics of the complete foundation nor of the site as a whole. Increased settlements or even deep-seated failures due to the combined loads of a number of foundations may occur; this should be checked. It should be noted that the settlements recorded in connection with loading tests will rarely give an indication of the settlement of the full-scale structure.

When a foundation is required to carry eccentric or inclined loads, the ultimate bearing capacity should be estimated as indicated in **2.3.2.4.6** or **2.3.2.4.7**.

2.1.2.3.2.2 Soft layers below foundation. When soft layers lie below the proposed foundation level, the bearing pressures will need to be adjusted so that the stresses in the soft layer do not lead to excessive plastic deformation (see **2.2.2.3.6**). Attention will also need to be paid to possible settlements in the soft layer.

2.1.2.3.2.3 Effect of adjacent cuttings, excavations or sloping ground. Cuttings, excavations or sloping ground near foundations may increase the possibility of shear failure in the ground supporting the foundations. In some areas the natural ground in the slopes and cuttings may itself be close to limiting equilibrium, and if these sites cannot be avoided, steps may have to be taken to increase the general stability of the site before new structures are erected, in all cases the possible effect of slopes and excavations on foundation stability should be carefully investigated (see **2.2.2.3.6.2**).

2.1.2.3.2.4 Inclined strata. Inclined or jointed strata in which the bedding or joint planes dip towards an excavation or natural depression may present stability problems, as failures can occur on the surfaces of weakness along the bedding or joints. It may be necessary to improve the resistance of the strata (see section 6) or to carry foundations deep enough to prevent sliding on these surfaces of weakness.

2.1.2.3.3 Settlement. The magnitude of the settlement that will occur when foundation loads are applied to the ground depends on the rigidity of the structure, the type and duration of loading and the deformation characteristics of the ground (see 2.1.2.4). In silts and clays consolidation settlement may continue for a long period after the structure is completed, because the rate at which the water can drain from the voids under the influence of the applied stresses is slow; allowance will need to be made for this slow consolidation settlement. In beds of organic soils settlement may be prolonged almost indefinitely due to a phenomenon known as creep or secondary consolidation, and will need special consideration. In sands and gravels and most rocks, however, the settlement is likely to be substantially complete by the end of the construction period.

The magnitude and rates of settlement to be expected may be gauged by:

- a) the study of the behaviour of similar structures having essentially the same foundation conditions, on the basis of structural condition and any available settlement records;
- b) an analysis of the settlement based on measured soil properties and the stresses applied to the ground by the structure.

It should be appreciated that new construction may lead to additional settlement of adjacent structures. The design of the foundations should take this possibility into account (see 2.3.2.7).

2.1.2.4 Movements independent of applied foundation load. Movements may result from causes not connected with the loads applied by the foundations (Cooling and Ward 1948). The causes of ground and foundation movement are as follows:

- a) seasonal changes or the effects of vegetation leading to shrinking and swelling of clay soils;
- b) frost heave;
- c) the application of artificial heat or cold to the supporting ground;
- d) changes in groundwater level resulting from designed or unforeseeable drainage, or modification of the groundwater regime by construction, or by natural causes;
- e) loss of ground due to erosion (including internal erosion) or solution by percolating water, and loss of fines by pumping operations;
- f) changes in the state of stress in the ground due to adjacent excavations, dredging, scour or erosion by streams or floods (loss of support) or due to the erection of adjoining structures;
- g) continuing settlement of natural deposits or fill;

- h) soil creep or landslides on slopes;
- i) movement of ground resulting from sink or swallow holes or underground workings (including mining and tunnelling);
- j) vibrations, including seismic disturbances;
- k) deterioration of made ground or fill;
- l) deterioration of the substructure itself;
- m) alteration of the properties of the ground due to natural or artificial causes;
- n) coast erosion.

In the design of foundations the possibility of movements unconnected with the application of load should be considered and, when necessary, steps should be taken to minimize any damage that may result from these causes.

2.1.2.5 Influence of type of foundation and structure. In assessing ground movements and the limiting values which can be accepted, consideration needs to be given to the capacity of the proposed structure to accommodate foundation movements and to the ability of the type of foundation chosen to meet the particular site conditions encountered.

Although foundations exist in considerable variety, they may be broadly grouped into three main types: shallow foundations; subaqueous and deep foundations; pile foundations.

General design considerations relating to the three main types of foundation are given in 2.3.3.

2.1.2.6 Influence of construction procedure, excavation and foundation installation. The construction procedure required to install foundations may have an adverse effect on the supporting properties of the subsoil or on soil movements subsequent to construction. Subsequent construction operations, even on an adjacent site, may affect existing operations. Details of construction procedures involved in foundation engineering are given in sections 4 to 9.

General design considerations arising from construction procedures and foundation installation are given in 2.4.

2.1.3 Groundwater

2.1.3.1 The presence of groundwater frequently causes difficulty during construction or in completed works. Section 6 gives recommendations on the removal of water from works during construction. Information on the exclusion of water from completed structures is given in 2.3.4 and CP 102.

2.1.3.2 In the design of foundations, the effect of groundwater should always be carefully considered. During construction, water levels may be seasonally at a low level or they may be artificially lowered. Construction may alter the groundwater regime over a wide area by, for example, the clearance of vegetation, the modification of existing drainage facilities or basement construction. The construction may interfere with the natural groundwater flow causing a rise or fall in the water table. Substructures should be designed to be stable with any groundwater level that is likely to occur. Where it is necessary for the structure to be watertight, it should be made watertight to a sufficient height to guard against capillary action and to allow for a subsequent rise in the water table. The effects of groundwater are dealt with in **2.3.2.7**, **2.4.3**, and section 4.

Some naturally occurring materials (e.g. rock salt and some limestones) as well as some artificial chemical contaminants are soluble. Expert advice should be sought where there is any likelihood of such materials being present.

2.1.4 Flooding

The effect of floods, both during construction and on the completed structure, should be considered.

2.2 Ground considerations

2.2.1 Ground exploration and tests

2.2.1.1 General. Before foundations are designed and a method of construction determined, it is essential to carry out a site exploration to ascertain the character and variability of the strata underlying the site of the proposed structure. In particular, it is necessary to assess those properties which may affect the performance of the structure and the choice of method of construction.

Site investigations should be carried out generally in accordance with BS 5930. In this code (BS 8004), brief reference is made only to points of particular importance in foundation design.

2.2.1.2 Collection of available information. As a preliminary, it is advantageous to collect information relating to the site prior to commencing its exploration. The geology should be investigated. Details that are available from records and maps relating to geological information should be studied and advantage should be taken of existing local knowledge. The study of aerial photographs or geomorphology may prove valuable. Enquiries about earlier uses of the site may reveal whether there have been underground workings or filled-in quarries, brickworks, etc. A preliminary visual examination of the site and surrounding country may also give an indication of the groundwater level and the drainage and flooding conditions to which the site may be subjected. Local observation may also indicate whether the site is susceptible to landslips or to local subsidence arising from mining operations or swallow holes. The inspection of buildings in the neighbourhood, particularly for signs of settlement and the incidence of cracking, may yield valuable information when structures of a similar nature are proposed. Local authority archives and other records may be useful.

NOTE For further information see Dumbleton and West (1976) and Dumbleton (1979).

2.2.1.3 Type of exploration. The exploration of a site for an important structure requires the exploration and sampling of all strata likely to be significantly affected by the structural load. The extent of this exploration will vary with the site and structure. In built-up areas where experience of similar structures and sufficient quantitative information is available for a satisfactory design, exploration may be limited to checking that the ground conditions are those expected in the neighbourhood. If existing information is not sufficient or is inconclusive, the site should be explored in detail to obtain a knowledge of the type, consistency, thickness, sequence, discontinuities and dip of the strata and knowledge of the groundwater conditions.

For examination to fairly shallow depths, where conditions are suitable, trial pits have the advantage that the sides of the pit can be inspected at all levels. Where “made” ground is encountered, trial pits enable it to be identified and may enable its probable bearing capacity to be assessed. For deeper explorations where the ground is unsuitable for trial pits, boreholes or drill holes are required.

Samples or rock cores should be procured for the purposes of identification from each stratum. Where appropriate, undisturbed soil samples should be taken so that tests can be made on them to assess bearing capacity, or if necessary to carry out a settlement analysis. Continuous sampling may be required to establish minor variations in strata which could have an important effect on, for example, drainage (see 6.2.2). In certain conditions, in situ tests will more readily provide the necessary information or may be the only available means of investigation. An investigation may require both sampling and in situ testing; the choice of method should be carefully related to the type of material encountered at various levels in the borehole or test pit. There are other methods of investigation, such as probing and geophysical exploration, which may provide useful supplementary information.

2.2.1.4 Number and disposition of trial pits, borings and drillings. The disposition and spacing of the trial pits, borings or drillings should be such as to reveal any major changes in thickness, depth or properties of the strata. The number of trial pits, borings or drillings required will vary with the size and type of structure, the general nature of the site and the completeness or otherwise of local geological and geotechnical records. It is sometimes difficult to give definite indications in advance, and the programme should therefore be flexible and allow for modifications as information is obtained. If there is disagreement between the records and the boreholes or drill holes, a very detailed exploration may be necessary to give a clear picture of the pattern of subsurface strata. Supplementary tests such as probings may be particularly useful for this purpose.

Exploratory boreholes, drill holes and trial pits should be positioned and reinstated as far as possible, in such a way as to minimize interference with construction work; their positions and depths should all be fully recorded and taken account of in planning construction work.

2.2.1.5 Depth of exploration. Exploration should be carried to a depth which includes all strata likely to be significantly affected by the structural load and the construction operations. The depth of exploration will depend on the type of structure, the size, shape and disposition of the loaded areas and the nature of the strata. While the nature of the stratum on which the foundation directly rests is normally of most importance in determining the allowable bearing pressure, special consideration should be given to deeper strata which have smaller allowable bearing pressures (see 2.2.2.3.6).

Precautions are necessary to ensure that boulders are not mistaken for bedrock, particularly on sites where borings are made through glacial material or deeply weathered rock.

In deciding the depth of exploration, attention should also be given to the effect of the ground conditions on construction methods; for example, if work is to be done in a cofferdam, the possibility of reaching an impervious layer with sheet piles to provide a cut-off should be explored.

If the upper strata are soft, a piled foundation may be considered, in which case the ground will be stressed to an appreciable depth below the pile toes. It is therefore necessary to know the character of the underlying strata, and it may be necessary to carry out an analysis of settlement of the structure resulting from consolidation of the deeper layers beneath the pile toes.

2.2.1.6 Groundwater (see also section 6). Owing to the low permeability of many soils, the water levels in boreholes or observation wells may take a considerable time to reach equilibrium with the groundwater. Spot readings of the water level in boreholes may therefore give an erroneous impression of the true groundwater level. Readings should be made in piezometers or simple standpipes over a sufficient period of time in order to obtain a proper assessment of fluctuations in the groundwater level. Water tests in drill holes (such as packer tests, see BS 5930) may give valuable information.

Seasonal variation in the level of the water table may also be important. Where deep excavation is required, levels of water-bearing strata should be determined with particular care, and the water level and pressure in each stratum should be observed so that necessary precautions may be taken during excavation.

On sites liable to be waterlogged in wet weather, it is desirable to determine the contour of the water table relative to the ground surface in order to indicate the directions of the natural drainage. This will facilitate the design of intercepting drains to prevent the influx of groundwater into the site from higher ground.

In certain localities the groundwater, soil or rock may contain constituents in amounts sufficient to cause damage to concrete, buried metals or plastics (see section 10). In such cases it will be desirable to obtain a chemical analysis of samples of the groundwater, soil or rock to assess the necessity of special precautions. It is essential that the groundwater samples should not be diluted by water added to assist drilling of the boreholes; special precautions will be needed where rock drilling uses water or mud flush.

2.2.1.7 Test loading and in situ tests. Test loadings are usually made on bearing plates or piles. The fact that a small isolated area of ground or a single test pile will carry a heavy load without excessive settlement is not necessarily proof that a much larger area or a group of piles will carry load proportionally, particularly where a stronger layer of soil overlies a weaker, one (see BS 5930).

2.2.1.8 Responsibility for exploration and interpretation of results. Owing to the close dependence of the design of the structure on the soil conditions, it is essential that the exploration be adequately supervised and that the interpretation of the results should be closely related to the design and construction operations as a whole.

2.2.2 Allowable bearing pressure on various types of ground

2.2.2.1 General. Computation of moments, thrusts and shears, and the design of sections to resist them, should in general follow principles set out for superstructures. To obtain maximum strength and economy, the foundations, substructure and superstructure should be studied as a whole.

The design of the permanent and temporary works should be considered together; modification of the design of either may be necessary until the best solution is obtained.

The foundations and substructure should have an adequate factor of safety under all conditions that may occur during any stage of the construction of the entire work. The construction procedure should be arranged to avoid endangering the strength or stability of the partially completed structure.

Possible extensions or alterations should be borne in mind, particularly when designing foundations for industrial plant.

Variations of the strata in a horizontal direction (see **2.2.2.3.6**) are likely to be particularly important for buildings. Where portions of the foundation are underlain by soft materials or where the layers of such soft materials vary in thickness over the area of the site, the assessment of the allowable bearing pressure may require a settlement analysis.

2.2.2.2 Allowable bearing pressure and presumed allowable bearing value

2.2.2.2.1 General. Universally applicable values of allowable bearing pressure cannot be given. Factors affecting bearing capacity have been discussed in **2.1.2** and they show that, for any important structure, the allowable bearing pressure cannot be assessed without taking into account the effect of settlement. However, foundation design is at present possible only by trial and error methods, so that it is desirable to have some basis for preliminary design assumptions. Therefore, the concept of presumed bearing values is used in this code. It is emphasized that the presumed bearing value should be used by the designer only for preliminary foundation design purposes and, in all cases, he should then review and, if necessary, amend his first design. This will frequently entail an estimate of settlements.

2.2.2.2.2 Presumed allowable bearing value. For the purpose of assigning presumed bearing values, rocks and soils have been divided into five categories:

- category 1: rocks
- category 2: non-cohesive soils
- category 3: cohesive soils
- category 4: peat and organic soils
- category 5: made ground or fill

Presumed bearing values for each of these categories are given in Table 1.

2.2.2.3 Characteristics of various types of ground and allowable bearing pressures

2.2.2.3.1 Allowable bearing pressure on rocks

2.2.2.3.1.1 General. Rocks generally have a high allowable bearing pressure which may be reduced by the presence of weak layers, discontinuities or where the rocks are weathered, decomposed, heavily shattered or steeply dipping. Each case will need to be considered individually. The effect of joints, fissures, other discontinuities and of any infillings in them is very important and should be taken into account. Dangerous conditions may develop with stratified rocks, joints or old shear surfaces if they dip towards cuttings or deep basements.

The allowable bearing pressure depends on the compressibility and strength of the rock mass and the permissible settlement of the structure. A method of determining allowable bearing pressures on weak rocks from an assessment of these properties is given in **2.2.2.3.1.12**.

The compressibility of the rock mass is related to the uniaxial compressive strength of the intact rock, the lithology, the frequency, nature and orientation of the discontinuities in the rock mass. The compressibility of intact rock is also dependent upon the degree of decomposition or disintegration as a result of weathering or alteration; these effects are reflected in the uniaxial compressive strength. Massive rocks in the fresh or slightly weathered state generally have a high allowable bearing pressure that reduces with increase in the degree of weathering and jointing.

Attention is drawn to the possible existence, beneath an apparently sound formation, of layers of softer materials including clay, weaker rocks, or old land surfaces on which weathering developed and which were subsequently covered by younger formations.

2.2.2.3.1.2 Field measurements. The choice of method and location of field tests is determined by the size and nature of the proposed structure, the type and condition of the rocks underlying the site, including the degree, extent and variability of weathering and the spacing, opening and nature of the discontinuities, and by the groundwater conditions.

The condition of the rock is assessed by inspection in trial pits and by examination of drill cores taken in a suitable manner. It is usually necessary to concentrate field and laboratory testing on the weaker or more fractured zones or layers since these may control the design. These zones or layers are those in which core recovery may be poor or even zero. In those circumstances where it is essential to recover undamaged core from highly fractured rock, integral sampling (in which the previously grouted rock is overcored) is sometimes useful. Choice of core size, type of core barrel and bit and the flushing medium should be related to the rock type and condition; larger diameter core barrels and/or special core barrels (e.g. triple tube) may be required in poor rock conditions. Cores should be preserved at their natural water content for detailed logging and testing. Some very weak weathered rocks may be better explored by soft ground boring and standard penetration testing. Reference should be made to BS 5930 for guidance on the appropriate methods.

Rock strength, the major factor in the assessment of allowable bearing value, is generally estimated in the field by hammering and scratching specimens of the rock. When calculating allowable bearing pressure with rock strength determined by these means, a conservative approach is desirable. A numerical estimate of compressive strength can be obtained in the field and laboratory by the point load or unconfined compression strength test (Franklin and Broch 1972).

Plate loading tests in trial pits and adits are a usual method of assessing the modulus and stress/strain characteristics of the rock mass, but considerations of cost may limit the depths at which these tests may be done. Seismic refraction in association with other methods may be used to assess the character of the rock mass on a very large scale; seismic shooting between boreholes at selected depths enables the horizontal dynamic modulus of the rock to be determined on a smaller scale. The vertical dynamic modulus is obtained by uphole or downhole seismic shooting in boreholes.

Table 1 — Presumed allowable bearing values under static loading (see 1.2.3 and 1.2.4)

NOTE These values are for preliminary design purposes only, and may need alteration upwards or downwards. No addition has been made for the depth of embedment of the foundation (see 2.1.2.3.2 and 2.1.2.3.3).				
Category	Types of rocks and soils	Presumed allowable bearing value		Remarks
		kN/m ² ^a	kgf/cm ² ^a tonf/ft ²	
Rocks	Strong igneous and gneissic rocks in sound condition	10 000	100	These values are based on the assumption that the foundations are taken down to unweathered rock. For weak, weathered and broken rock, see 2.2.2.3.1.12
	Strong limestones and strong sandstones	4 000	40	
	Schists and slates	3 000	30	
	Strong shales, strong mudstones and strong siltstones	2 000	20	
Non-cohesive soils	Dense gravel, or dense sand and gravel	> 600	> 6	Width of foundation not less than 1 m. Groundwater level assumed to be a depth not less than below the base of the foundation. For effect of relative density and groundwater level, see 2.2.2.3.2
	Medium dense gravel, or medium dense sand and gravel	< 200 to 600	< 2 to 6	
	Loose gravel, or loose sand and gravel	< 200	< 2	
	Compact sand	> 300	> 3	
	Medium dense sand	100 to 300	1 to 3	
	Loose sand	< 100	< 1	
		Value depending on degree of looseness		
Cohesive soils	Very stiff boulder clays and hard clays	300 to 600	3 to 6	Group 3 is susceptible to long-term consolidation settlement (see 2.1.2.3.3). For consistencies of clays, see Table 5
	Stiff clays	150 to 300	1.5 to 3	
	Firm clays	75 to 150	0.75 to 1.5	
	Soft clays and silts	<75	<0.75	
	Very soft clays and silts	Not applicable		
Peat and organic soils		Not applicable		See 2.2.2.3.4
Made ground or fill		Not applicable		See 2.2.2.3.5
^a 107.25 kN/m ² = 1.094 kgf/cm ² = 1 tonf/ft ² .				

^a 107.25 kN/m² = 1.094 kgf/cm² = 1 tonf/ft².

The horizontal modulus on a very small scale can be determined at selected depths in boreholes by means of the pressuremeter, dilatometer or borehole jack. The tightness of the rock mass is customarily investigated in boreholes by carrying out water injection tests between packers. The permeability or Lugeon value in conjunction with the intensity of the natural discontinuities seen in the cores from the test zone gives a useful qualitative indication of the average aperture of discontinuities but clogging with debris may occur.

In some problems the foundation performance will be dominated by a possible mode of failure involving movement along some joint surface, fissure or weak layer within a generally strong rock system. In such cases it is necessary to investigate the strength and stress/strain characteristics of the joint, fissure or layer. In situ shear tests are sometimes attempted, but it is frequently more satisfactory to carry out large shear tests in the laboratory on blocks obtained from trial pits or on large diameter drill cores or using a portable direct shear apparatus (Hoek and Bray 1977).

2.2.2.3.1.3 Laboratory measurements. Strength and deformation characteristics are obtained from uniaxial tests on representative rock cores. The quantity of testing should be related to the variability of the rock or rock types present and the size of the foundation. Rock cores should be protected against loss of moisture immediately on extraction from the ground and should be tested at their natural water content. Dried out or partially dried out rock should not be resaturated. Strain measurements should be made by electrical resistance strain gauges bonded onto the rock cylinders or by transducers between girth clamps; measurement of strain between end caps will underestimate the modulus of the rock. For this and other reasons oedometer tests are not suitable for assessing the compressibility of rocks except in the more highly weathered state.

2.2.2.3.1.4 Strong igneous and gneissic rocks in sound condition. The allowable bearing pressure on these rocks is so high that only in very rare cases is the full capacity likely to be used. The possibility of foundation failure is very remote, provided that it is certain that the rock is not merely a dyke or other small intrusion.

2.2.2.3.1.5 Strong limestones and strong sandstones. In general, these rocks when massively bedded are stronger than good quality concrete. In assessing the allowable bearing pressure, account should be taken of the nature, spacing and direction of any joints. If there is any doubt about the hardness of the intact rock, the crushing strength of representative specimens should be determined, due regard being paid to bedding (see also **2.2.2.3.1.8**).

Limestones, including dolomites and chalk, are particularly susceptible to solution by groundwater containing dissolved carbon dioxide flowing along joints. The joints become enlarged and may range in size from fissures several millimetres wide to large crevasses and caves. In places these are connected to the surface by swallow holes (sinkholes) which may be filled with much softer material or masked by surface deposits. Solution may also produce an extremely irregular surface to the rock which may be buried under superficial deposits. Abandoned underground workings may exist in some limestone and chalk areas.

Great care is needed in limestone and chalk areas to ensure that a site selected for construction is fully investigated for these features.

2.2.2.3.1.6 Schists and slates. Schists and slates are examples of metamorphic rocks in which the properties have been modified by intense pressure and/or heat. In the unweathered state, these are hard rocks with pronounced cleavage. If the beds are shattered or steeply inclined, a reduction in the values in Table 1 should be made. Slips on cleavage planes may cause trouble if there are cuttings or basements close to the foundations. Mica-schist is particularly sensitive to movement along cleavage.

2.2.2.3.1.7 Strong shales, strong mudstones and strong siltstones. Strong shales, strong mudstones and strong siltstones have been formed from clayey or silty deposits by intense natural compaction and, in some cases, by cementation, and have a fairly high allowable bearing pressure. Uncemented shales and mudstones may soften in contact with water or on exposure, and allowance should be made for this (see also **2.2.2.3.1.9**).

2.2.2.3.1.8 Chalk. The Upper and Middle Chalk consist of nearly pure calcium carbonate. The more common chalk is highly porous, having a porosity greater than 35 %, and is white and friable. A less porous chalk which is considerably stronger can be distinguished from the more common chalk by the fact that it does not rub off on the fingers. The Lower Chalk contains varying amounts of clay, has a wide ranging porosity and is intermediate in strength (Hobbs and Healy 1979).

A convenient classification for high porosity chalk (Ward et al. 1968) is given in Table 2. Within any grade the bearing value selected should depend upon the tightness and spacing of the joints and upon the size of the foundation; the larger the foundation the lower the pressure, particularly in the poorer grades of chalk.

The presumed bearing values of grades V and VI chalk, i.e. chalk that has lost its structure through reworking (remoulding), solifluction or frost shattering, should be determined by means of tests used for cohesive soils or by field loading tests, particularly if the chalk is soft. If grade V chalk is located well above the water table and is comparatively dry, the values given in Table 2 may be used for a preliminary assessment. If the poorer grades of chalk are thin or shallow in relation to the size of the foundation, higher values than those given in Table 2 may be used but likely settlement should be estimated.

If the joints in low porosity pure chalk and in the clayey chalk of the Lower Chalk are open (see Table 2) the presumed bearing value of grades III and IV chalk may be taken from the table, if the joints are tight, Figure 1(a) may be used for the low porosity pure chalk, Figure 1(b) for the less clayey Lower Chalk and Figure 1(c) for the more clayey Lower Chalk. Strong chalk rock may be regarded as a limestone (see **2.2.2.3.1.5**).

For important structures the modulus and strength profiles of the chalk should be assessed by appropriate in situ and laboratory testing methods and both immediate and creep settlement considered in determining the allowable bearing pressure.

2.2.2.3.1.9 Keuper Marl. The strong red brown mudstones and strong argillaceous siltstones of this formation in the unweathered state have a fairly high allowable bearing pressure which may be determined from Table 1. Generally, the siltstones will have a higher bearing capacity than the mudstones.

Table 2 — Classification and presumed bearing values for high porosity chalk

Grade	Brief description	Presumed bearing value	
		kN/m ²	kgf/cm ² tonf/ft ²
VI	Extremely soft structureless chalk containing small lumps of intact chalk	See 2.2.2.3.1.8	
V	Structureless remoulded chalk containing lumps of intact chalk. Dry chalk above the water table	125 to 250	1.25 to 2.5
IV	Rubbly partly-weathered chalk with bedding and jointing. Joints 10 mm to 60 mm apart, open to 20 mm, and often infilled with soft remoulded chalk and fragments	250 to 500	2.5 to 5.0
III	Rubbly to blocky unweathered chalk. Joints 60 mm to 200 mm apart, open to 3 mm, and sometimes infilled with fragments	500 to 750	5.0 to 7.5
II	Blocky medium-hard (weak) chalk. Joints more than 200 mm apart and closed	750 to 1 000	7.5 to 10
I	As for grade II, but hard (moderately weak) and brittle	1 000 to 1 500	10 to 15
NOTE For further information see Ward et al. (1968).			

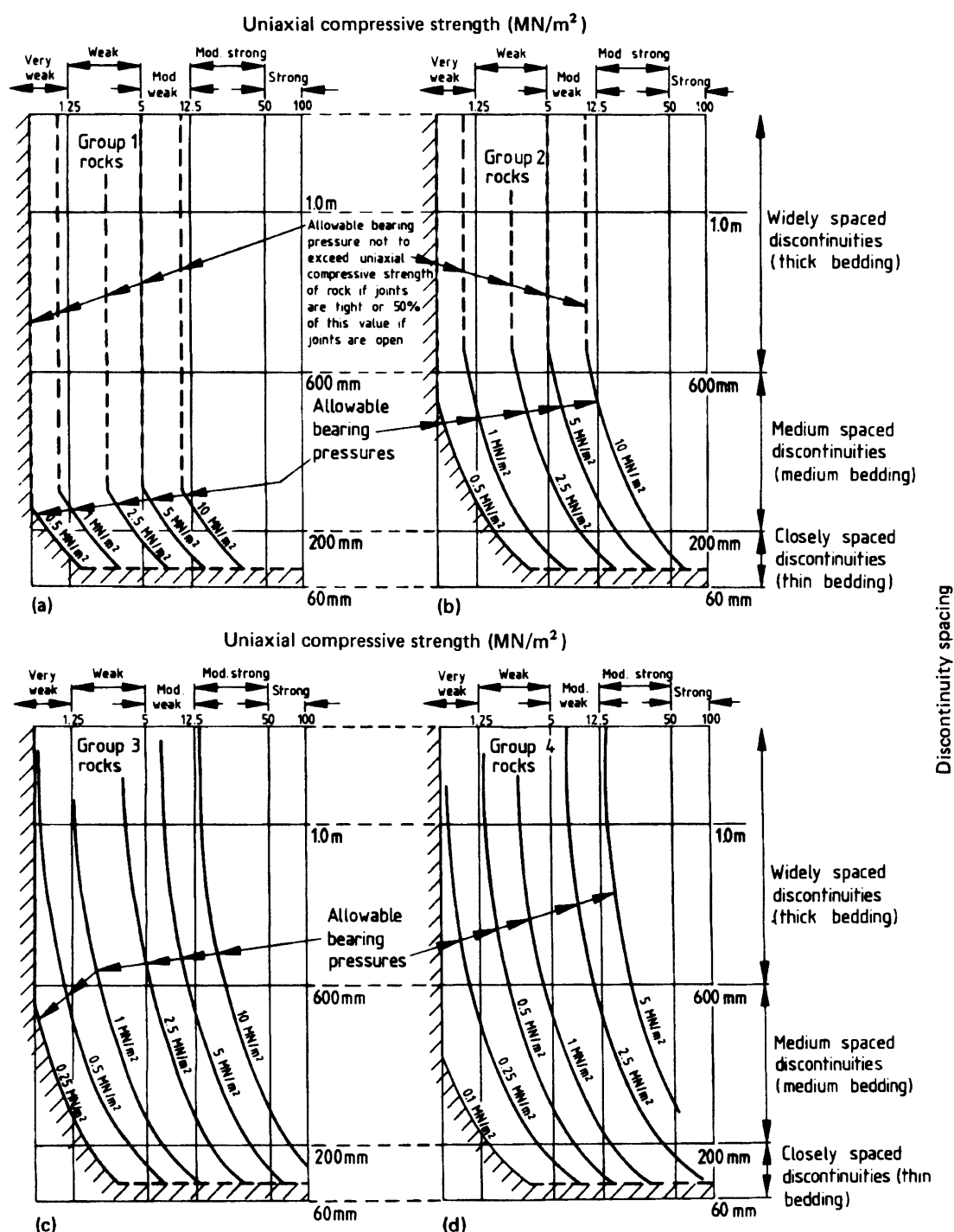
A weathering scheme for weathered Keuper Marl is given in Table 3 (Chandler 1969). A range of presumed bearing values is given in the table against each of the weathering zones with the exception of the fully weathered, zone IVb marl since in this state the marl is properly regarded as a cohesive soil for purposes of estimating the allowable bearing pressure. Zone IVa and possibly zone III marls may also be amenable to soil testing procedures and analysis (see 2.2.2.3.3). Within each zone the coarser grained and more calcareous the marl the higher the presumed bearing value.

For important structures the modulus and strength profiles of the marl should be assessed by appropriate in situ and laboratory testing methods and the long-term settlement of the foundations considered in determining the allowable bearing capacity (Meigh 1976).

2.2.2.3.1.10 Conglomerates. The constituent fragments in conglomerates vary widely. Where the fragments are of strong rock and well cemented, the rock, if dense, may be classified as having an allowable bearing pressure similar to that of a strong sandstone (2.2.2.3.1.5). Where the fragments are of shale or mudstone, or weakly cemented, the allowable bearing pressure should be determined by the method given in 2.2.2.3.1.12.

2.2.2.3.1.11 Evaporites. These deposits are typified by gypsum and salt. The most important characteristic for foundations is likely to be the solubility of the rock. In cases of doubt, expert advice should be sought.

2.2.2.3.1.12 Allowable bearing pressure on weak and broken rocks. Allowable bearing pressures on weak and broken rocks depend on the compressibility and strength of the rock mass and the permissible settlement of the structure. As stated in 2.2.2.3.1.1 the compressibility of the rock mass is related to the uniaxial compressive strength of the intact rock, the lithology, frequency, nature and orientation of the discontinuities in the rock mass. For the purposes of determining allowable bearing pressures, rocks have been placed in groups having broadly similar ratios of the Young's modulus of the intact rock to its compressive strength (modulus ratio) as given in Table 4.



NOTE 1. Allowable bearing pressures in hatched areas to be assessed after inspection and/or making tests on rock.

NOTE 2. For types of rock in each of four groups, see 2.2.2.3.1.12.

Figure 1 — Allowable bearing pressures for square pad foundations bearing on rock (for settlement not exceeding 0.5 % of foundation width)

Table 3 — Weathering scheme and presumed bearing values for Keuper Marl

Degree of weathering	Zone	Description	Notes	Presumed bearing value	
Fully weathered	IVb	Matrix only	Can be confused with solifluction or drift deposits, but contains no pebbles. Plastic slightly silty clay. May be fissured	kN/m ²	kgf/cm ² tonf/ft ²
				See 2.2.2.3.1.9	
Partially weathered	IVa	Matrix with occasional clay-stone pellets less than 3 mm in diameter but more usually coarse sand size	Little or no trace of original (zone 1) structure, though clay may be fissured. Lower permeability than underlying layers	125 to 250	1.25 to 2.5
	III	Matrix with frequent lithorelicts up to 25 mm. As weathering progresses lithorelicts become less angular	Water content of matrix greater than that of lithorelicts	250 to 500	2.5 to 5.0
	II	Angular blocks of unweathered marl with virtually no matrix	Spheroidal weathering. Matrix starting to encroach along joints: first indications of chemical weathering	500 to 750	5.0 to 7.5
Unweathered	I	Mudstone (often fissured)	Water content varies due to depositional variations	750 to 1 000 See text	7.5 to 10.0

NOTE For further information see Chandler (1969).

Table 4 — Grouping of weak and broken rocks

Group	Type of rock
1	Pure limestones and dolomites
2	Carbonate sandstones of low porosity Igneous Oolitic and marly limestones Well cemented sandstones Indurated carbonate mudstones
3	Metamorphic rocks, including slates and schists (flat cleavage/foliation) Very marly limestones Poorly cemented sandstones Cemented mudstones and shales Slates and schists (steep cleavage/foliation)
4	Uncemented mudstones and shales

Curves for selected values of allowable bearing pressure against rock strength and bedding and discontinuity spacing in the rock mass are presented in Figure 1(a) to Figure 1(d) for the preceding four rock groups using the strength and discontinuity classifications given in BS 5930. The bearing pressures shown have been calculated on the basis of rigid square foundations, with an arbitrarily chosen settlement ratio (settlement to foundation width) of 0.5 %, supported on homogeneous rock. Values of allowable bearing pressure for other settlement ratios may be obtained by direct proportion and for other shapes of rectangular foundation by reducing the bearing pressure by the customary shape factors. The derivation of the curves in Figure 1 is given in appendix A.

Chalk and Keuper Marl are excluded from this classification and are considered individually in 2.2.2.3.1.8 and 2.2.2.3.1.9.

For the more rigid group 1 and group 2 rocks, adoption of the settlement ratio of 0.5 % leads to excessively high bearing pressures in the less fractured rock, exceeding the compressive strength of the rock. In these circumstances the allowable bearing pressure should be reduced to a value not exceeding the compressive strength if the joints are tight and not exceeding one-half of the strength if the joints are open. The allowable bearing pressure curves in this region are shown in Figure 1(a) and Figure 1(b) as broken vertical lines at pressures corresponding to the compressive strength of the rock.

Normally the allowable bearing pressure should not exceed the permitted stress in the foundation concrete.

The curves of Figure 1 should only be used for preliminary design purposes or for conservative design of foundations for relatively small structures. Heavier foundations or foundation structures that are sensitive to settlement should be considered individually having regard to the size of the foundation, variation in the strength and degree and nature of the fracturing in the rock mass, both with depth and laterally (Hobbs 1974).

The presence or possible existence of layers of weaker rock or compressible materials, including clay, within the zone of influence of the foundations and the effect of open joints and fissures and of any infillings should be taken into account. Staining on joints and fissures due to present or past water percolation is an indication that these features are open in varying degrees, the more open the fissures the higher the permeability of the rock mass and the greater its compressibility. Where staining or other indications of open joints are present, the allowable bearing pressure should be reduced, as the values shown in the figures have been based on the assumption of reasonably tight discontinuities. Special consideration should be given to foundations located over heavily shattered rock or on slopes affected by cambering or valley bulging where the rock may be severely loosened to considerable depths and may require grouting in order to provide adequate bearing capacity and reduce settlement.

Unweathered stratified rocks under high lateral stresses may have sprung upwards near the ground surface on relief of the geological overburden load, resulting in opening of the bedding discontinuities. This condition can lead to unexpected settlements which may occur suddenly unless the rock is grouted. Dangerous conditions may develop where stratified rocks, joints or old shear surfaces are dipping towards valleys, cuttings, deep excavations and basements.

In stronger more rigid rocks in which compression within the rock mass from the imposed load is relatively small, loosening of the formation by blasting, ripping or rough handling may lead to higher than expected settlements. Loosening or disruption of the formation can also be caused by water pressure in underlying jointed rock causing uplift of less permeable rock when excavating below the water table. Relief wells, dowelling or anchoring installed in good time are essential in these circumstances.

Very heavy, large structures may so compress the joint or fissure system within the rock mass as to considerably reduce its permeability. The effect of such a possibility on the groundwater regime and on the stability and performance of the proposed structure and on neighbouring structures should be considered where the particular circumstances require.

2.2.2.3.2 Allowable bearing pressure on non-cohesive soils

2.2.2.3.2.1 General. This category includes gravels and sands which are composed mainly of the larger sized particles derived from the weathering of rocks. Their particle sizes range upward from 0.06 mm (see BS 5930). Their behaviour under load depends on the following factors.

- a) *Composition.* Sand and gravel deposits may contain lenses of silt, clay or even organic deposits. On occasions some of the particles making up the sand and gravel may be composed of relatively soft and compressible material, e.g. chalk and mica. Unless the deposit consists of hard mineral particles, the presumed bearing values in Table 1 may have to be reduced.
- b) *Density.* This is a very important factor; on loose deposits the allowable bearing pressures are generally low, and large settlements may occur.
- c) *Grading.* Deposits composed of a mixture of different sized particles are in general stronger than those where the particles are nearly all one size.
- d) *Size of particles.* In general, the larger the particles, the higher is the allowable bearing pressure.
- e) *Shape of particles.* In general, well rounded particles give a lower allowable bearing pressure than angular particles.
- f) *Groundwater level.* This affects the stability of foundations, since the allowable bearing pressure is reduced when the groundwater level is near the foundation level.
- g) *Flow of water.* This may have an adverse effect by eroding the finer particles.

h) *Degree of lateral confinement.* All non-cohesive deposits need to be confined laterally. If non-cohesive deposits are not naturally and adequately confined, particularly if loose or fine or below the groundwater table, then artificial confinement may be necessary.

The combined effect of grading, particle size, shape and density is that gravels tend to have higher allowable bearing pressures than sand. In arriving at the allowable bearing pressure of foundations on non-cohesive soils, both the ultimate bearing capacity and the permissible settlements should be taken into account. Normally, settlement governs design but for very narrow foundations the design is likely to be governed by the need for an adequate factor of safety against shear failure, while for wide foundations the settlements are likely to be the criteria.

Loose to medium-dense deposits of non-cohesive soils are likely to settle when either the foundation or the ground is subjected to vibrations, even when these are applied some distance away.

2.2.2.3.2.2 *Field and laboratory observations and measurement.* Satisfactory undisturbed samples of non-cohesive soils are difficult to obtain from boreholes, and the results of field tests and observations are commonly used to evaluate the allowable bearing pressure.

Field tests may consist of small-scale loading tests or various types of penetration or sounding tests. The most common of these tests are the standard penetration test and the Dutch cone test (see BS 5930). These tests, if carefully executed in soil undisturbed by boring operations, enable an estimate of the relative density of packing to be made (Terzaghi and Peck 1967; Sutherland 1975). It should be noted that great care should be taken to avoid disturbance of non-cohesive soils, particularly below the water table; misleading penetration results are often obtained because of disturbance. Sands or gravels may be classified by the standard penetration test into the broad groups given in Table 5.

Table 5 — Classification of sands and gravels by standard penetration test

Relative density of packing	Number of blows for 0.3 m penetration
Loose	Less than 10
Medium dense	10 to 30
Dense (or compact)	More than 30

A rough correction for the effect of depth may be made (Department of the Environment 1976).

When examined in excavations, dense non-cohesive soils offer a high resistance to penetration by a hand bar. Some beds of gravel are so tightly packed that the material requires a pick for removal and when struck may give out a metallic ring.

In the loose condition, non-cohesive soils offer only a small resistance to penetration by a hand bar and can easily be excavated with a shovel.

Disturbed samples may be used to examine the nature of the particles and to determine the particle size distribution. Composition and grading are of particular importance.

2.2.2.3.2.3 *Ultimate bearing capacity.* The ultimate bearing capacity of foundations on non-cohesive soils depends not only on the angle of shearing resistance of the soil, but also increases with the breadth and the depth of embedment of the foundation.

The angle of shearing resistance depends primarily on the grading and the relative density, and this may be assessed from the results of penetration or other tests or by the inspection of the deposit in the field.

Empirical methods for relating the ultimate bearing capacity to the results of field tests are described in a number of textbooks and papers e.g. Skempton (1975).

The position of the groundwater level, relative to a foundation, has an important effect on the ultimate bearing capacity of foundations on non-cohesive soils. With high groundwater levels, the effective stresses in the ground are lower than when the soils immediately below the foundations are dry, and the ultimate bearing capacity is reduced. In extreme cases, the ultimate bearing capacity of a flooded foundation may be only one-half of that in a dry condition.

For a given relative density of packing, gravels or mixtures of sand and gravel have a higher ultimate bearing capacity than the finer-grained non-cohesive soils.

Wide and deep foundations on non-cohesive soils may have a very high ultimate bearing capacity, but the allowable bearing pressures will have to be very much smaller than the ultimate bearing capacity in order to limit the settlements.

2.2.2.3.2.4 *Settlement.* The settlement of a foundation on non-cohesive soils depends on the bearing pressure, the relative density of packing and the width and depth of embedment of the foundation. Very dense deposits can sustain heavy foundation pressures with only small settlements, but the settlement of foundations on loose deposits may be considerable.

For a given relationship between relative density and bearing pressure, well graded gravels or mixtures of sand and gravel settle less than the finer non-cohesive soils, and the allowable bearing pressures will be correspondingly higher.

The magnitude of the settlement of foundations on non-cohesive soils may be conservatively estimated using empirical relationships between settlement, width of the foundation and the relative density (Skempton 1975).

In general, the settlements of foundations on dry non-cohesive soils well above the water table are less than those of foundations located close to the water table, and in assessing the allowable bearing pressure due account should be taken of the water table and its possible fluctuations. In extreme cases, the settlement of a foundation subjected to flooding may be twice that of a foundation located well above the water table.

The settlements due to the static loading of foundations on non-cohesive soils take place as the loads are applied. Additional settlements may, however, occur if the groundwater level fluctuates or if the foundation or the soil are subjected to vibrations. The effect of vibrations is most marked in very loose soils and, if vibrations are to be expected, for example from nearby machinery, railways, highways, construction operations or from landslips or earthquake tremors, these soils should be compacted by the driving of displacement piles, deep vibration methods or other suitable means.

2.2.2.3.2.5 Special features of various types of deposit. Well graded deposits of non-cohesive soils are able to carry heavier foundation pressures than the more uniformly graded materials.

Fine uniform sands, especially in the loose state, are particularly susceptible to boiling when excavations are taken below the groundwater table. Care should be taken when excavating or pumping in these materials to avoid internal erosion or loss of ground, which might cause the settlement of adjacent structures. It may be necessary to confine these materials below foundations by the use of sheet piling.

Some loose wind-blown non-cohesive soils above the groundwater table may be weakly cemented with material which deteriorates in the presence of water. Any percolation or rise in the groundwater levels may lead to the weakening of the cementing medium and large settlements may ensue.

Unsuitable construction methods may lead to the disturbance of dense deposits of non-cohesive soils with a consequent reduction in the relative density of packing. In these circumstances the use of the predetermined allowable bearing pressure may result in excessive settlements.

Where beds of sand and gravel contain softer material such as chalk and mica, the allowable bearing pressure may need to be reduced.

In designing foundations on non-cohesive soils it should be ascertained that there is no danger of future instability resulting from dredging or scour. This is of particular importance in the design of bridge piers, because the depth of scour action can be several times greater than the rise in water level.

2.2.2.3.3 Allowable bearing pressure on cohesive soils

2.2.2.3.3.1 General. Cohesive soils consist of the finer and altered products of rock weathering. In their natural state they possess cohesion and plasticity (see BS 5930). Particles of diameter less than 0.002 mm are said to be of clay size, but in fact pure clays are rare and natural clay deposits may contain up to 70 % or even more of material belonging to the sand and silt grades.

The most marked characteristic of cohesive soils from the engineering standpoint is their susceptibility to slow volume changes. Changes in load give rise to changes in volume when water is expelled or taken up, with consequent settlement or swelling. These volume changes can occur even at bearing pressures below the allowable bearing pressure or independently of applied load (see 2.1.2.4). In most cases the volume change gives rise to settlement, and, unlike the non-cohesive sediments, this settlement in cohesive soils may take many years to reach its full value.

Short-period loading tests can therefore give no measure of its probable amount. In addition, immediate settlements due to “elastic” yield take place as the load is applied.

The examination of existing similar structures near the site may give some indication of the approximate order of settlement. A quantitative approach may be made on the basis of measurement of the relevant soil properties.

The process of consolidation may be partially reversible. Beds of clay, if relieved of pressure, may absorb water, with a consequent increase in volume. Deep excavations, for example for bridge piers, have been known to cause a rise of several inches, greatly increasing the gross settlement when the full load is applied (see section 4).

The magnitude of settlement may in many cases be the controlling factor in choosing the allowable bearing pressure, but in all cases it should be established that there is an adequate margin of safety against shear failure. Tests for shear strength on undisturbed samples in the laboratory or on undisturbed material in the field will generally give a good indication of their ultimate bearing capacity.

2.2.2.3.3.2 Field and laboratory observations and measurement. Field and laboratory observations and measurement are as follows.

a) *Sampling.* It is a great deal easier to obtain tube samples of cohesive soils from boreholes than it is to do the same for non-cohesive soils. Tube samples of cohesive soils are commonly referred to as “undisturbed samples”, although there are many limitations in obtaining truly undisturbed samples. Methods of sampling are described in BS 5930.

Boring and sampling tend to cause remoulding of soft sensitive clays. Disturbance and stress changes in stiff fissured clays or layered clays of variable permeability may make the sample not truly representative of the in situ condition. Large gravel and cobble-size particles in boulder clays may obstruct sampling. In some of these cases, some form of in situ testing may be a more suitable approach.

b) *Field tests.* Simple methods of estimating the consistency of clays in the field are given in Table 6 and BS 5930. The shear strength of soft sensitive soils may be measured by in situ vane tests (see BS 5930).

In soils with an appreciable stone content, such as certain boulder clays, and where undisturbed sampling and testing are impracticable, loading tests may sometimes be employed with advantage. Loading tests are discussed in 2.2.1.7 and BS 5930. Penetration tests are sometimes used in these soils to compare consistencies. The limitations of such tests should be fully appreciated (see BS 5930).

c) *Laboratory tests.* In assessing the engineering behaviour of cohesive soils it is useful to obtain values of the liquid and plastic limits and the in situ water content (see BS 1377). It is normal practice to determine the shear strength and consolidation characteristics of undisturbed samples in order to evaluate the allowable bearing pressure. Methods of measuring the shear strength and consolidation characteristics of undisturbed samples of cohesive soils are described in appendix H of BS 5930:1981 and BS 1377.

In Table 1, clays and silts are classified on the basis of consistency, which is related to the corresponding undrained shear strength as in Table 6 (see BS 5930).

It is common practice to assess the shear strength of stiff fissured clays by undrained triaxial tests, but careful consideration should be given to the effects of disturbance and stress changes in sampling; the results of such tests may be misleading when applied in situations where stresses in the ground are decreased as a result of excavation either during construction or subsequently.

2.2.2.3.3.3 Ultimate bearing capacity. The ultimate bearing capacity of foundations on cohesive soils depends on the shear strength of the soil and the depth and shape of the foundations (Skempton 1951; Skempton and Bjerrum 1957; Little 1961). Where soil conditions are uniform, the ultimate bearing capacity does not vary appreciably with the width of the foundation. The shear strength may be determined from the results of undrained shear tests or in situ vane tests as mentioned in 2.2.2.3.3.2. In the case of silts, an increase in strength during construction can often be allowed. On clay soils, embankments are usually constructed sufficiently slowly to permit an allowance of some increased strength during the construction period.

Table 6 — Undrained (immediate) shear strength of cohesive soils

Consistency			Undrained (immediate) shear strength	
In accordance with BS 5930	Widely used	Field indications		
			kN/m ²	kgf/cm ² tonf/ft ²
Very stiff	Very stiff or hard	Brittle or very tough	Greater than 150	Greater than 1.5
Stiff	Stiff	Cannot be moulded in the fingers	100 to 150	1.0 to 1.5
	Firm to stiff		75 to 100	0.75 to 1.0
Firm	Firm	Can be moulded in the fingers by strong pressure	50 to 75	0.5 to 0.75
	Soft to firm		40 to 50	0.4 to 0.5
Soft	Soft	Easily moulded in the fingers	20 to 40	0.2 to 0.4
Very soft	Very soft	Exudes between the fingers when squeezed in the fist	Less than 20	Less than 0.2

2.2.2.3.3.4 Settlement. For cohesive soils it is always necessary to ensure that the foundation is safe against shear failure by taking a presumed bearing value not greater than the ultimate bearing capacity divided by a suitable factor of safety, conventionally of the order of 2 to 3.

When a layer of softer clay occurs at some depth below a foundation, the possibility of plastic deformation of this layer needs to be checked. The net increase of pressure in the soft clay layer due to the foundation loading should be evaluated. This net increase of pressure will have to be limited to a value which will provide an adequate factor of safety against shear failure in the soft layer.

It is also necessary to ensure that the total and differential settlements of the proposed foundations are not too great. After the presumed bearing value has been assessed, the settlement under this pressure is estimated, and if it is too great the foundations will need to be redesigned either by reducing the bearing pressure or by taking the foundations down to a lower level at which less settlement will occur. When founding a structure on peat or organic soils is unavoidable, it should be appreciated that the secondary consolidation referred to in 2.1.2.3.3 gives rise to considerable and large settlements which may be prolonged indefinitely. Specialist advice should be sought in this situation.

The nature of settlement in cohesive soils is discussed in 2.1.2.3.3. It is dependent on the bearing pressure, the compressibility of the soil and the dimensions of the foundation, namely depth, width and shape. As with non-cohesive soils, for the same bearing pressure wide foundations settle more than narrow foundations and, in general, the allowable bearing pressures on wide foundations will be smaller than on narrow foundations, in order to keep the settlements within acceptable limits.

The magnitudes of the settlement of foundations on cohesive soils may be estimated from the results of laboratory tests in which the consolidation characteristics and Young's modulus are determined (Terzaghi 1943; Skempton 1951; Gibson and Lumb 1953). Values of elastic constants obtained from in situ tests are in many cases preferable to those obtained from laboratory tests.

The conventional method of estimating settlement is based on standard oedometer tests and consideration of one-dimensional consolidation (Terzaghi 1943; Skempton 1951; Gibson and Lumb 1953). This method frequently overestimates the rate of settlement, particularly in soils having a horizontal permeability greater than the Vertical. In such cases more elaborate laboratory testing will be necessary, together with a three-dimensional analysis of the consolidation (Schiffman et al. 1969).

2.2.2.3.4 Peat and organic soils. Peat and organic soils are materials with a high proportion of fibrous or spongy textured vegetable matter formed by the decay of plants, mixed with varying proportions of fine sand, silt or clay. All these soils are highly compressible, and even lightly loaded foundations will be subject to considerable settlements over a long period if placed on them. For this reason these soils are not suitable for carrying the loads from important structures. Lowering of the groundwater also produces a considerable and prolonged settlement.

In general, it is necessary to carry foundations down through peat and organic soil to a reliable bearing stratum below.

Care is necessary with services in such ground, particularly where they enter the structure.

2.2.2.3.5 Made ground and fill. All made ground should be treated as suspect because of the likelihood of extreme variability. Any proposal to found a structure on made ground should be investigated with extreme care. Made ground may be insanitary or may contain injurious chemicals and toxic and flammable gases. Industrial waste or town refuse may still be in a state of chemical activity, and waste often ignites and burns below ground. Loading tests may be completely misleading because of the variability of such deposits.

Where a filling operation is adequately controlled by placing selected suitable fill material in sufficiently thin layers, properly compacted, then the methods described in this code may be used to assess the allowable bearing pressure. Suitable fill materials include most graded natural granular materials and certain waste materials such as selected colliery waste and pulverized fuel ash. Some cohesive materials may be suitable but organic materials have to be excluded (Skempton 1964).

2.2.2.3.6 Non-uniform site conditions

2.2.2.3.6.1 General. Non-uniform conditions include layers of softer material, non-level sites and strata of varying thickness. Variations can be relatively systematic or random. They are related to a number of factors, including both broad and minor variations in deposition conditions, subsequent tectonic effects and the development of significant irregularities in topography.

An example would be the occurrence of irregular lenses of soft clay within a deposit of alluvial gravel. In glacial deposits, lenses of firm laminated clays can occur within deposits of very stiff boulder clay; "kettle holes" may occur, filled with softer deposits than those immediately surrounding.

2.2.2.3.6.2 Non-level sites. On sloping ground on clay soils there is always a tendency for the upper layers of soil to move downhill, the extent depending on the type of soil, the angle of slope, groundwater regime and climatic conditions. In some cases the uneven surface of a slope on virgin ground will indicate that the area is subject to small landslips, and therefore will obviously necessitate special precautions if used as a building site. In other cases there may be a creep of the surface layers of the soil, the indications being curved tree trunks, tilted fence posts, tilted boundary walls, etc. This kind of movement may occur on slopes no steeper than 1 in 10, particularly where the subsoil is a clay of the stiff, fissured type. The moving zone may vary from less than a metre to several metres in depth.

Any site showing recent landslide or instability will normally be avoided, but it is possible that a sloping site may already be very close to instability before building operations begin, and small additions to or alterations of the natural loading may precipitate failure. Instability may develop even after a long period of apparent stability, particularly in stiff, fissured and overconsolidated clay soil. Clay cuttings in such materials have slipped after remaining apparently stable for some 50 years, and natural clay slopes which had not moved within living memory have recently slipped (Skempton 1964).

New works may interfere with the existing natural or artificial drainage of the ground and adversely affect the stability.

Land drainage may also require special consideration, particularly on the uphill side of a building, to divert the natural flow of water away from the foundations. If excavation involves cutting through existing land drains, they should be carefully diverted into the ground-drainage system.

Cuttings and excavations adjoining foundations reduce stability and increase the likelihood of differential settlement. Their effect should be investigated not only when the cutting or excavation exists already but also when there is a possibility that they may be made subsequently (see 2.3.2.7). Where the structure is to be placed on sloping ground, additional complications are introduced. The ground itself, particularly if of clay, may be subject to creep or other forms of instability, which may be exaggerated if the strata dip in the same direction as the ground surface. If the slope of the ground is large, the overall stability of the slope and substructure may be affected. These aspects should be carefully investigated.

2.2.2.3.6.3 Strata of varying thickness. Strata of varying thickness, even when at appreciable depth, may increase differential settlement. Where necessary, calculations should be made of the settlement expected from the different thicknesses of strata, and the structure should be designed accordingly. In extreme cases of change in thickness of soft strata the stability of foundations may be affected. Site investigations should always be sufficiently extensive to ensure that significant variations in strata thickness are detected.

2.2.2.3.6.4 Layers of softer material. The need to avoid overstressing and excessive settlement in soft layers has been described in 2.2.2.3.3.4. Some soils and rocks have layers of harder material between thin layers of softer material, which may be overlooked unless very careful investigation is carried out. The softer layers may undergo significant alteration in properties if the loading on them is increased or decreased by the construction or affected by accompanying changes in groundwater conditions.

2.3 Structural considerations

2.3.1 General

The design of the substructure and superstructure should be related to the characteristics of the ground. All foundations move as a result of applied load; there may also be ground movements independent of load (see 2.1.2.4). The extent of such movements should be estimated and their likely effect on the structure should be considered. The designer should decide, having regard to the user's requirements, whether the effects of such movements can be tolerated.

2.3.2 Interdependence of ground, substructure and superstructure

NOTE For further information see Hooper (1979) and "Structure – soil interaction report" published by the Institution of Structural Engineers (ISE), 11 Upper Belgrave Street, London SW1X 8BH.

2.3.2.1 General. To obtain maximum economy the supporting ground, substructure and superstructure should be studied as a whole. In general it will be necessary to consider the overall stiffness of an important structure together with its substructure and their interrelation with ground settlements. Although there are theoretical treatments of this subject, they are difficult to apply to many practical problems, and experience and judgement form the best guide at present.

2.3.2.2 Variability of ground conditions. The importance of determining the properties of all strata likely to be affected by the structural load has been discussed in 2.2.1. However, the properties of a particular stratum can vary between one part of a site and another, and it is important to examine such variations and their influence on the design (see also 2.2.2.3.6).

In some cases it will not be economic to delineate all softer zones within a stratum prior to construction, but it is important to make an assessment of their probable occurrence, based on the information available, so that means of dealing with such zones can be formulated beforehand. With rafts, for example, it may be necessary to allow in the design for bridging over weak zones which it is neither feasible nor economic to remove.

2.3.2.3 Differential settlements. The total settlement of a structure may interfere with some special aspect of its functions, e.g. the effect on services, but it is the differential or relative settlement which, in general, produces the secondary stresses which are of such importance in the completed structure.

Not only is it frequently difficult to estimate the numerical value of differential settlement, but it is similarly difficult to assess the effect on the structure. Here again, experience is the best guide.

Differential settlement can be reduced by providing a stiff structure or substructure or a combination of both. A deep hollow box-like raft has the additional advantage of reducing the increase in net loading intensity where this is desirable as well as producing more uniform settlement. Where columns are provided with independent pad or piled foundations, it may be advantageous to increase the net loading intensity beneath the more lightly loaded columns to produce more uniform settlement. If time permits, another possible method of reducing settlement is by preloading the soil.

Occasionally, special arrangements are made for jacking up parts of the structure where particularly severe settlement is expected. This method is most commonly used for bridge piers, but it has wider application.

Structures may also be split into a number of smaller independent units to minimize the effect of differential settlement between them, or the stiffness of a framed structure may be reduced by modifying any cross bracing or altering the connections between the frame and the cladding. When large differential settlements are expected, it is particularly important to avoid continuous structures that are sensitive to settlement. The effect of differential settlement on services, pavements and rail tracks should also be considered, particularly at discontinuities. In considering the effects of differential settlement between parts of a structure or adjacent structures, it should be borne in mind that in many cases a part of the settlement will occur during construction.

Whilst it is increasingly common for estimates of settlement to be made at the design stage, it is rather uncommon for measurements of settlement to be made during and after construction. The lack of such data makes any statement regarding tolerable differential settlement for various types of structure somewhat tentative. Wherever possible systematic monitoring of the settlement of a structure should be carried out, as this will provide valuable information on its performance and provide data for design of subsequent structures.

2.3.2.4 Foundation loads

2.3.2.4.1 General. The maximum load on the foundation will be the sum of the dead, live (imposed) and wind loads. The maximum bearing pressure will depend on the distribution, direction and eccentricity of the loads. However, the load to be considered in the various aspects of design will not necessarily include the full value of these loads. Loads used in the design of foundations should be unfactored values and not the factored loads calculated in BS 8110.

2.3.2.4.2 Dead and live loads. The dead load will include the weight of the foundations and of any backfilled material above the foundations.

Some live loads are directly calculable. In other cases, live loads will include allowances for loads which may not necessarily be realized in practice. These are governed to some extent by codes of practice and statutory requirements.

2.3.2.4.3 Wind loading. Where the foundation loading beneath a structure due to wind is a relatively small proportion of the total loading, it may be permissible to ignore the wind loading in the assessment of the allowable bearing pressure, provided the overall factor of safety against shear failure is adequate. For example, where individual foundation loads due to wind are less than 25 % of the loadings due to dead and live loads, the wind loads may be ignored in this assessment. Where this ratio exceeds 25 %, foundations may be so proportioned that the pressure due to combined dead, live and wind loads does not exceed the allowable bearing pressure by more than 25 %.

2.3.2.4.4 Ultimate bearing capacity. When the factor of safety, against shear failure is being checked, the loads to be used in computing the loading intensity under foundations should be the dead and live loads of the structure, plus a due allowance for wind load if indicated by 2.3.2.4.3, reduced by the intensity of immediately surrounding dead loads. In most cases this calculation can be done by using the net loading intensity. However, in certain cases, such as in an isolated foundation in a basement excavation or an isolated bridge pier in an excavated area, the gross loading intensity has to be used.

2.3.2.4.5 Settlement. In considering settlement, a close estimate should be made of the average dead and live load likely to be imposed, since loads that act infrequently and for short periods are less important from a consolidation point of view. On the other hand, heavy vibrations or transient loads may cause settlement of sands and gravels, particularly where they exist in a loose condition (see 2.3.2.5).

2.3.2.4.6 Eccentric loading. Eccentric loading of foundations due to any cause other than wind pressures should be fully investigated, and the maximum pressure should not exceed the allowable bearing pressure. Foundations for structures carrying live loads such as bridges and crane gantries may be subjected to considerable eccentricity of loading.

2.3.2.4.7 Non-vertical loading. If a load on a horizontal foundation is inclined to the vertical, careful consideration should be given to the capacity of the ground to resist the horizontal component, because this has a pronounced influence on the vertical loading which the foundation can safely carry and may lead to reduced values of the allowable bearing pressure.

The following simple procedure is suggested for the design of foundations under non-vertical loads. Resolve the inclined load into its vertical and horizontal components and apply the rule:

$$\frac{V}{P_v} + \frac{H}{P_h} < 1,$$

where

- V is the vertical component of the inclined load;
- H is the horizontal component of the inclined load,
- P_v is the allowable vertical load;
- P_h is the allowable horizontal load.

The allowable vertical load can be assessed from the principles described in 2.2.2; the allowable horizontal load would equal the allowable passive resistance of any of the ground in contact with the vertical face of the footing, plus the allowable friction and/or adhesion on the horizontal base of the footing corresponding to the applied vertical component. For calculation purposes, reference should be made to the Civil Engineering Code of Practice No. 2.

In practice, for most footings the rule given above is on the safe side, but with shallow footings on non-cohesive soil (with the ratio of the depth to breadth less than one-quarter) more conservative values may be necessary, especially where the horizontal component is large compared with the vertical one.

In developing the maximum safe passive resistance on a vertical face an appreciable horizontal movement may occur, so that only a fraction of this resistance can usually be utilized in practice.

If the requirements of this rule cannot be met, provision should be made for the horizontal thrust to be taken by some other part of the structure or by raking piles, by tying back to a line of sheet piling or by other means (see DIN 4150).

2.3.2.5 Dynamic loads

2.3.2.5.1 General. The dynamic loads considered include both impulsive and pulsating loads where the time period is sufficiently short to induce a vibratory response into the structure and its foundations. In some areas, special attention may be required to the possible effects of earthquakes. The supporting ground under dynamic loading can suffer progressive non-reversible strains. With granular materials in a loose to medium dense state, settlement can be induced by vibration even at a distance. In fine grained soils there may be instability due to the build-up of excess pore pressure. Weakly cemented materials may undergo progressive deterioration. Overconsolidated clays may soften under pulsating loading. In the latter case, the softening takes place on unloading and where the soil is free to take up water without a counteracting consolidation load, e.g. under railway sleepers.

2.3.2.5.2 Soils data. Qualitative information concerning the behaviour of foundations under dynamic loading can be obtained from a knowledge of the relative density and of index properties such as bulk density, moisture content and particle size distribution. It is noted in the case of granular soils in particular that the relative density, rather than absolute density, is significant, and may require laboratory measurements of maximum and minimum densities. Particular attention should be paid to obtaining samples with a minimum disturbance, and in saturated fine granular material care should be taken that such disturbance does not give rise to apparently very low relative densities in situ. Estimation of the relative density by means of in situ testing methods may also be employed.

In addition, information is required about the dynamic elastic properties of the ground; the rigidity modulus G or the compression modulus E with Poisson ratio ν , or some equivalent, should be determined. The use of small vibrators for in situ determinations of dynamic parameters is desirable. Care should be taken to ensure that the vibrators have a resultant unbalanced vertical force. They should be well bedded into the soil to be tested, and the static load they impose should be high enough to resist any tendency to “jump” off the ground during operation. By varying the frequency of the applied dynamic force and measuring the amplitude both at the vibrator itself and in the adjacent soil it is possible to derive a dynamic modulus of rigidity and a damping factor from the amplitude frequency curves.

2.3.2.5.3 Design considerations. The first essential of a design is to ensure that resonance will not occur between the frequency of the pulsating load and a critical frequency in the foundation-soil system. Resonance may lead to excessive amplitudes of vibration of the system.

Even when resonance is avoided it is still necessary to limit amplitudes to levels that can be tolerated by the machine and the foundation. While the level of vibration may be acceptable by the machine and the soil from the point of view of settlement, it may give rise to resonance in windows, doors and partitions of associated structures, and this may call for reduction of amplitudes or isolation of the vibrating system.

There have been numerous attempts to draw up tables of tolerable vertical vibration amplitudes against frequency for various types of machine and structure. These presentations can be referred to in DIN 4150 and Steffens (1952).

Detailed information concerning methods of design of foundations to sustain dynamic loads can be found in a number of publications, including Barkan (1962), Whitman (1966); Whitman and Richart (1967).

The preferred method of analysis of foundation block response to dynamic load is based upon the theory of the “elastic half space” and requires the dynamic elastic moduli of the ground to be either measured or estimated. A full discussion can be found in CP 2012.

A number of factors to be considered in the design are as follows.

a) If the preliminary calculations indicate that the critical frequency of the foundation system is too close to that of the operating frequency, the following measures can be taken.

1) If the operating frequencies are lower than the critical frequencies for the foundation, as is usual for reciprocating engines, then the natural frequency for the foundation may be further increased by increasing the contact area of the foundation to the soil and its moments of inertia. Increasing the rigidity of the foundation soil system will also increase the natural frequency. This can be achieved by grouting where feasible or by piling.

2) If the operating frequency is likely to be higher than the critical frequency of the foundation, then an enlargement of the foundation contact area and an increase in soil rigidity could worsen the situation. Under these conditions it is better to decrease even further the natural frequency of the foundation; this can be achieved by increasing the foundation mass with no change in area, resulting in a greater static pressure on the soil, the upper limit being the allowable bearing pressure.

3) For some reciprocating machines, anti-vibration mountings may be inserted between the machine and the foundation.

b) The effect of a number of vibrating sources close together cannot be computed theoretically, and it is therefore advisable that such combination effects as can arise from multiple units be avoided as far as possible by placing them on a combined foundation. The interconnections should be such that the stiffness is high and deformations taking place in the combined foundation are small compared with the amplitude of vibration.

c) In general, the foundations of machinery should not be connected with the foundations of the structure enclosing it. Occasionally, it may be advisable to make a provision for the connection of the foundation block to a floor slab which itself is separated from the footings, walls or columns supporting the roof loads. This enables remedial measures to be economically applied in the event of excessive horizontal or rocking vibrations.

d) Typical examples of impact sources are forging hammers, and there is considerable literature giving practical details for their design. Should preliminary calculations indicate that the amplitudes of a vibration cannot be reduced to permissible levels without uneconomic foundation blocks, it may be necessary to incorporate an anti-vibration mounting consisting of spring elements, often with the addition of damper brackets, e.g. oil dampers. The spring and dampers are generally placed in an inertia block between the anvil and the foundation element, which is usually a concretelined pit.

2.3.2.6 Distribution of contact pressure under foundations. In the structural design of a foundation the computation of the bending moments requires a knowledge of the distribution of contact pressure beneath the foundation. For most footings, the simple assumption is made that the contact pressure is distributed uniformly over the effective foundation area. For footings on sand, the actual contact pressure tends to decrease from centre to rim, so that the real bending moments are usually less than the computed ones. For rigid footings on soft cohesive soils, the contact pressure may increase towards the rim, and the real moments may exceed the computed ones. However, the difference is usually amply covered by the margin of safety customarily provided in structural design.

In the case of large spread foundations and raft foundations, the design may require a detailed investigation of the contact pressure distribution. This is a complex problem which calls for a close study of the soil conditions at the site and, for considerations of the flexural rigidity, of the foundations and superstructure. At present no satisfactory rigorous solution exists, and the number of records of field observations of stress distribution under large foundations are still very few.

In some cases, a trial and error approach may be attempted in which compatibility is sought between calculated differential settlements for various contact pressure distributions and the corresponding calculated deformations of the structure.

For a flexible structure, the soil reactions correspond to the loading on the raft, but in this case serious consideration needs to be given to the differential settlements that would be induced by this load distribution.

Although, as an approximation, the soil reaction over the base of a rigid structure may be taken as uniform, if the distribution of loads over the raft is not uniform, or if the supporting soil is relatively compressible, very severe bending moments may be induced. If the structure rests on sand, the differential settlement may be kept sufficiently small to have no injurious effect on the superstructure.

In the case of clays, it may not be possible to construct a substructure stiff enough to keep the differential settlements within prescribed limits.

Structural separation into a number of units is one method of overcoming the high bending moments. As an alternative, it may be possible to use partial flotation by constructing basements or hollow substructures of different depths, so arranged as to make a more favourable distribution of net pressure.

2.3.2.7 Changes in ground conditions after construction. When assessing the allowable bearing pressure, consideration should be given to possible deterioration during construction (see 2.4). Ground movements, independent of the applied load, which may lead to removal of support, are described in 2.1.2.4.

Erosion of soft or weakly cemented material along faults, dykes or in beds can lead to general or local loss of support. In chalk or limestone country, or in areas bordering this country, solution caverns, e.g. swallow holes (see 2.2.2.3.1.5) develop, which may be masked at the surface. Support can also be removed by down-cutting of streams or by soil creep on slopes or by landslides. Consideration is necessary of the way in which the site will be affected by present or foreseeable conditions.

Nearby excavations, whether for new foundations, cuttings or other reasons, can reduce the stability of existing foundations even when the new excavation is not taken below the existing foundation level. Similarly, subsequent filling can induce lateral loads or other loading on the foundations, e.g. an approach embankment behind a pile bridge abutment.

When designing new foundations adjacent to existing structures, the designer should consider the effect of the new structure on the existing work both during construction and subsequently. In addition to the effects of excavation, the presence of the new structure may, by reason of the increased load, induce settlement under adjacent structures. The effect of possible future developments on this proposed work after it is completed should be borne in mind.

Stability may be affected by inadequate backfilling, even when adjoining excavation has been adequately supported.

Mining subsidence and the abstraction of minerals by pumping or other means may induce severe stresses in the structure. In addition to vertical movements, the effect of surface stretching or compression of the ground should also be considered. Generally speaking, the effects of mining subsidence are more severe than the effects of differential settlement, therefore special precautions may be required. Disturbance may also be caused by tunnelling.

Vibrations, including those due to pile-driving, blasting or earthquake, may have adverse effects on non-cohesive soils.

Heavy seepage flows caused by pumping or other changes in the groundwater regime may cause removal of fines from sands and gravels leading to settlement. A rise or fall in water table may also disturb foundation equilibrium. Shrinkage or swelling of the soil, whether due to natural or artificial causes, is a further hazard.

Shrinkage of clay type soils may be increased by the drying effect produced by nearby trees and shrubs. Swelling may occur if they are cut down (see section 3).

Frost heave, naturally or artificially induced, may cause distortion of the structure, and the following thaw may remove support either generally or locally.

When a foundation is being designed, consideration should be given to all the possible causes of removal of support including excavation, even when it is not immediately contemplated. The precautions to be taken will depend, to some extent, on the hazard and the nature of the supporting ground, as well as on the type of substructure and superstructure.

NOTE Information on the deterioration of the substructure is given in section 10.

It should be noted that not all countries give an owner the right of redress for damage resulting from loss of support.

2.3.2.8 Mining subsidence

NOTE For further information see NCB (1965) and “Ground subsidence” published by the Institution of Civil Engineers (ICE), 1 to 7 Gt George St., London SW1P 3AA.

In mining areas, subsidence of the ground is liable to occur. The magnitude of the movement and its distribution over the area of the workings and their vicinity can be roughly estimated. Long-term estimation is often more uncertain. Attention should therefore be directed, where future subsidence is likely, to making the superstructure and substructure sufficiently strong or sufficiently flexible to cater for the probable ground movements. Long continuous buildings should be avoided, and extensive buildings should be divided up into independent sections of appropriate size, each with its own foundations. Expert advice should be sought. The appropriate mining authority, local records and archivists should also be consulted for subsidence history, previous structural failures, and for the intentions as to future mining.

2.3.3 Types of foundation

2.3.3.1 Choice of foundation. The choice of foundation is discussed in relation to the soil and rock types (see BS 5930 and Leonards (1963)).

2.3.3.2 Shallow foundations. Shallow foundations are generally suitable for most structures where rock occurs near ground level. The allowable bearing pressure will be relatively high, and the proportions of the foundations will normally be economic.

For shallow foundations on soft shales, mudstone and sound chalk under very heavy loadings, consideration of the settlement will be necessary. Special anchorages, for example stressed cable anchors or rock bolts, will be required for tension loading.

Where thinly bedded rocks or heavily shattered rocks occur consideration will need to be given to the weakest zones in the depth of significant stressing. In such cases compensated foundations or deep foundations penetrating to an adequate bearing stratum may be required.

Any softened zones at formation level should be removed, and where the rock is prone to softening the formation should be protected immediately.

Shallow foundations will often be suitable on dense gravel or sand deposits above the water table, and on firm or harder clays. In firm or stiff clays with heavy loadings the settlement may be important and deep foundations may be required to limit settlement.

Where shallow foundations are founded on clays, the possibility of ground movements due to seasonal variations in moisture content should be considered. Where this is a problem, it may be advisable to place the foundations at a lower level or to use a piled foundation.

Where compact sands or gravels are found below the water table, there is the danger that these may be disturbed in excavating to formation level. Such disturbance is more likely in sands and may be avoided or minimized by careful control of pumping, either by providing filters to sumps or by some groundwater lowering method, usually external to the excavation.

It may be necessary to limit loadings on loose granular soils to restrict settlement. The bearing capacity of such soils can sometimes be improved by deep compaction (see 6.3.5).

Shallow foundations are not generally suitable for soft or very soft clays and silts, peat and organic soils. Where such soils occur in comparatively thin layers, it may be expedient to excavate the soft material and replace it with better material or to provide compensated foundations.

Spread foundations should only be used with caution on made ground or uncontrolled fill and then only for lightly loaded structures that can tolerate the resulting settlements (see 2.2.2.3.5). Where fill has been adequately compacted under controlled conditions, the methods described in this code may be used to assess the allowable bearing pressure.

2.3.3.3 Deep foundations. When no adequate bearing stratum exists at depths shallow enough to permit economic construction of strip, pad or raft foundations, deep foundation including piles, cylinders or caissons will be needed. Alternatively, it may be advantageous to alter the design to include a deep basement, in some cases with another form of deep foundations.

Where rock occurs near ground level it is very uncommon that deep foundations are needed. Deep foundations may be needed for special structures where only extremely small deformations can be tolerated. When thinly bedded rocks or heavily shattered rocks, or weathered rocks occur, deep foundations will sometimes be necessary. When abandoned mine workings occur within significant depths it may be necessary either to fill up the workings with suitable material, or to carry the foundations below them.

Where loose sands occur near ground level it may be necessary to use deep foundations to reach an adequate bearing stratum. Alternatively, the bearing capacity of such soils can be improved by deep compaction. Where driven piles are used, this may result in compaction of the loose deposit but, with closely spaced piles, difficulty in driving may occur.

Where soft or very soft clays, silts, peat, organic soils and made ground or fill are met near ground level, it is often necessary to employ deep foundations penetrating to an adequate bearing stratum (see 2.3.3.2). More often than not this will take the form of a piled foundation, but where the form of construction requires, and when there are difficulties in driving or forming piles, cylinders and caissons may be employed or foundations constructed inside cofferdams or open excavations.

Where a bearing stratum is too deep for a conventional basement, it may be advantageous to use a semi-buoyant foundation (see 4.3). If an unsaturated clay layer exists, it may swell as a result of accumulation of water beneath the structure. Deep foundations (basement or piles) can be used to support the structure from the lower saturated zone, due attention being given to the possible effects of upward drag at the side of the foundations.

2.3.4 Exclusion of ground moisture

When the structure is located above the water table, moisture from the ground can be excluded from the occupied parts by damp-proof membranes. It may be desirable, as an alternative, to provide a properly designed and ventilated air space. When the occupied parts are in direct contact with the ground, damp-proof membranes may be required (see CP 102).

NOTE Attention is drawn to current building legislation.

2.4 Design in relation to construction procedure

2.4.1 General

The choice of foundation type will be determined partly by the soil conditions (see 2.3.3.1), partly by the physical and legal restrictions of the particular site and partly by the resources available. For example, the soil conditions at a site may indicate that driven piles are the most suitable foundation, but the vibration or noise resulting from piledriving operations may be detrimental to neighbouring structures or unacceptable to their users. At another site a particular design may be impracticable locally because of the lack of suitable resources in materials, plant or labour.

Structural design should be considered in relation to constructional methods. Economies can sometimes be effected by incorporating part of the temporary works into the permanent works. Details arising from any necessary constructional features such as access, timbering, shoring, temporary drains, pump discharge lines, compressed-air locks and other temporary obstructions should be borne in mind.

All cellular construction below groundwater level should be checked for flotation (see 2.1.3 and 2.1.4). It is not sufficient merely to assess the properties of the ground at a site seen possibly under favourable seasonal conditions during a site investigation and before the ground has been disturbed by constructional operations.

For example, the allowable bearing pressure should be the lowest value likely under the worst probable conditions (see 2.3.2.7 and 2.4.3).

Artificial improvement of the initial site conditions, which can influence the design of the foundations, may include replacement of unsuitable material, mechanical compaction, drainage including groundwater lowering, injection processes or freezing (see section 6).

2.4.2 Strength of partially completed structures

The foundations and substructure should have an adequate factor of safety under all conditions that may occur during the construction of the entire work. The processes of excavation, filling in, pumping, etc., should be arranged so as to avoid endangering the strength or stability of the partially completed structure. The strength of the partially completed structure will need to be sufficient to carry loads previously taken by temporary works which, as part of the constructional procedure, have to be transferred before work is completed. Additional temporary works may be necessary.

Basements or pits below groundwater level, which rely on the weight of the superstructure for their stability against flotation, may require day and night pumping, kentledge or drainage holes to ensure stability until the superstructure is sufficiently weighty.

Filling around high foundations piers should be brought up in such a way as not to endanger their stability.

2.4.3 Protection of the foundation soil

It is essential to consider the protection of the sides and bottoms of excavations. Traffic over bottomed-up excavations, exposure to the weather (rain, frost and drying conditions) and the action of water (floodwater or the removal of groundwater) are potential causes of deterioration.

A bearing stratum of clay or certain weak rocks is particularly susceptible to deterioration, and surface protection such as a layer of "blinding" concrete should be placed immediately after excavation to foundation level to protect the soil and to keep the new construction clean. Where there are problems of heave in clays, this concrete may be insufficient and special precautions will then be necessary.

Excavations should be kept free of water during concreting except in special circumstances when concrete should be placed by a suitable method such as by tremie pipe. This requires experienced operators and special care.

Where climatic or other conditions may result in deterioration of the sides of excavations, consideration should be given to their support and protection. The stability of the sides of excavations under all conditions that may occur during construction should be investigated. Special consideration should be given to the proximity of mechanical plant, spoil and material dumps to the edge of excavations (see BS 6031).

With a bearing stratum of sand, gravel or silt an upward flow of water into the excavation is particularly liable to cause deterioration. Special measures should be taken to stop or control the flow to prevent disturbance of the stratum; if such a flow seriously impairs the density and structure of the bearing material, the allowable bearing pressure may be reduced to that for the material in the loose condition. Similar precautions should be taken with foundations in layered deposits of variable permeability (Rowe 1968a).

Heavy pumping in excavations may tend to remove the fine material from sands and gravels, and protective filters should be used to prevent loss. Pumping from a number of points spread over an area may be preferable to heavy pumping from one central sump. It is preferable to draw the water away from the excavation rather than through the ground towards the excavation (see section 6).

2.4.4 Tolerances

Some forms of foundation work cannot be constructed with great accuracy and reasonable tolerances should be allowed.

Pile caps should be made large enough and pile groups should be designed to cover unavoidable variations up to 75 mm each way in the positions of individual piles. When the variation is in excess of 75 mm, it may be necessary to amend the design of the pile cap.

Caissons may deviate many millimetres out of line and sufficient allowance should be provided.

2.4.5 Extra cover in reinforced concrete when cast against excavated ground

A minimum additional allowance of 40 mm should be added to the cover recommended in Table 3.4 of BS 8110-1:1985 when concrete is cast directly against an excavated soil face. Moreover when the concrete is to be cast against an irregular surface an increase in the average allowance is advisable in order to ensure a 40 mm minimum cover. When the concrete is to be cast upon adequate blinding concrete, the cover excluding the thickness of the blinding should be as given in Table 3.4 of BS 8110-1:1985.

2.4.6 Retaining walls

While this subject is covered in Civil Engineering Code of Practice No 2, it is relevant to mention here that in situ walls by cast in situ methods, contiguous piles or sheet piles form part of structures to provide stability against earth movement.

A recent development in retaining wall technique is the use of polymer grid reinforcement and reference should be made to the literature on this technological advance where it is relevant to a foundation problem.

Section 3. Shallow foundations

3.1 General

Shallow foundations are taken to be those where the depth below finished ground level is less than 3 m and include strip, pad and raft foundations. The choice of 3 m is arbitrary; shallow foundations where the depth/breadth ratio is high may need to be designed as deep foundations (see 2.3.3). Similarly, some of the considerations in this section apply to deeper foundations where the depth/breadth ratio is low. The various forms of shallow foundations are illustrated in Powell (1979). For non-industrial buildings of not more than four storeys, reference should also be made to CP 101. The foundation requirements for many buildings can be met by shallow foundations. A preliminary discussion of shallow foundations and the principles of design have been covered in section 2. This section amplifies certain aspects that require special consideration and, in certain circumstances, specialist advice may also be necessary.

3.2 Design considerations

3.2.1 General

The depth to which foundations should be carried depends on three principal factors:

- a) reaching an adequate bearing stratum;
- b) in the case of clay soils, penetration with suitable precautions (see 3.2.8) below the zone where shrinkage and swelling due to seasonal weather changes, trees, shrubs and other vegetation are likely to cause appreciable movement (Burland et al. 1978; BRE Digests 240, 241 and 242).
- c) penetration below the zone in which trouble may be expected from frost.

Other factors such as ground movements, changes in groundwater conditions, long-term stability and heat transmitted from structures to the supporting ground may be important.

Shallow foundations are particularly vulnerable to certain soil conditions, e.g. sensitive clays, loose water-bearing sands and soils that change structure when loaded. Specialist advice should be sought where such conditions are indicated by ground investigation.

3.2.2 Allowable bearing pressure and settlement characteristics

The general principles relating to bearing pressure and settlement characteristics are covered in section 2. The centre of area of a foundation or group of foundations should be arranged vertically under the centre of gravity of the imposed loading (see 2.3). If this is not possible, the effects on the structure of rotation and settlement of the foundation need to be considered.

Where foundation support is provided by a number of separate bases these should, as far as practicable, be proportioned so that differential settlement is minimal (see 2.3).

3.2.3 Selection of types of shallow foundation

3.2.3.1 General. The selection of the appropriate type of shallow foundation will normally depend on the magnitude and disposition of the structural loads, the bearing capacity and settlement characteristics of the ground and the need to found in stable soil (see also 2.1).

A pad foundation is used for the purpose of distributing concentrated loads. Unless special conditions control the design, relatively heavy column loads make it advantageous to use pad foundations. However, strip foundations may be more appropriate where column loads are comparatively small and closely spaced or where walls are heavy or heavily loaded. Adjacent pad foundations can be combined or joined together with ground beams to support eccentric loads, to resist overturning or to oppose horizontal forces. Walls between columns may be carried on ground beams spanning between the pad foundation.

Where the allowable bearing pressure would result in large isolated foundations occupying the majority of the available area, it may be logical to join them to form a raft and spread the loads over the entire area. The combination of isolated foundations to form a raft sometimes results in a complex design and a large increase in the reinforcement requirement.

3.2.3.2 Extensions. In connection with the provision of foundations to an extension of an existing building, allowance should be made for differential movement of the foundations between the new and existing structure; such movement affects the structure above foundations. Where a degree of cracking and subsequent remedial work is not acceptable, provision for a joint between the extension and existing building should be considered. Where the foundations of an extension abut the foundations of the existing building, the stability of the existing foundations should be ensured.

3.2.4 Pad foundations

For buildings such as low rise dwellings and lightly framed structures, pad foundations may be of unreinforced concrete provided that the angle of spread of load from the pier or baseplate to the outer edge of the ground bearing does not exceed one (vertical) in one (horizontal) and that the stresses in the concrete due to bending and shear do not exceed those in Table 11 of Civil Engineering Code of Practice No. 2 1951. Where brick or masonry foundations have been used, the same rules apply with permissible stresses as given in Civil Engineering Code of Practice No. 2. For buildings other than low rise and lightly framed structures, it is customary to use reinforced concrete foundations.

The thickness of the foundation should under no circumstances be less than 150 mm and will generally be greater than this to maintain cover to reinforcement where provided.

Where concrete foundations are used they should be designed in accordance with the code of practice appropriate to the loading assumptions, e.g. BS 8110 or CP 114.

3.2.5 Strip foundations

3.2.5.1 General. Similar considerations to those for pad foundations apply to strip foundations. On sloping sites strip foundations should be on a horizontal bearing, stepped where necessary to maintain adequate depth.

3.2.5.2 Continuous wall foundations. In continuous wall foundations it is recommended that reinforcement be provided wherever an abrupt change in magnitude of load or variation in ground support occurs. Continuous wall foundations will normally be constructed in mass concrete provided that the angle of spread of load from the edge of the wall base to the outer edge of the ground bearing does not exceed one (vertical) in one (horizontal). Foundations on sloping ground, and where regrading is likely to take place, may require to be designed as retaining walls to accommodate steps between adjacent ground floor slabs or finished ground levels.

At all changes of level unreinforced foundations should be lapped at the steps for a distance at least equal to the thickness of the foundation or a minimum of 300 mm. Where the height of the step exceeds the thickness of the foundation, special precautions should be taken.

The thickness of reinforced strip foundations should be not less than 150 mm, and care should be taken with the excavation levels to ensure that this minimum thickness is maintained.

For the longitudinal spread of loads, sufficient reinforcement should be provided to withstand the tensions induced. It will sometimes be desirable to make strip foundations of inverted tee beam sections, in order to provide adequate stiffness in the longitudinal direction. At corners and junctions the longitudinal reinforcement of each wall foundation should be lapped.

3.2.5.3 Wide strip foundation. Where the use of ordinary strip foundations would overstress the bearing strata, wide strip foundations designed to transmit the foundation loads across the full width of the strip may be used. The depth below the finished ground level should be the same as for ordinary strip foundations.

3.2.5.4 Trench fill or deep strip footings. Where the nature of the ground is such that narrow trenches can be neatly cut down to the bearing stratum (but see 11.1.2), an economical foundation may be achieved by filling the trenches with concrete. When deciding the trench width, account should be taken of normal building tolerances in relation to setting out dimensions. Where the thickness of such a foundation is 500 mm or more, any step should be not greater than the concrete thickness and the lap at such a step should be at least 1 m or twice the step height, whichever is the greater.

Where fill or other loose materials occur above the bearing stratum adequate support is required to any excavation. Consideration may be given to the use of lean mix mass concrete replacement under ordinary strip footings placed at shallow depth. This mass concrete can be poured against either permanent or recoverable shuttering. This form of foundation provides a method of dealing with local areas where deeper foundations are required.

3.2.6 Raft foundations

3.2.6.1 General. Suitably designed raft foundations may be used in the following circumstances.

- a) For lightly loaded structures on soft natural ground where it is necessary to spread the load, or where there is variable support due to natural variations, made ground or weaker zones. In this case the function of the raft is to act as a bridge across the weaker zones. Rafts may form part of compensated foundations (see section 4).
- b) Where differential settlements are likely to be significant. The raft will require special design, involving an assessment of the disposition and distribution of loads, contact pressures and stiffness of the soil and raft (see 2.3).

c) Where mining subsidence is liable to occur. Design of the raft and structure to accommodate subsidence requires consideration by suitably qualified persons; the effects of mining may often involve provision of a flexible structure (see 2.3 and NCB 1965).

d) When buildings such as low rise dwellings and lightly framed structures have to be erected on soils susceptible to excessive shrinking and swelling, consideration should then be given to raft foundations placed on fully compacted selected fill material used as replacement for the surface layers.

e) For heavier structures where the ground conditions are such that there are unlikely to be significant differential settlements or heave, individual loads may be accommodated by isolated foundations. If these foundations occupy a large part of the available area they may, subject to design considerations, be joined to form a raft.

NOTE Compensated foundations involving rafts are discussed in section 4.

3.2.6.2 Service connections. The layout of service pipes, drains, etc., should be considered at the design stage so that the structural strength of a raft does not become unduly reduced by holes, ducts, etc. In soft soils particularly, provision should be made for relative movement where services and drains are supported partly on a raft and partly on soil, so that they may not be damaged by unequal settlement.

The layout and design of service pipes and ducts should be such as to allow their future maintenance without the necessity for breaking through the raft.

3.2.7 Short piling (see also section 7)

3.2.7.1 Where it is necessary to transmit foundation loads from buildings such as low rise dwellings or lightly framed structures through soft or made ground, or unstable formations or shrinking/swelling clays more than about 2 m deep, the use of short piles should be considered as an alternative to shallow foundations, particularly where a high groundwater table is encountered.

3.2.7.2 The type, method of construction, size and load capacity should be carefully considered in relation to the associated requirements of pile caps and ground beams necessary to transfer loads from the superstructure to the piles.

3.2.7.3 Many proprietary piling and ground consolidation techniques are available. Specialists should be consulted where other forms of foundation or methods of construction are found to be unsuitable (see section 6).

3.2.8 Shrinking and swelling of clay soils

3.2.8.1 Effect of seasonal weather changes. During periods of hot, dry weather a deficiency of water develops near the surface of the ground, and in clay soils this is associated with a decrease of volume causing ground shrinkage and the development of cracks and fissures. The extent and depth of the drying is a function of the soil type, lack of precipitation, evaporation potential of the weather, supply of water to the area from adjacent bodies of free water (such as rivers and aquifers) and, particularly, the extent and concentration of growing vegetation (Ward and Green 1948; Leonards 1963).

In periods of wet weather clay soils swell and the cracks and fissures tend to close, although the water deficiency developed in the ground during previous dry periods may be only partially replenished. Even when the precipitation is more than sufficient to do this a low infiltration rate may reduce replenishment and produce surface run off. A subsurface zone, or zones, deficient in water may persist.

Changes in the volume of clay ground are of world-wide importance in the design of foundations, particularly in areas with long, dry seasons; volume changes may well need consideration in Great Britain for foundations and basement structures located within, for example, 5 m of the finished ground surface. In areas of heavy overconsolidated clay, such as London, Gault, Kimmeridge, Oxford and Lias Clays, the seasonal drying in hot summers, usually with complete replenishment during the winter, occurs to a depth of 1.5 m to 2 m under crops and grass pastures where irrigation is not practised. During periods of hot dry weather shrinkage can develop in areas not normally associated with this phenomenon, thus shallow foundations should never be adopted in any cohesive soil without careful consideration of the vertical and horizontal movements that can occur due to moisture content changes (Tomlinson et al. 1978).

Under such conditions the strip foundations of traditional brick and masonry buildings should be founded at a minimum depth of 900 mm below finished ground level in order to reduce the relative movement to acceptable limits. Not all clays are subject to significant change and BRE Digest 240 discusses methods of assessing how susceptible a given clay is.

Vertical and lateral movements due to changes of moisture content of the surrounding ground may affect the performance of slabs, ground beams and substructure walls, even when deep footing or pile foundations have been used. In these situations consideration should be given to separation, articulation or movement gaps. Particular care is called for where wall returns act as stiff buttresses.

3.2.8.2 Effects of vegetation. The shrinkage of clay may be increased by the drying effect produced by the root system of shrubs, hedges and trees (Biddle 1983; Driscoll 1983). The range of influence of vegetation increases during dry periods and depends on the species, size and number. A permanent depletion of water in heavy clay ground may occur in the immediate vicinity of trees. For mature trees this may extend to a depth of 5 m. The depth to which seasonal or permanent drying has occurred in a particular location is indicated by the finest roots of plants and trees. When trees are removed from such areas the ground may swell slowly for many years (Samuels and Cheney 1974).

Care should be exercised to ensure that there is adequate space between new buildings and existing trees or the sites of trees that have been removed. Alternatively, special foundations founded at depths greater than 900 mm should be provided (see BS 5837).

Where swelling is expected, the foundation should be carried below the affected zone. Under these circumstances piled foundations are preferable with beams and floors suspended or cast on or against a suitable compressible layer so as to protect the structure from possible vertical or horizontal movement of the mass of clay within the foundation. Alternatively, the structure may be designed to accommodate the expected movements. Where piles are used, sleeving or other approved methods of separation should be provided, or the piles should be reinforced to cater for any tensile forces that may occur (BRE Digests 240 and 241).

3.2.8.3 Underground services. Shrinkage and swelling of clay soils may sometimes cause damage to underground services, such as drains and water mains. These may need to be provided with flexible joints, particularly where they enter the structure.

3.2.9 Other factors causing ground movement

3.2.9.1 Frost. Certain soils, particularly silts, chalks, fine sands and some clays, when situated close to the ground surface are subject to expansion when frozen, owing to the formation of ice (see TRRL Report No. 90 and Supplementary Report No. SR829). During construction when buildings are unglazed and unroofed, this expansion may be sufficient to lift portions of a structure. The conditions most likely to lead to this frost heave are severe and prolonged frost in situations where the groundwater level is close to the surface. If such conditions are likely to occur, the foundations should be carried below the frost line. Generally, in the British Isles a depth of 450 mm below ground level is sufficient for protection. Thin concrete rafts or floor slabs should not be left exposed to long periods of frosty weather where soils, susceptible to frost heave, are within 450 mm of the concrete surface (Tomlinson et al. 1978).

Deepening of foundations against frost effects is not usually required in party walls and in reinforced concrete raft foundations which are sufficiently stiff to withstand uplift pressures due to frost heave around the periphery of the raft. Care should be exercised to ensure that differential movements associated with frost heave which may affect unheated structures, such as garages, greenhouses and screen walls, do not affect abutting heated buildings.

Special precautions may be required for refrigerated installations where very severe conditions may be imposed.

3.2.9.2 Heating. Boiler installations, furnaces, kilns, underground cables and services and other artificial sources of heat may cause shrinkage of clay by drying out adjacent and underlying ground; such drying out can be to a substantial depth. These installations may also cause damage to the foundation material, particularly concrete. Care should be exercised when foundations are built in fill or other soils likely to contain combustible material.

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