



Code of practice for  
**Foundations —**  
**(Formerly CP 2004)**

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# Committees responsible for this British Standard

The preparation of this British Standard was entrusted by the Civil Engineering and Building Structures Standards Committee (CSB/-) to Technical Committee CSB/4, upon which the following bodies were represented:

- 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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## 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the direction of the Civil Engineering and Building Structures Standards Committee, supersedes CP 2004:1972 which is withdrawn.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 <sup>2</sup> <sup>a</sup>	kgf/cm <sup>2</sup> <sup>a</sup> tonf/ft <sup>2</sup>	
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	
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	

<sup>a</sup> 107.25 kN/m<sup>2</sup> = 1.094 kgf/cm<sup>2</sup> = 1 tonf/ft<sup>2</sup>.

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 <sup>2</sup>	kgf/cm <sup>2</sup> tonf/ft <sup>2</sup>
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.



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

Degree of weathering	Zone	Description	Notes	Presumed bearing value	
				kN/m <sup>2</sup>	kgf/cm <sup>2</sup> tonf/ft <sup>2</sup>
Fully weathered	IVb	Matrix only	Can be confused with solifluction or drift deposits, but contains no pebbles. Plastic slightly silty clay. May be fissured	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
3	Oolitic and marly limestones
	Well cemented sandstones
	Indurated carbonate mudstones
	Metamorphic rocks, including slates and schists (flat cleavage/foliation)
	Very marly limestones
4	Poorly cemented sandstones
	Cemented mudstones and shales
	Slates and schists (steep cleavage/foliation)
	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, dewatering 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		
Very stiff	Very stiff or hard	Brittle or very tough	kN/m <sup>2</sup> Greater than 150	kgf/cm <sup>2</sup> tonf/ft <sup>2</sup> 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  $\nu$ , 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.



**3.2.10 Chemical attack**

The possibility of damage occurring from the presence of chemicals or due to subsequential leakage of chemicals into the surrounding ground or directly onto foundations, should not be overlooked (see also section 10 and **2.2**).

**3.2.11 Ground movements**

Movements of the ground which may result from changes not connected with the loads applied by the foundations (including those mentioned in **3.2.9** and **3.2.10**) are given in **2.1.2.4**. Further information is given in **2.2** and **2.3**.

## Section 4. Deep and subaqueous foundations

### 4.1 General

This section of the code deals with factors that should be considered in the design and construction of deep foundations, i.e. foundations generally deeper than 3 m (see 3.1) including piled basement foundations.

NOTE Piled foundations are covered in detail in section 7. Cofferdams and caissons are discussed in section 5.

Circumstances requiring deep foundations are discussed in 2.3. Careful consideration should be given to possible alternative forms of shallow foundations before deciding to use deep foundations, as cost and difficulty of construction are frequently many times greater than those of shallow foundations. However, the use of deep foundations may permit much higher loadings, reduce settlements, give greater certainty of support and allow the use of additional facilities, e.g. underground car parks, thus increasing the value of the development.

Reference should be made to 2.2, to ensure that adequate site exploration is carried out.

### 4.2 Types of deep foundation

Deep foundations may be classified as follows.

- a) *Deep pad or strip footings.*
- b) *Basements or hollow boxes* (compensated foundations).
- c) *Caissons.* These may be open well caissons or pneumatic caissons.
- d) *Cylinders and piers.* These may be excavated in the dry by hand, by mechanical means or, in the wet by mechanical means including boring and slurry wall techniques.
- e) *Piles.*
- f) *Peripheral walls.* Concrete walls constructed in a slurry-filled trench or as contiguous bored piles, as well as being used as basement or retaining walls, may also carry vertical loads in conjunction with their retaining functions.
- g) *Mixed foundations.* These may be a combination of any of a) to f).
- h) *Ground improvement or replacement.* Where the ground does not have adequate bearing characteristics and stability, consideration may be given to general or local improvement of the bearing characteristics or to replacement of the ground in depth. It may then be possible to use a shallow foundation or a cheaper type of deep foundation.

The choice of foundation type in many instances will depend on a combination of technical, programme and economic factors. Sometimes, particularly in building works, the foundation work is sublet to a specialist contractor to avoid complication below ground in the building contractor's work such as might arise in a deep excavation.

Such proposals should not prevent consideration of alternative ways of founding heavily loaded structures, such as the employment of a compensated raft foundation in a deep excavation, which may require the employment of a suitably experienced main building contractor.

### 4.3 Choice of type of deep foundation

#### 4.3.1 General

Given predicted satisfactory performance of the type of deep foundation selected, economy, contract programme and possible use of basement space are then the only criteria for the selection of one type rather than another.

One of the commonest forms of deep foundation is the conventional basement, constructed either in open excavation or in a supported excavation, depending on the nature of the ground, groundwater conditions and the proximity of other structures or services. A basement foundation may be selected because ground having better bearing and settlement characteristics can be reached or because of the advantage of the extra space gained.

In general, where a deep foundation is unavoidable, a large number of fairly small but dispersed loads can most economically be carried by piles, either prefabricated or cast in situ. Heavy concentrated loads may better be carried on deep footings, piers, cylinders, large bored piles or caissons. The larger the load, the greater the expense will be in achieving the appropriate degree of certainty in the performance of the foundation.

#### 4.3.2 Deep pad or strip foundations

These are the simplest of all deep foundations and are generally used for carrying heavy column loads, the load being transmitted to the footing by means of a concrete pedestal or directly by the column; such foundations may be of various shapes, including circular, square, rectangular or strip, the shape being adjusted in the design to accommodate the effect of eccentricity arising from imposed moments and shear forces on the column and the method of construction. They are generally used where groundwater problems are unlikely to arise or where the groundwater if present can be readily controlled.



### 4.3.3 Basement or hollow boxes

**4.3.3.1 General.** Where the groundwater table is at or below formation level or can be lowered economically for construction, then a basement or hollow box foundation may be constructed in open excavation if the site is large enough. The techniques are then very similar to those for building above ground level, except that special attention will have to be paid to water-proofing when a watertight structure is required (see CP 102) and allowance will need to be made for the lateral earth pressure and any negative skin friction which may develop after backfilling. The effect of possible fluctuations in the groundwater table and of flooding during and after construction should be considered.

Where there is insufficient room for open slopes or where the groundwater table is inconveniently high, some form of water retaining structure such as a cofferdam or diaphragm wall will be necessary.

It should be borne in mind that constructing a basement below groundwater level is a difficult undertaking so one should consider whether the required accommodation can be provided elsewhere. Basement construction can also present problems of watertightness, even when not below the water table (as defined by site investigation) since the water table can vary at a later date, and burst water mains are always possible. A basement or part basement designed to be merely damp-proof is likely to leak if there is a positive head of water, even if this is only temporary. Reference should be made to the design aspects given in 4.6.

**4.3.3.2 Compensated foundations.** In addition to the extra space gained by basement construction a further advantage is the reduction in net loading applied to the supporting ground by the removal of the excavated soil and water. Tall structures may then be built on ground which would otherwise be incapable of carrying the same load on shallow foundations without excessive settlement or failure (D'Appolonia and Lambe 1971). A foundation in which the weight of the structure and substructure, including any imposed loading, balances the total weight of the excavated soil and water such that the net applied load is zero is said to be fully compensated. When the gross load exceeds the weight of the excavated soil and water, as is generally the case, the foundation is partially compensated, the degree of compensation being expressed by the ratio of the total excavated weight to gross load. A foundation or structure in which the total excavated weight exceeds the gross load is over-compensated. Where the variation in the gross load is significant, consideration may be given to varying the degree of compensation so as to achieve a more uniform distribution of net loading.

The problems of heave and swell of clay soils have to be carefully considered as well as buoyancy at times of possible high groundwater levels, particularly in the case of over-compensated foundations such as empty submerged tanks, swimming baths and buried garages.

The depth of the basement is economically limited by the stability of the excavation and the question of groundwater control (D'Appolonia and Lambe 1971). Considerably deeper excavations are easier in stronger soils than in weaker soils, but even in the former case support measures are necessary for the walls of deep excavations (see section 5). Deep foundations can be constructed in soft soils, for example by sinking cellular units in which the soil is removed from within the cells until the foundation sinks to the required depth; but even here the depth is limited by the question of base stability and the inflow of ground, particularly into the perimeter cells.

**NOTE** The term "compensated" should be preferably used rather than the more colloquial "buoyant" or "floating foundations".

Many foundations of this type below a high groundwater level will become buoyant when partially completed if the groundwater is allowed to rise too early in the construction programme. Relief wells and/or bleeder holes may be introduced into the bottom slab in order to prevent uplift when the groundwater level is not reduced externally. Alternatively, the bottom slab may be loaded temporarily with soil or building materials. Where site conditions permit and the ground at formation level provides adequate bearing for constructional operations, and where use can be made of the extra space, the partially compensated foundation may show some economy over other forms of deep foundation. However, where methods of construction such as the sinking of cellular units are necessary, these foundations are rarely economic unless the depth to an adequate bearing stratum is great.

A further advantage of compensated foundations lies in the possibility of reduction in the long-term settlement in compressible soils; the greater the degree of compensation, the smaller the long-term settlement. It should be recognized, however, that with increasing compensation the problems of heave, swell, external ground movement and groundwater control become rapidly more severe. If there is appreciable heave on excavation, the settlement may be large even if the net ground movement is small (Burland et al. 1977).

#### 4.3.4 Caissons

Caissons are structural elements of a foundation which are wholly or partially constructed at a higher level (sometimes in a different position as well) and are then sunk to their final position by various expedients (see section 5). They are used when the final foundation level is at some depth below the water table. They may consist of a single cell or be subdivided into a number of cells.

Open well caissons (monoliths) are sunk to their final position by excavating or dredging material from within the cells. Pneumatic caissons are provided with an airtight deck and airlock; they are sunk by excavation from within, under compressed air which is kept at sufficient pressure to exclude water. Physiological restrictions prevent the use of pneumatic caissons to much more than 25 m below water level. Open well caissons have no such restrictions and have been used to 80 m below water level.

Caissons are frequently used for bridge piers, particularly where the foundation needs to be some depth below sea or river bed level to avoid the effects of scour at flood times. Caissons are sometimes sunk through artificial sand islands in order to make possible the construction of the caisson from a working level above water level; they have also been used to form the foundations for buildings. Owing to the high cost of labour in working under compressed air, other possible forms of foundation should be investigated carefully before a pneumatic caisson is adopted. In addition, certain hazards to health exist which have to be considered (see CIRIA Report 44).

#### 4.3.5 Cylinders and piers

Cylinders are essentially small open well caissons comprising single cells which are of circular cross section. (The term pier is sometimes used for similar foundations of circular or other shaped cross section.) The distinction between cylinders and caissons is one of size and is necessarily arbitrary. Because of their smaller size, cylinders lend themselves more readily to the use of precast elements in their construction. For example, complete precast rings may be added at the top as the cylinder is being sunk. Cylinders are sometimes constructed by the method in which the ground beneath the lower edge of the cylinder so far constructed is removed and a further ring lowered in sections is installed below it, and so on. This method is likely to be employed only in appropriate ground conditions in the dry, or where pumping, grouting or freezing is practicable. Another method is to excavate to a small depth with an unsupported face, then to place formwork, behind which concrete is placed to support the soil.

Below the water table, methods of sinking cylinders are usually similar to those used for open well caissons. Sinking cylinders (or large bored piles) by mechanical drilling or grabbing methods has largely superseded hand methods of excavation (see 7.4).

Cylinders are often filled with concrete and are sometimes reinforced.

#### 4.3.6 Piles

Piled foundations are described in section 7. They include large bored piles which normally differ in scale but not in principle from bored piles of conventional size.

#### 4.3.7 Peripheral walls

The techniques for constructing diaphragm walls and walls of contiguous or secant bored piles are described in 6.5. With such walls, formed by excavating under a slurry, care is necessary to ensure that slurry is not trapped between the concrete and the base of the excavation.

Diaphragm walls are relatively slender structural members and they should be capable of withstanding earth pressure and hydrostatic pressure at each stage in the sequence of excavation and support by anchorages and by temporary and permanent shoring. At the final stages of excavation of a substructure with peripheral diaphragm walls, the walls are restrained from inward movement by the passive resistance of the soil below the base of the excavation until such time as the floor slab of the substructure is placed. The depth of penetration of the peripheral walls should be adequate to mobilize the required passive resistance of the soil and also to prevent erosion by seepage of water beneath the toe of the wall into the excavation (see 6.5.3).

Where inclined ground anchors are used to provide support against external pressures, the thickness of the wall at the toe should be sufficient to prevent excessive settlement due to forces induced by the vertical component of the anchor stress (see 6.5.3). The thickness of the wall at the toe should also be sufficient to prevent excessive settlement resulting from vertical loading applied to the crest or intermediate levels in a wall from floors or columns.

Diaphragm wall techniques are used to form loadbearing foundation structures (sometimes referred to as barrettes) which may be of rectangular, cruciform, hollow box or other plan shapes (Corbett et al. 1974).

#### 4.3.8 Mixed foundations on non-uniform sites

When site investigation indicates that ground conditions are not uniform, it is important to provide the type and size of foundations appropriate to the ground conditions existing on the site, and it should be recognized that the extent of ground variation is not always fully revealed in a site investigation. On any site further information about ground conditions becomes available during construction, and on a non-uniform site information can often make changes in foundations necessary. A typical example is a change in level of the ground-bearing stratum, revealed during foundation construction, leading to revised reinforcement requirements and problems in coping with groundwater, with consequent delays on site. A form of construction which can easily be varied to suit the local ground conditions has advantages in such a case. Conversely, if the choice of foundations is of a kind that cannot be easily modified to suit variation in ground conditions, then a more sensitive site investigation would be appropriate. A knowledge of previous site history and site geology are helpful in providing a sense of the extent of variation of ground conditions that may be expected.

When ground conditions are not uniform, the use of more than one type of foundation could result in greater economy. For example, if an old buried river channel crosses a site where a satisfactory bearing stratum exists elsewhere near ground level, it may be economical to use shallow foundations combined with piles in the deeper part.

When using mixed foundations, special attention will need to be paid to the effect on the structure, particularly where differential settlements can occur.

If the foundation and the supporting ground are looked at as one entity, as they should be, it will be seen that any foundation may behave in a complex way. In particular, differential settlements may occur for a variety of reasons (see 2.1). However, there is generally some uncertainty as to what extent a calculated settlement relates to the actual settlement which will occur, and this uncertainty may lead to the decision to articulate or separate the superstructure into sections. Articulation of a superstructure, however, may easily prove ineffective if the forces required to initiate joint movement are excessive so that the building may be damaged without movement or damage in the joint areas. Since the cross-sectional area of a joint filler may be considerable, the force required to compress a joint filler by, for example, 50 % could amount to several tonnes and there may thus be less resistance to damaging the structure elsewhere rather than compressing the filler.

## 4.4 Ground movements within and around deep excavations

### 4.4.1 General

The change of stress, both vertical and lateral, caused by a large excavation results in progressive upward movement of the base of the excavation and a progressive downward and inward movement of the surrounding ground.

The ground movement depends upon the dimensions of the excavation, the type and properties of the ground, the existing horizontal stresses in the ground, the water pressure beneath and around the excavation, the method of excavation and the support to the perimeter, if any, and the time during which the excavation remains open. The horizontal stresses are important, particularly in heavily overconsolidated soils, since these, unless properly resisted, can result in large inward movements of the ground surrounding the excavation, thereby affecting adjacent buildings and structures. The extent of such stresses may be determined by means of such instruments as the self-boring pressuremeter (Windle and Wroth 1977).

The vertical extent of ground movement may be estimated with varying degrees of accuracy by customary soil mechanics procedures. The components of the movement outside the excavation are complex; theory may be of little use in estimating these movements, but they can be measured by surface surveying and the installation of plumbbobs or inclinometers in boreholes prior to the excavation (Cole and Burland 1972; Ward and Burland 1972; Burland and Moore 1973; Burland and Hancock 1977).

No opportunity should be lost to observe the behaviour of other excavations in the vicinity or early excavations on site, as a possible guide to the behaviour of a deep excavation. It is also of value to undertake dewatering trials where this is practicable.

In the temperate climate of the British Isles all the operations of excavation and construction will, in general, lead to a swelling and weakening of the ground from the virgin condition, and the settlement of the structure will generally be increased (Begemann 1976). In dry climates, however, there may be an improvement in the ground conditions during the excavation due to evaporation. The structure may settle less than anticipated and it may even heave owing to subsequent wetting and swelling if there has been severe drying of the ground during construction. It is not usual to allow for any local strengthening of the ground at the sides of the excavation in the event of drying because fracturing and a net weakening of the ground usually occurs.

If the excavation and its support have not been adequately designed, plastic deformation, shear failures, uplift or failures due to water pressure may occur. It is usually possible to restrict the amount of inward yielding of the sides of the excavation by means of a suitable lateral supporting system. It is not possible to prevent all movements around excavations and allowance for these movements and for their effects on adjoining property should be made in the design of the temporary and permanent works. The choice of the method of construction, support system and groundwater control should be such as to restrict these movements to tolerable limits (Burland and Hancock 1977).

#### 4.4.2 Heave, swell and uplift (May 1974; Fernandez-Renau 1976; SMFE 1976)

The upward movement of the base of the excavation may be considered as consisting of two parts:

- a) a short-term movement called heave which occurs without volume change in unfissured materials as the excavation proceeds;

NOTE This is commonly referred to as elastic or immediate movement.

- b) a long-term movement called swell, which occurs as a result of the absorption of water into the pores of the soil as the ground adjusts itself to the new stress conditions.

The theory of elasticity is sometimes used to calculate movements due to heave. An adequate factor of safety against base failure has to be used so as to prevent the onset of plasticity. In normally consolidated clays compression tests tend to overestimate the undrained strength (Bjerrum et al. 1972).

Heave movements may be calculated by the theory of elasticity provided that there is an adequate factor of safety against base failure to prevent the onset of plasticity. The most important parameter in the calculation is the Young's modulus of the various strata of soil or rock throughout the depth of ground affected by the reduction of stress. The parameter in the case of soil cannot be reliably determined in the laboratory. In anisotropic ground there will be two values of Young's modulus, vertical and horizontal, but in these cases the movement cannot be reliably assessed by the simple theory of elasticity (Hooper 1975).

Better estimates of the modulus can be made in stiff clays by properly conducted plate loading tests at various depths in the soil profile (Marsland 1971a). It should be noted, however, that the modulus on unloading is larger than that on loading. Calculations on the basis of the theory of elasticity and laboratory or plate tests will in general over-estimate the amount of heave; in practice, where heave is an important consideration it is always advisable to take account of the observations made on other sites with similar ground conditions (Serota and Jennings 1959; Ward 1961; Measor and Williams 1962).

Swell may be calculated from consideration of the change in effective stress beneath the excavation, having regard, to the change in the groundwater conditions caused by the excavation or by the dewatering system, if any. Whether the final total swell, that is the condition in which the ground will have reached equilibrium, is reached will depend upon the period over which the excavation is left open, the flow of ground water and surface water to the swelling ground and the coefficient of swelling. In fissured ground swelling will tend to be more rapid than in unfissured ground.



In uniform relatively impermeable clays, e.g. London Clay, it may be possible to reduce swelling to a low order provided excavation and construction are rapid and proper precautions are taken to prevent softening of the formation by rainwater. In stratified ground in which water has easy access to the ground beneath the excavation, swelling may be rapid even if the water pressure within the permeable layers is lowered by artificial means. In practice, little reliability can be placed upon the estimates of the amounts, and still less of the rates, of swelling and softening in the ground beneath excavations on the basis of laboratory tests on soil samples; in general, the amount of swell tends to be overestimated and the rate of swell underestimated. If swell, estimated on the basis of consolidation theory, is considered to be important then it should, along with heave, be actually measured. Few such field measurements are available. It may be advantageous to delay the construction of an over-compensated structure until an adequate proportion of the swell has occurred.

When designing excavations below the water table, whatever the nature of the ground, the questions of seepage forces and hydraulic uplift have to be considered. When it is necessary to reduce or eliminate these forces, the necessary steps should be taken by means such as those given in sections 5 and 6. The possibility of artesian pressures should always be considered and the necessary investigations carried out to determine the seat of such pressure, if any, and its head. Particular attention should be paid to uplift pressure in stratified ground in which seams, layers or beds of permeable soil or rock occur below or within less permeable ground beneath and surrounding the excavation. Except in very narrow excavations, such as shafts in good ground and where a careful study has been made of the peripheral shearing resistance of the ground, the total weight of soil and water at any level below the base of the excavation should always exceed the total water pressure at that level.

If this condition is not satisfied, uplift or fracture of the ground beneath the excavation will occur, the quantity of water entering the excavation depending upon, inter alia, the permeability and thickness of the permeable layer. Under these circumstances the question of short-term heave and long-term swell becomes of secondary importance. In excavations in granular soil the perimeter wall should be taken to a sufficient depth below the excavation to ensure that in combination with the groundwater control measures, if any, there is an adequate factor of safety against uplift or buoyancy of the soil adjacent to the wall. Seams or layers of less permeable soil within the granular mass can also cause uplift problems and should be carefully sought during the investigation.

Particular care should be observed in excavating below the water table in very fine-grained soils to prevent loss of ground into the excavation by seepage through timbering or sheet piling. (Damage and declutching of sheet piles driven through resistant strata is more common than is generally realized.)

The installation of relief wells in good time before the excavation is completed is one way of dealing with this problem; these wells can subsequently be taken through the blinding and structural floor, with or without an underdrain, to avoid uplift of the floor. As soon as sufficient building load has been applied to counteract the buoyancy, the wells may be shut off (see 5.3). The floor should be designed to resist the uplift pressure as well as the reaction from the ground. It should be appreciated that in excavations below the water table in rock masses with beds of varying permeability, uplift pressures can disrupt the floor of the excavation often with little evidence of having done so at the time, the extent of the disruption only becoming apparent during construction when large settlements unexpectedly occur. Disruption of the formation of excavations in rock can also be caused by blasting and rough handling; account should be taken of this possibility in making estimates of settlement.



#### 4.4.3 Perimeter and external ground movements

These movements are highly complex (May 1974; Burland and Hancock 1977) arising from the change in vertical pressure due to the removal of ground within the excavation, changes in lateral pressure during excavation, the application of support in relation to the excavation programme, the rigidity of the support and also from the lowering of the groundwater level within and outside the excavation. While it might be expected that the vertical movement immediately outside the excavation should be upwards due to the relief of stress, in view of the difficulties in preventing inward movement of the support in practice the ground will settle over a considerable distance around the excavation depending upon the dimensions of the excavation. In general, the settlement will decrease with the increasing distance from the excavation. Observations on a variety of braced or tied excavations with sheet pile or soldier pile supports have indicated that settlements can occur over a distance up to four times the depth of the excavation (Peck 1969). The settlements are likely to be particularly severe adjacent to excavations in soft plastic clays. If an excavation in over-consolidated clay remains open for some time and swell occurs, the settlements at the perimeter wall will start to decrease, and given time, may eventually be exceeded by the upward movement (Ward and Burland 1972). On the other hand, if the groundwater level is appreciably lowered outside the excavation, the increase in effective weight of the over-burden will cause additional settlements in the area surrounding the excavation due to the consolidation of underlying soils. In the event of serious problems arising from this cause it will be necessary to consider other means of controlling or excluding the groundwater or, where conditions and circumstances permit, to maintain the groundwater line by recharge wells located outside the excavation.

If the ground is excavated in such a way that inward movement of the walls of the excavation occurs, as is generally the case, then the surrounding ground will also tend to move towards the excavation and the effect of the movement on any adjacent structure, whether above or below ground, will be aggravated. The tensile strains set up in buildings by these movements can be particularly damaging (Burland et al. 1977). Such horizontal movements can be particularly severe and, where restricted by stiff bracing, can result in the development of high horizontal pressures.

Where the excavation is in the form of an open cut with sloping sides, the lateral forces on the foundation will be largely determined by the properties and method of placement of the backfilling. Backfilling that is heavily compacted or likely to swell may exert a considerable lateral pressure on the foundation. In this context a reasonably cautious view should be taken as to the extent that the actual pressures on the structure are likely to be limited to those pressures assumed in design.

The magnitude of the lateral earth pressure exerted on the vertical walls of foundations constructed in sheet pile cofferdams, by means of shaft or caisson sinking and concrete walls cast in mud-filled trenches, may initially be less than the “at rest” value on account of the lateral yielding that takes place during the stages of temporary support. In the course of time it is likely to increase. In granular soils the increase will not be serious and the earth pressure is unlikely to exceed the “at rest” value. In over-consolidated clays there is considerable uncertainty about the long-term magnitude of lateral pressure, which may be greater than the over-burden pressure (Gould 1970). In addition, unless special measures are taken to introduce complete and permanent drainage of the ground outside the foundation, the lateral pressure for which the walls should be designed should include the full water pressure in the ground if there is any possibility that this might be developed.

With propped, strutted, braced or tied excavations horizontal movements should always be expected depending upon the properties of the ground, the history of the site, the dimensions of the excavation, the stiffness of the struts and the installation and excavation procedures. Particular care should be given to the design of the support and the workmanship. Ground anchors (ties) may not be so effective as strutted and braced excavations in reducing horizontal movements or settlements, particularly where they are generally anchored in ground which is going to yield significantly as a result of the excavation. If the anchors are very long they may extend the area of influence of the movement. Their chief benefit is to allow unrestricted access to the excavation.

#### 4.4.4 Reduction of ground movements

While ground movements around deep excavations can never be entirely eliminated, they can be controlled and reduced by careful investigation, study, design, planning and execution of the excavation and construction. Softening of the formation can be minimized by rapid final excavation to the required level with immediate concrete blinding of the exposed areas. The quality and thickness of this concrete needs to be adequate for its purpose. This procedure can be taken a stage further if necessary by completing the excavation and substructure in small areas when the amount of heave will also be reduced (Peck 1969). A further development could involve loading the small areas with spoil or kentledge, such as precast concrete units, which may then be subsequently removed as the above-ground building progresses. The full development of this procedure is found in the trench method where, following the installation of perimeter diaphragm walls, the structural beams, foundations and columns are installed in trenches and shafts and the ground excavated after the above-ground weight of the building exceeds the weight of the remaining soil. These further developments can be costly and consideration should be given to the alternative possibility of repairing any damage to adjacent buildings and structures resulting from the method of construction.

Contiguous or closely adjacent structures that are likely to be affected by the excavation should be underpinned, or, if ground conditions permit, the ground supporting the foundation may be consolidated where suitable by grouting (Tomlinson 1980).

Where swelling and softening at foundation level are aggravated by water, freezing and thawing and traffic, any ground that has been weakened in this way should be removed before any concrete is placed. The use of a thin blinding layer of concrete, such as is frequently used to prevent local surface puddling of the ground by light traffic, will not necessarily prevent swelling.

Where the foundation soil consists of over-consolidated clays, clay shales and certain argillaceous rocks, swelling accompanied by absorption of water and softening takes place in the unloaded soil at the base of the excavation. The process is quite-rapid at the excavated surface and proceeds progressively in depth as time elapses. The amount of swelling can be reduced by excavating quickly and reloading the ground without delay. In a large excavation this is most easily done by excavating the final 2 m to 3 m of ground in small areas and replacing it with a substantial thickness of concrete (which may form part or all of the foundation floor structure) and by subsequently imposing additional vertical loadings as soon as possible. In the case of clays with a low degree of saturation, swelling and softening will take place very rapidly when surface water is admitted to the foundation area (Jennings and Kerrich 1962; Peterson and Peters 1963; Blight 1965).

Long-term swelling forces and uplift forces due to water pressure can also be resisted by long piles which may be under-reamed and which are designed to act in tension during the early stages of construction (as in the control of movements of structures on expansive clays). With over-compensated structures ground anchors, fully and permanently protected against corrosion, may be used in place of piles (Fernandez-Renau 1976) provided the conditions for anchoring are suitable at depth. The foundation between the piles or anchors should be designed having regard to the local upward loading.

A method that reduces the inward horizontal movement of the perimeter walls, particularly at the ground surface, but does not necessarily reduce the heave, involves the construction of the floors (supported on piles) successively downwards with the removal of the spoil beneath the floors as they are completed. It is necessary here to design the basement floor to accept either the subsequent swelling and uplift pressures or to be free to move independently of the walls and columns, a solution that can lead to problems in meeting any watertightness requirements in basements beneath the water table. Alternatively, the floor may be suspended off the walls and columns and left free of the ground with a generous allowance for future swell.

In soft ground, particularly if sufficiently clayey to prevent the ingress of free water, a considerable degree of control of external movements can be achieved by sinking caissons (see section 5) provided inflow of the ground can be avoided. Downdrag of the ground immediately surrounding the caisson may be prevented by the use of an oversize cutting shoe, the gap between the structure and the ground being maintained with bentonite mud, which can later be replaced with cement grout.

## 4.5 Design considerations

### 4.5.1 General

Factors to be considered in the design of foundations generally have been outlined in 2.2 to 2.4. These considerations concerning the supporting ground, the structure and the constructional procedure should be studied and taken into account where they apply to deep foundations. It should be emphasized, however, that in deep foundations the effect of the excavation on the soil properties will be far more severe than is the case with shallow foundations and has to be allowed for in considerations of stability, bearing capacity and settlements.

It is of the greatest importance in any contract that the responsibilities of engineer and contractor with regard to design are clearly defined.

### 4.5.2 Presumed bearing values

The values recommended in section 2 for preliminary design purposes may be taken to apply to deep foundations, but allowance has to be made for any likely deterioration in the condition of the supporting ground as a result of deep excavation and allowance may be made for the greater depth.

### 4.5.3 Ultimate bearing capacity

The ultimate bearing capacity can be estimated by customary soil mechanics procedures following tests on undisturbed samples of cohesive soils or from in situ tests in boreholes or test pits. In granular soils undisturbed samples cannot be recovered except by using costly special procedures and it is usual to resort to deep soundings and standard penetration tests. The results of such tests cannot be used directly in the assessment of the ultimate bearing capacity but provide data from which the appropriate bearing capacity factors can be estimated (Tomlinson 1980).

In assessing the bearing capacity of deep basements two cases arise:

- a) the stability of the basement as a whole when the net pressure having regard to the level of the surrounding ground surface and groundwater table is used; and
- b) the stability of isolated column and wall footings where the net pressure has to be related to the basement floor level.

### 4.5.4 Allowable bearing pressure and settlement

The allowable bearing pressure has to be assessed in the light of the limiting acceptable settlements of the structure under consideration. Frequently it is possible with rigid structures to satisfy this criterion empirically by means of an appropriate factor of safety applied to the calculated ultimate bearing capacity, the factor increasing with the size of the foundation. In the design of major structures or structures particularly sensitive to settlement and differential settlement, settlement analysis may be carried out to assess the limiting bearing pressure which can be considered without imposing greater settlements and differential settlements than the structure can safely withstand (Ward 1961). The pressure distribution under all structures, other than “completely flexible” structures, will change with time as the structure deforms with the settling ground. In these circumstances where a fuller study is required, it will be necessary to consider the structure and supporting ground as an interacting system using the method of finite element analysis or other appropriate procedures (Burland et al. 1977). Solutions have been published for straightforward raft foundations of varying degrees of rigidity which may be used for guidance (Poulos 1968a; Brown 1969(a) and (b); Brown and Gibson 1972).

For the structure itself, it may be possible to define settlement limitations fairly rigorously, at least for design purposes, but it is not so straightforward with the soil. It is important, therefore, that the mutual response of the soil and the structure being supported, considered as a combined system, are fully understood in principle before any detailed-analysis is carried out.

Advanced computer techniques, such as finite-element methods with variable elastic moduli, can yield good correlation of theoretical with observed movements. Some rationalization of the soil properties used as input data is inevitable with such methods, though; lack of uniformity in the actual soil may mean that high precision in estimates of soil movements may not be justified, either in terms of cost or of true accuracy.

The limitations of either simple or complex settlement calculations should be borne in mind and it should be appreciated that experience and judgement in this matter are overriding factors (Burland et al. 1977). It should also be noted that no structure can be said to be completely rigid or completely flexible. In particular, reference should be made to Lambe (1973).

Generally, calculations of the load applied to the ground may follow the recommendations given in 2.3. It should be borne in mind that the flexure of the structure and the consequent pressure distribution is not affected by legal load requirements but by the actual load transmitted to the ground over a period of time.

Allowance should be made for the upward water pressure, which can be high. The gross weight of the structure should exceed the water uplift if flotation is to be avoided; this can give rise to problems with underground structures, such as car parks, dry docks and swimming baths, constructed on sites with high groundwater tables where special measures, such as weighting, anchoring or permanent underdrainage, are required. The effect of a fluctuating pressure on the supporting ground, whether due to variations in live loading (e.g. silos and dry docks) or in the groundwater level or to high wind loading, should be considered as many types of ground are susceptible to pressure changes of this sort, continuing to settle over periods and to extents exceeding those under static loading conditions (Bjerrum 1968).

#### 4.5.5 Sharing of vertical load between the sides and base of a deep foundation

Where the depth/width ratio of a deep foundation is large and the ground conditions are suitable, a degree of perimeter support will be offered to the sides of the foundation provided that the foundation has been cast directly against the wall of the excavation, excessive disturbance has not been caused to the surrounding ground by the excavation and the excavation has not, if in material likely to deteriorate, been left open for an unduly long time. No allowance should be made for side support in:

- a) loose backfill;
- b) in clays which may crack on drying;
- c) where scour is possible;
- d) in foundations in soft clay or fill;
- e) around the pedestal of deep footings even if the backfill is well compacted, although side support around the base may be included;
- f) where the supporting ground beneath the base is appreciably stronger than the surrounding ground; and

g) where the walls of the substructure or footing diverge downward with increasing depth, however slightly.

Where the excavation is made with vertical sides supported temporarily with walls, such as sheet pile cofferdams, cylindrical shafts, caissons, concrete walls cast in slurryfilled trenches which either become part of the final foundation or may be removed (as in the case of sheet piles), the skin friction between the ground and the sides of the foundation or may be removed (as in the case of sheet piles). Generally, the disturbance due to the construction operations will lead to a value of skin friction for the foundation as a whole considerably less than the undisturbed shear strength of the soil. Where liquids such as water or clay suspensions have been used to facilitate the sinking of shafts or caissons, the skin friction may be quite small. In some cases, e.g. in sinking caissons, it is possible to measure the average skin friction present during construction. For peripheral walls constructed under bentonite, there is some evidence to suggest that appreciable skin friction may be mobilized (Fleming and Sliwinski 1977).

#### 4.5.6 Effect of settling ground and downdrag forces

On sites underlain by recent or lightly over-consolidated clays, silt, peat or fill, especially where filling has recently been placed on the earlier surface, or where the groundwater is substantially lowered, the ground will settle around a deep foundation including piles. This may give rise to a downward drag on the sides of the foundation. If the sides converge downwards, it is unlikely that there will be any drag. The drag force should be added to the net additional vertical load applied to the base of the deep foundation in the assessment of allowable bearing pressure and settlement. In the case of piles and piers it will be necessary to allow for the effect of the reduced overburden pressure caused by the downdrag in the bearing capacity of the foundation. Downdrag can also occur where the groundwater level is substantially lowered or where backfill is placed around the foundation.



#### 4.5.7 Effect of unbalanced excavation: tilt

It is important, particularly with compensated and partially compensated foundations, to consider the tilting effects that can occur if the substructure applies an asymmetrical load to the ground, e.g. when a heavy, tall tower is located to one side of a hollow box foundation. This requirement is of particular importance on saturated and unsaturated clays where swelling is likely to occur. Tilt can also be caused by the erection or demolition of adjacent heavily loaded structures or by adjacent excavation. A frequent cause of tilt is variability in the ground conditions not detected in the site investigation and not taken account of in design because of undue extrapolation from limited borehole information.

#### 4.5.8 Distribution of load in piled basement foundations

At present this may be considered as a developing subject and the literature should be studied with this in mind. An understanding of soil structure interaction is necessary for design, and it is also necessary to have some appreciation of the consequences of having made the wrong assumptions.

Significant heave and possibly swell (see 4.5.2) are likely to take place in stiff over-consolidated clays, e.g. London Clay, if the excavation is more than 5 m deep. If heave and swell are ignored where the piles are designed to take the full load, there is a danger that the raft may be inadequately designed.

Piles installed before excavation begins are likely to be subject to tension. The same problems can occur particularly in areas of expansive clay such as those occurring in many tropical regions.

The proportion of total load carried by the basement and the piles varies with time and changes in the construction programme can substantially affect design assumptions. Cases have been recorded in London where the piles took about two-thirds of the load and the raft one-third. It is best, however, that each case is treated on its own merits (Butterfield and Banerjee 1971a and b; Hooper 1979).

#### 4.5.9 Basement watertightness

It should be assumed at the design stage that despite care a basement may leak or fail to provide the required degree of watertightness, either immediately after construction or at a later date. The work should be designed, planned and constructed to keep this risk to the economic minimum.

During the design stage consideration should be given to the consequences of any leakage or damp patches, the scope for repairs, the form of remedial works, and the possibility of testing during construction (such as by temporarily raising the water table by stopping pumping). If the basement is to be tested in this way, it will probably be necessary to seal any holes or shafts in the slab above the basement against rain and to seal ducts through the walls against groundwater. The designer may balance the costs of such testing against the benefits of having more time available to effect repairs to the basement (if tests prove repairs necessary) and thus possibly avoid delaying completion of the project as a whole while the basement is repaired.

An effective system of ground drainage such as by cut-off or interceptor drains can be an important factor in obtaining a dry basement.

The degree of watertightness of the basement should be adequate to prevent damage to the structure by aggressive groundwater. In addition, if the space formed by the basement is to be used, the basement should be sufficiently watertight and free from condensation for the intended use. In this connection damp caused by condensation within the basement may be just as great a nuisance as percolation from outside, and the designer should draw to the attention of those dealing with the problems, the necessity of providing adequate ventilation and/or heating and suitable treatment of ground and wall surfaces.

The designer should devote special attention to the investigation of the groundwater regime outside the basement, and this should be considered in the light of the proposed use of the basement and the consequential requirements for the basement. This is essential for assessing the most economical performance requirement for watertightness. The performance requirements for basement watertightness fall into the grades given in Table 7. Having considered the grade of watertightness which is suitable for the proposed basement use, it may on occasion be advisable to reconsider with the building owner the proposed basement use in respect of basement dryness, and to explain the possibility of basement leakage after completion. It is likely to be worthwhile considering the issues referred to in ISE (1975) and "Guide to the design of waterproof basements"<sup>1)</sup> published by CIRIA.

<sup>1)</sup> Available from CIRIA, 6 Storey's Gate, London SW1P 3AU.



After discussion with the building owner the basement performance requirement for watertightness should be established, so as to avoid unnecessarily expensive construction on the one hand or unsatisfactory performance on the other.

The choice of construction method can affect the degree of watertightness which it is practical to achieve, and the choice of construction method should thus be related to the specified performance requirement for watertightness. The liability for remedial works may affect the way in which work is carried out, itself a matter subject to economic consideration. The responsibility for the consequences of leakage and for remedial works should as far as possible be clearly defined in the contract.

In broad terms the contract may be for the supply of specified materials and labour only, with implied or stated standards of workmanship, or it may include, in addition, a performance requirement such as a specified degree of watertightness.

The inclusion of a performance requirement may necessitate adjustments to standard forms of contract. Careful consideration should be given in the early stages to choice by the contractor of construction methods, required standards of supervision and workmanship and the probable cost of repairs, since these all have a bearing on design and specification and on the actual cost of construction.

Although the risk of increased cost to the contractor in complying with a performance requirement may be reflected in the tenders, this cost may be more than balanced by clarification of responsibilities and hence avoidance of disputes, better use of the contractor's skill in providing what is required, closer identification of the aims of the client with those of the contractor and the avoidance of extra cost to the client of any necessary remedial measures.

The performance requirements for watertightness should be specified and the details of the method of achievement stated.

**Table 7 — Grades, performance, use and possible construction for basements**

Grade	Performance	Possible use depending on circumstances	Possible construction	Comments
Utility	Some seepage and damp patches can be tolerated provided water is drained away. It may be necessary to quantify allowable seepage as a design requirement. Seepage may not be acceptable when the water contains chemicals that may have a deleterious effect on the structure	Car parking, mechanical and electrical equipment, provided plant would not be damaged by repairs to basement for leaks and that such repairs would not cause undue inconvenience	Reinforced concrete (r.c.) floor slab, r.c. walls or in some circumstances, brick walls. Both r.c. walls and brick wall with external bituminous paint desirable where access allows. It may be sufficient in some circumstances merely to design the slabs to be dampproof	Means of draining any ingress of water may need to be allowed. This grade may not be suitable for all electrical equipment. Cars can be damaged by roof drips. The design and use of utility grade basements should be such that any repairs for watertightness can be carried out without damage to plant or undue inconvenience
Workshop	No visible penetration of water is acceptable and condensation may not form other than in exceptional circumstances. The design and use of workshop grade basements should be such that the consequences and inconveniences are acceptable for any repairs for watertightness which may at any time be necessary	Workshops and circumstances where a drier environment than utility grade is required but where it is accepted that some repairs may be necessary after construction is complete	Broadly similar to utility grade but standards are higher so it may be necessary to raise either the construction standard, and/or workmanship, and/or standards of repair	It may be very inconvenient to make space available for repairs and this could necessitate a higher performance requirement. Reinforced concrete is not vapour proof but membranes can be. External membranes offer the possibility of improving the watertightness but the source of leaks through an external membrane cannot usually be found
Residential	The standard of watertightness is the same as for workshop grade but the basement is designed so as to make evacuation of the premises to carry out repairs unlikely	Residential, commercial or industrial where evacuation to carry out repairs would be very undesirable	Drained cavity wall construction with suitable wall and floor treatment	Reinforced concrete is not vapour-proof but membranes can be. External membranes offer the possibility of improving the watertightness, but the source of leaks through an external membrane cannot usually be found
Special	No visible water penetration or vapour to cause dampness and no condensation whatever are allowed	Archives and stores	The basement is designed and constructed so that later repairs are not likely to be required. Unless the site is permanently dry a vapour barrier will be necessary which may be in the form of an external membrane. Drained cavity wall construction with suitable wall and floor treatment, with provision for pumping cavity, within a reinforced concrete basement. Ventilation and air conditioning are likely to be essential	A perfect external membrane is very desirable. Reinforced concrete is not vapour proof, but membranes can be. External membranes offer the possibility of improving the watertightness but the source of leaks through an external membrane cannot usually be found

## Section 5. Cofferdams and caissons

### 5.1 General

#### 5.1.1 Introduction

The provisions of this section are intended to apply to cofferdams and caissons used for constructing works below water levels in waterways or in waterlogged or unstable ground which necessitates lateral support. Their function is to provide a space down to foundation level from which water is excluded sufficiently to permit the descent of workmen, plant, etc. the removal of spoil and the execution of the permanent work. In some instances work may be carried out under water.

#### 5.1.2 Preliminary investigations

An investigation of the ground and other local conditions should be made in accordance with BS 5930 before selecting the type of cofferdam or caisson. Borings should be taken and samples of the soil should be tested with regard both to the completed structure and to the temporary work. The permeability and strength of the soil and the water levels in it should be considered, to determine the depth to which cofferdam sheeting should be driven to ensure stability, to cut off any excessive flow of water into the cofferdam and to eliminate the danger of a boil. In rivers and tidal waters, heights of tides, flood levels and current velocities should be carefully studied. In exposed conditions the effect of wave action should be considered. Account should be taken of the acceptable degree of noise and vibration in the locality.

### 5.2 Materials and stresses

#### 5.2.1 Quality

The quality of the timber, concrete, steel or other material used for the component parts of cofferdams and caissons should comply with the appropriate British Standards.

#### 5.2.2 Timber

**5.2.2.1 General.** A wide range of timbers can be used for bracing to cofferdams depending on their local availability. Guidance as to their properties can be found in BS 5268 and in "A handbook of softwoods"<sup>2)</sup> and "A handbook of hardwoods"<sup>2)</sup> published by the Building Research Establishment.

For cofferdams the quality used should be not less than SS grade as defined in BS 4978-3. Timbers of species SC3, SC4 or SC5 should be used, adequate account being taken of the stress values assigned to these species groups or to the individual species within them, as given in BS 5268.

#### 5.2.2.2 Stresses

**5.2.2.2.1 General.** As a guide the grade stresses for SS grade timber of the species groups SC3, SC4 and SC5 are given in Table 8, the values being extracted from BS 5268 to which separate reference should be made for stress values of individual species.

##### 5.2.2.2.2 Permissible stresses

**5.2.2.2.2.1 General.** The permissible (or working) stresses should be calculated by applying the relevant modification factors given in BS 5268. The design of the members and fastenings should comply with the relevant clauses in that standard.

**5.2.2.2.2.2 Slenderness ratio.** The grade stresses should be modified in accordance with BS 5268. The effective length of struts should be as recommended in 5.3.3.8.

**5.2.2.2.2.3 Compression members subject to bending.** Members subject to axial compression and bending should comply with 15.6 of BS 5268-2:1984.

**Table 8 — SS grade timber grade stresses**

Type-of stress and modulus	Strength class (see BS 5268)		
	SC3	SC4	SC5
	N/mm <sup>2</sup>	N/mm <sup>2</sup>	N/mm <sup>2</sup>
Bending parallel to grain	5.3	7.5	10.0
Tension parallel to grain	3.2	4.5	6.0
Compression parallel to grain	6.8	7.9	8.7
Compression perpendicular to grain	2.2 or 1.7	2.4 or 1.9	2.8 or 2.4
Shear parallel to grain	0.67	0.71	1.00
Modulus of elasticity-mean	8 800	9 900	10 700
Modulus of elasticity-minimum	5 800	6 600	7 100

**5.2.2.2.2.4 Factor for deterioration.** The grade stresses should be reduced by the application of a factor between 1.0 and 0.7, according to the type and condition of the timber and the amount of deterioration that may be expected during the period of use. Timber with fungal or pest attack should not be used.

#### 5.2.3 Reinforced concrete

The quality of concrete should be in accordance with BS 8110 or clauses 201 to 209 of CP 114:1969. Concrete stresses should be in accordance with BS 8110 or clause 303 of CP 114:1969.

#### 5.2.4 Steel

The steel used in the construction of cofferdams and caissons should comply with BS 4360; workmanship and stresses should be in accordance with BS 449.

<sup>2)</sup> Available from the Building Research Establishment, Building Research Station, Garston, Watford, Herts WD2 7JR.

## 5.3 Design considerations

### 5.3.1 Choice between cofferdams and caissons

The main difference between a cofferdam and a caisson is that although they are both enclosures for the purpose of excluding water and soil from a work during construction, a cofferdam may be generally a temporary structure, part or all of which is removed after construction, whereas a caisson is primarily a permanent structure or one which is subsequently incorporated in the permanent work.

The chief factors influencing the type of construction used are ground conditions and the depth to which the work is to be carried.

Where the work can be safely carried out in free air, cofferdams or open caissons may be used. Where conditions require the use of compressed air, caissons or cofferdams fitted with an air-deck should be employed.

For deep cofferdams it may be preferable not to lower the water below a certain level, the last stages of excavation, concreting and if necessary strutting being carried out through water.

### 5.3.2 Determination of pressures

The earth pressure or the combined earth and water pressure should be determined in accordance with Civil Engineering Code of Practice No. 2 and this standard. During design a review should be made with the object of identifying any adverse forces or conditions not specifically mentioned in these codes, including an effective stress analysis and provision should be made for these.

### 5.3.3 Cofferdams

**5.3.3.1 General classification.** The main types of cofferdam are shown in Figure 2, the upper part showing all types, the lower those with steel sheet piling.

#### 5.3.3.2 Selection of cofferdam

**5.3.3.2.1** In the selection of a suitable cofferdam type for a given duty the following factors should be taken into account:

- a) whether a land cofferdam or water cofferdam is required;
- b) nature of the permanent structure to be built within the cofferdam;
- c) plan dimensions of the working area required inside the cofferdam;
- d) total depth of soil and/or water to be retained;
- e) soil conditions below and above foundation level;
- f) groundwater levels and their fluctuation, and for water cofferdams the tidal, seasonal and flood levels;

g) for water cofferdams, the strength of the current, wave action and scour before, during and after construction;

h) possible effects of the cofferdam construction on existing buildings or other structures close to it;

i) availability of materials;

j) methods of constructing and dismantling the cofferdam;

k) time available for construction of the cofferdam;

l) noise, vibration, fumes and fire risk;

m) relative total costs of the cofferdams where two or more types are otherwise suitable;

n) accessibility especially for cofferdams in water.

**5.3.3.2.2** Certain of the types shown in Figure 2(a) are suitable for both land and water cofferdams, while others are applicable to one or the other only. The extent of the working area will further limit the choice. When the depth is great, the soil and water level conditions will be particularly significant. Possibilities that can be considered are:

a) combinations of cofferdam types;

b) final excavation carried out underwater;

c) special dewatering methods, etc.

For extreme depths, depending on the soil conditions, it may be necessary to resort to monolith or caisson construction.

#### 5.3.3.3 Earth and rock-fill cofferdams.

Earth and rock-fill cofferdams are enclosures formed by mounds of earth or rock. Earth-fill cofferdams may be used where the head is low, the velocity is small, there is no risk of overtopping and suitable filling material is available.

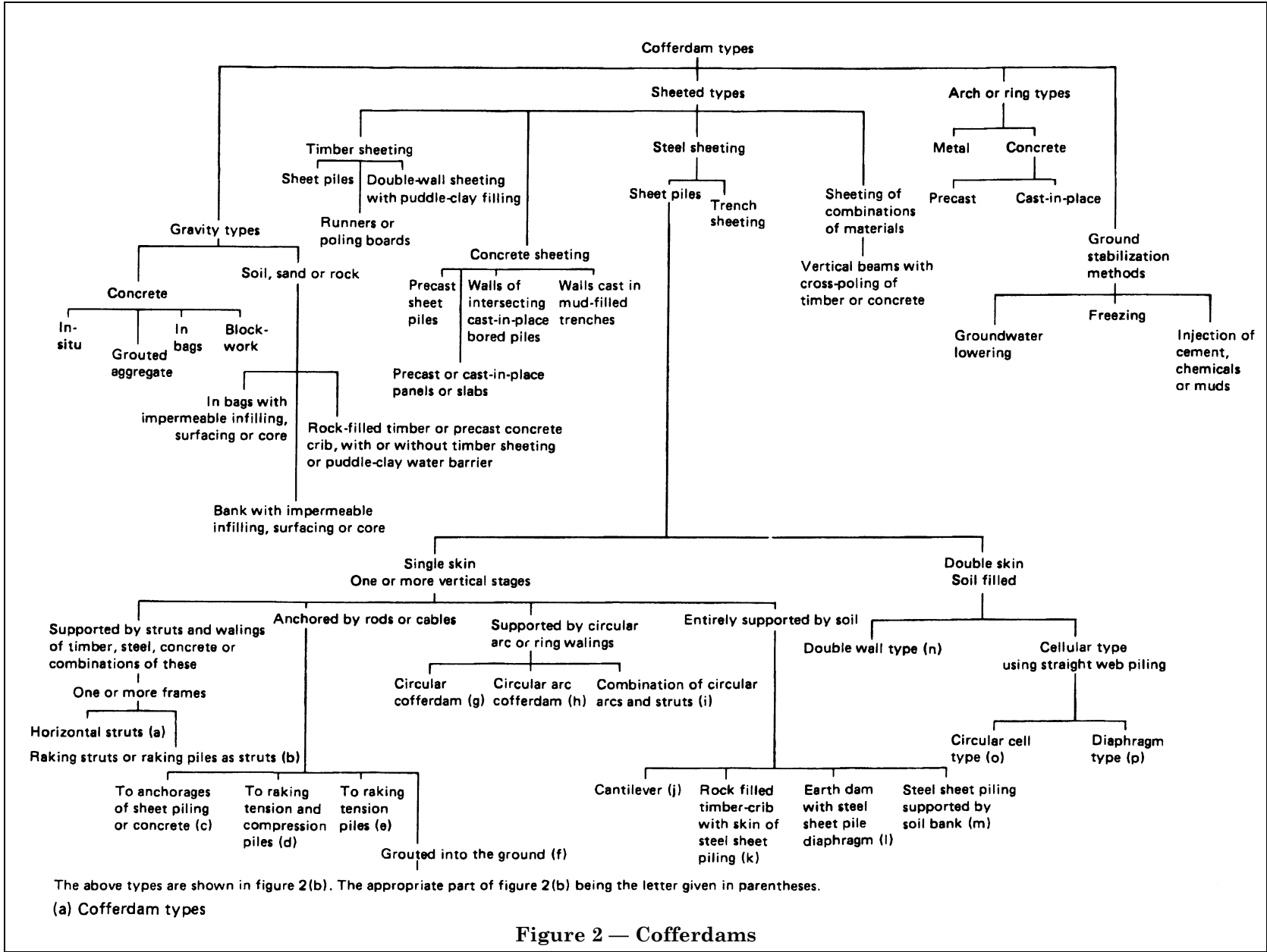
The most favourable condition for their use is where they can be constructed in the dry during low-water periods, so that controlled construction methods can be adopted. These include the compaction of the fill in layers by means of a suitable type of roller with adequate control of moisture content to ensure maximum density (see BS 6031).

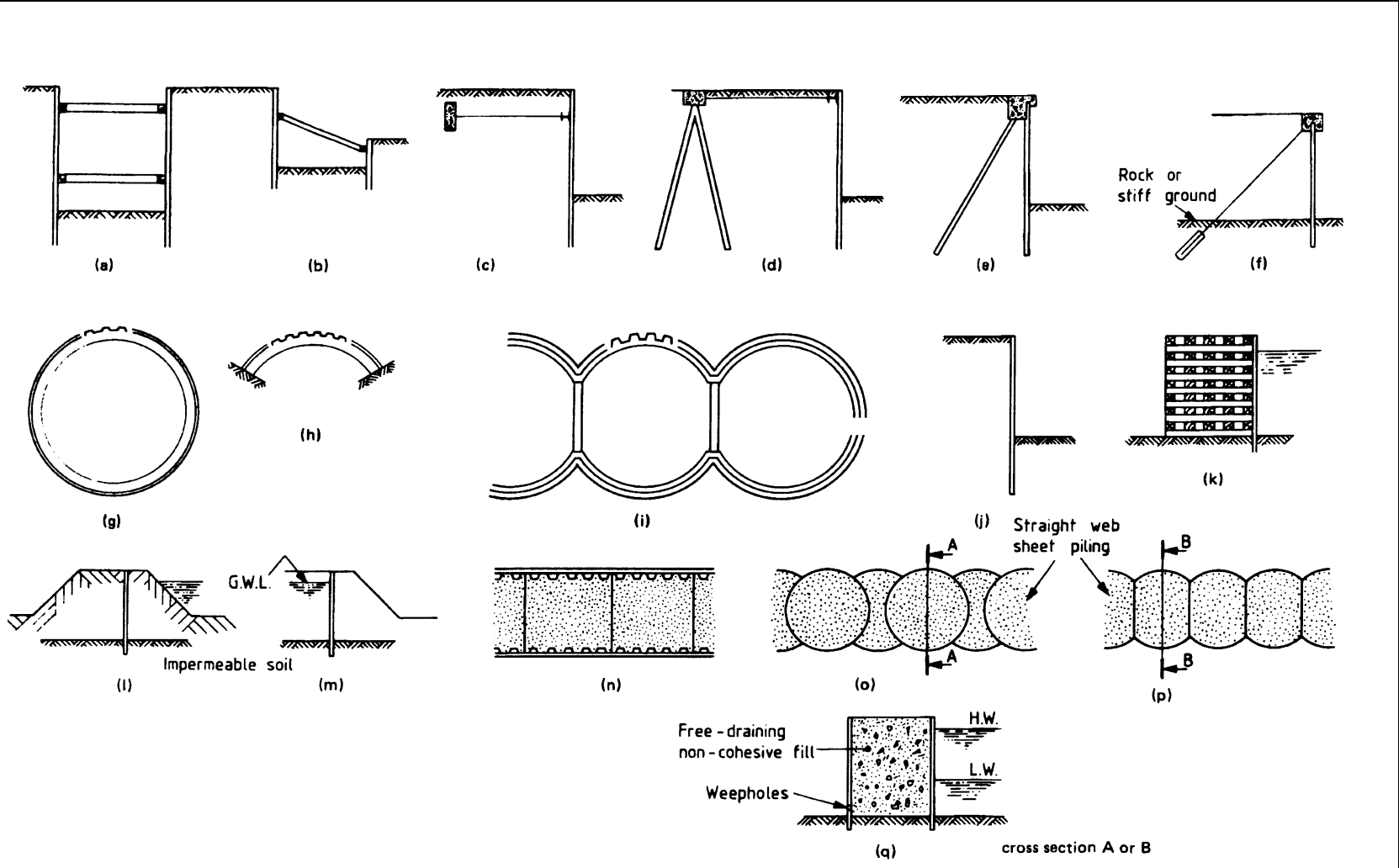
Earth embankments used as cofferdams should be designed and constructed generally in accordance with the principles used in permanent earth dams. Because of their temporary nature, less rigorous precautions against leakage or damage than for permanent earth dams may be acceptable. The upstream face may be protected by pitching, rubble, concrete or bitumenized sand facing. Resistance to erosion may be improved by placing the earth in bags of natural or synthetic fibre.

Rock-fill dams should be considered where there are swift currents, hard rock bottoms or deep water.

Rock-fill dams should be constructed mainly of massive rock or boulders. Where rock of sufficient size is not available, a quantity of smaller rocks may be enclosed in a wire-mesh net or basket, sometimes called a gabion. If the dam is constructed in the dry, its impermeability may be improved by filling the voids with smaller material.







(b) Cofferdams using sheet piling

Figure 2 — Cofferdams (concluded)

In making a closure on a deep and fast flowing river, the rock should be heavy and massive to resist erosion and should be dumped quickly over as wide a base as possible, thus maintaining the flow over as large an area as possible and at the same time reducing its depth. Under these conditions, the flow of water may be reduced, after closure of the dam, by tipping progressively smaller material as a graded filter on the upstream face.

The side slopes to which a rock-fill dam can be built depend on the size and shape of the stone, whether it is tipped or individually placed, and on the swiftness of the current. If there is a risk of overtopping, the crest and downstream face will have to be resistant to erosion, and, in extreme cases, very large stone or specially designed concrete units may be needed.

#### 5.3.3.4 Crib cofferdams

**5.3.3.4.1 Timber crib cofferdams.** A timber crib cofferdam is formed by a framework of heavy timbers, either square or round in section, the timbers being laced together in criss-cross fashion to form pockets up to 3.5 m<sup>2</sup>.

Bolts, driftbolts, spikes and timber connectors may be used to connect the members, which should also be laced together in the vertical direction.

The timber cribs may be floated into position, suitably weighted with rock for sinking and then completely filled with rock. The water face may be covered with sheet piling or timber sheeting to make it more watertight. The timber on the base of the crib may be shaped beforehand to fit the conformation of the rock bed.

**5.3.3.4.2 Precast concrete crib cofferdams.** In suitable locations cribs of precast concrete members may be made in situ on the same principles as timber cribs, using appropriate connections.

#### 5.3.3.5 Double-skin cofferdams

**5.3.3.5.1 General.** Double-skin cofferdams are self-supporting gravity structures, of either the parallel-sides double-wall cofferdam type or the cellular cofferdam type. The stability of both of these types is dependent on the properties of the filling and of the soil at foundation level, as well as on the arrangement and type of the steel sheet piling.

**5.3.3.5.2 Double-wall cofferdams.** Double-wall cofferdams consist of two parallel lines of steel sheet piling connected together by a system of steel walings and tie rods at one or more levels, the space between the lines of piling being filled with a cohesionless material such as sand, gravel or broken rock.

The inner line of piling is designed as an anchored retaining wall (see Civil Engineering Code of Practice No. 2) while the outer line of piling acts as the anchorage. Ordinary flat types of steel sheet piling are, therefore, inappropriate. The width of the dam should be not less than about 0.8 times the retained height of water or soil or both.

Weepholes, in association with filters if necessary, have to be provided near the bottom of the exposed portion of the piles on the inner side. Drainage of the filling is essential in order both to reduce the pressure on the inner line of piling and to prevent a decrease in the total shear strength of the filling. As complete drainage of the filling may not always be possible, allowance will need to be made for any water pressure acting on the piling. It is essential that clay or silt are not used as filling, and any soft soils of these types occurring above the main foundation stratum will need to be removed before placing the filling.

The piling has to be driven into the soil below excavation or dredged level to a sufficient depth to develop the necessary passive resistance. When cohesionless soils occur at and below excavation level, the penetration of the piling has to be sufficient to control the effects of seepage. The presence of rock at excavation level makes this type of cofferdam unsuitable, unless the rock is of a nature such that steel piling can be driven into it to an adequate penetration or unless tie rods can be installed at a low level. Alternatively, it may be possible, for example at low water, to excavate a trench into which the piling may be placed and concreted.

**5.3.3.5.3 Cellular cofferdams.** Cellular cofferdams are constructed in the manner shown in Figure 2(b), using straight web steel sheet piling. The piling is interlocked and driven to form cells, which are filled with cohesionless soil such as sand, gravel or broken rock. The outward pressure on the filling develops high circumferential tensile forces in the piling, which straight web piling is designed to resist. Trough-shaped sections of steel sheet piling are not suitable for cellular cofferdams because the troughs and interlocks will open out.

Cellular cofferdams are self-supporting and enable considerable heights of water and soil to be retained. The stability of the cofferdam depends on the tensile strength of the piling, the size and shape of the cells and the properties of the filling and of the foundation soil. Rock provides a suitable base for cellular cofferdams and penetration is not essential for stability, but some driving of the piling is desirable in order to assist in preventing seepage of water through the dam. With a sand or gravel foundation, penetration by the piling has to be sufficient to prevent seepage affecting the stability of the structure. Especially with sands, it may be necessary to provide a berm on the inner side of the dam. Cellular cofferdams will be satisfactory on a clay foundation only if the clay is very stiff or hard, depending on the height of the dam. Whatever the type of soil at foundation level, any overlying soft clays or silts will need to be removed from inside the cells before they are filled. Adequate drainage will need to be provided near the bottom of the piles on the inner side of the cofferdam by means of weepholes, together with filters if necessary. As complete drainage of the filling may not always be possible, allowance will need to be made for any water pressure acting on the piling.

To provide an adequate factor of safety against tilting, distortion and sliding, the ratio of the average width of the cofferdam to the retained height of water and/or soil should be not less than about 0.8 to 1.0. The ratio of the ultimate tensile strength of the interlock to the estimated maximum tension across the interlock should normally be not less than 2.0. Special care should be taken in handling the straight web piling due to its lack of bending strength when in the flat position.

#### **5.3.3.6** *Cofferdam of cast-in-situ diaphragm walls.*

A single-skin land cofferdam may be constructed by surrounding the site of the excavation by continuous cast-in-situ diaphragm walls of concrete or other materials as described in 6.5.3. The wall may be supported by bracing or by a soil bank.

#### **5.3.3.7** *Single-skin sheet pile cofferdams*

**5.3.3.7.1** *General.* The sheet piling forming the dam should be capable of being driven through the strata to the depths required. It should also be able to withstand the external earth and water pressures and to transmit them to the bracing without exceeding the safe stress in the material of the sheeting.

When designing sheet pile walls, it is essential that due regard is paid to the stresses imposed on piles during driving. Pile sections that are deficient in driving strength will suffer damage during installation and expensive delays may result.

Sheeting without any bracing should be designed as cantilevers. Sheeting with a single waling near the top may be designed as simply supported beams spanning between the waling and a point at some depth below excavation level. Alternatively, if the sheeting is driven deep enough into suitable soil, it may be designed as a vertical beam, having a fixing moment in the ground and a support at the waling.

For cofferdams with several settings of bracing, the design should allow for the sequence of construction, and each stage in dewatering and placing the frames has to be considered separately. The diagrams of active and passive soil and water pressures for each successive stage in the construction of the dam should be drawn, to determine the corresponding maximum stresses in the piling and the bracing loads. Usually, the conditions existing temporarily during construction will be more severe than when the cofferdam is completed.

The design will also need to provide for the temporary conditions which will arise, in the reverse order, when the bracing is being removed to make room for the permanent construction inside the cofferdam.

#### **5.3.3.7.2** *Section modulus of steel sheet piling.*

Sections of "Z" type steel sheet piling that have their interlocks in the flanges may be considered to develop the full section modulus of an undivided wall or piling under all conditions.

Sections of deep trough steel sheet piling with close-fitting interlocks along the centreline or neutral axis of the sheeting may be assumed to develop substantially the strength of the undivided section only when the piling is fully driven into ground, with the exception of the following.

The shear forces in the interlocks may be considered as resisted by friction due to the pressure at the walings and the restraint exercised by the ground. In certain conditions it is advisable to connect together the inner and outer piles in each pair by welding, pressing, or other means, to ensure that the interlock common to the pair can develop the necessary shear resistance. Such conditions arise when:

- a) the piling passes through very soft clay or other weak material;
- b) it is prevented by rock from penetrating to the normal depth of cut-off;
- c) where the piling is used as a cantilever or if it is cantilevered to a substantial height above the level of the highest waling;
- d) if there is backfilling placed against one side of the piling after it has been driven.

If any of these conditions arise and the pairs of piles are not connected together as described, the section modulus of individual piles should be used, and not that of a pair of piles acting as an undivided section.

**5.3.3.7.3 Effect of depth of cut-off and methods of pumping in cohesionless soils.** This subclause is concerned only with the case where the soil at and below final excavation is granular and cohesionless, where the piling does not penetrate to an impermeable stratum and, if the cofferdam is of the land type, where there is a high groundwater level. Consideration has to be given not only to the depth of cut-off required to prevent the piling moving inward under the influence of hydrostatic and soil pressures, but also to any further depth required to counteract the effect of seepage of water under the toe of the piles and upwards into the base of the excavation which, in these particular conditions, will cause effects which may endanger the stability of the cofferdam. The likelihood of overdig should also be considered and allowances made as necessary. Quite apart from the question of stability, a larger cut-off will reduce the amount of pumping capacity required to keep the cofferdam dry.

The following three cases differ in the manner of pumping.

- a) *Pumping from sumps at excavation level only.*  
Upward seepage of water:
  - 1) causes a reduction in the passive resistance of the soil below excavation level; and
  - 2) may be sufficient to cause boiling of the soil at excavation level.

Both can lead to instability.

To counter the effect of 1) the penetration of the piling and its bending strength need to be adequate, and the lowest bracing frame has to be able to carry larger loads than would occur if there were no upward seepage. Boiling may be prevented by providing an adequate cut-off (Ward 1957) as shown in Table 9 and Figure 3. Table 9 gives, as a guide, minimum values of the cut-off in uniform cohesionless soils depending on the cofferdam width and the height from outside water level (or groundwater level) to excavation level.

Where very fine uniform sands or a more permeable layer below the tips of the piles exists, a greater cut-off than shown may be required.

In the case of a water cofferdam, the tendency to boiling can be reduced if it is possible to place a less permeable layer of material as a blanket outside and all around the cofferdam right up to the sheet piling. The object here is to increase the length of the seepage paths in a manner similar to the increased path length that results if the cut-off is increased. Boiling may also be averted by sinking sufficient relief wells that extend well below the final excavation level inside the cofferdam. Relief wells are not pumped and may consist of porous or perforated pipes, or boreholes filled with suitably graded granular filter material. They provide a safe path for the upward flowing water, which can be pumped away at excavated level from sumps. If practicable, pumping sumps should be located away from the walls of the cofferdam.

**Table 9 — Minimum values for depth of cut-off for cohesionless soils where there is no significant lowering of the external water level** (see Figure 3)

Width of cofferdam, $W$	Depth of cut-off, $D$
$2H$ or more	$0.4H$
$1H$	$0.5H$
$0.5H$	$0.7H$
NOTE $H$ is the height from lowered water level to excavation level.	



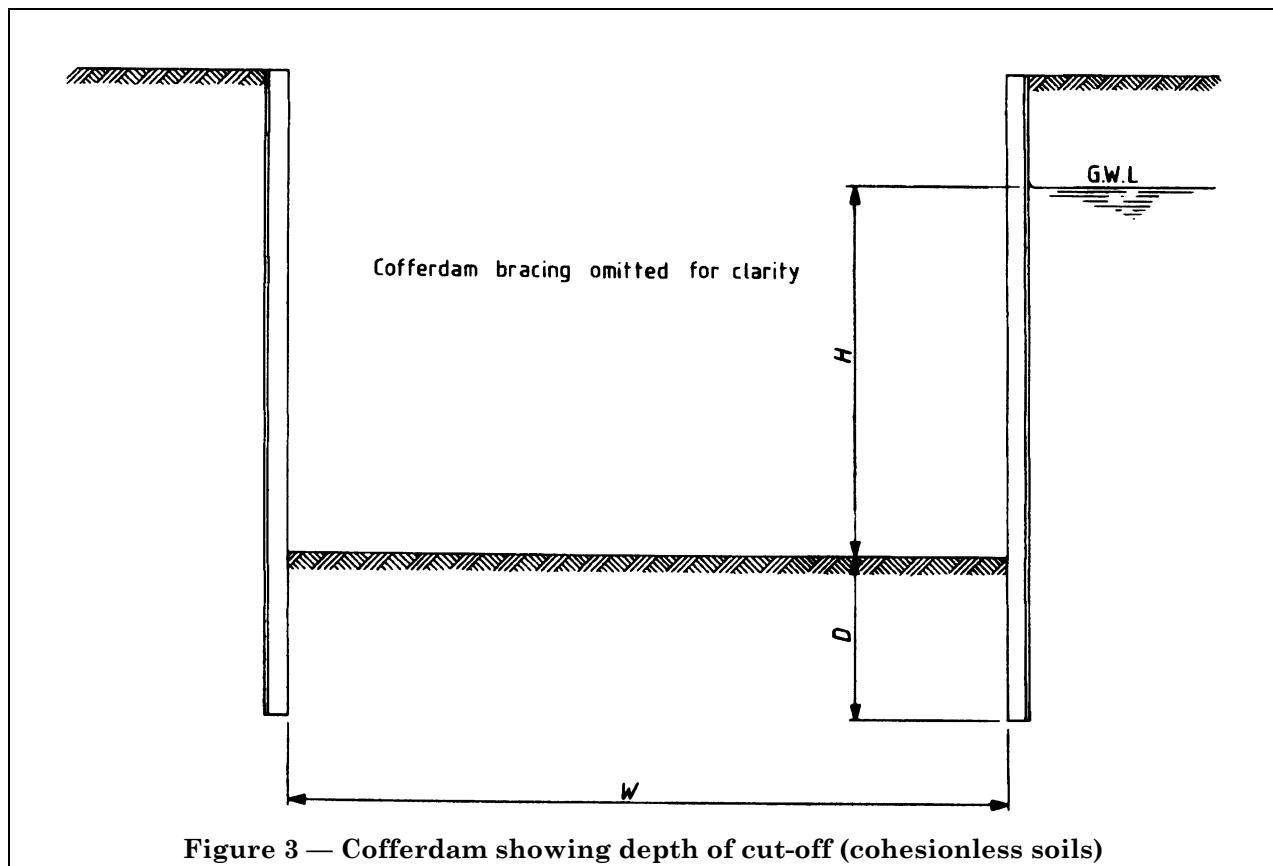


Figure 3 — Cofferdam showing depth of cut-off (cohesionless soils)

b) *Pumping from wells located inside the cofferdam.* The water level may be taken below formation level by means of adequate pumping wells inside the cofferdam, to reduce the water level to a depth sufficient to avoid boiling at excavation level. Greater passive resistance will be provided at the cofferdam walls if the groundwater level is lowered substantially below formation level, and consequently the depth of cut-off will be governed mainly by consideration of transferring external soil and water pressures to the soil inside the cofferdam below excavation level.

c) *Groundwater lowering outside the cofferdam by means of borehole pumps in deep wells or well points.* If the soil is sufficiently permeable, it may be possible to lower the groundwater level to below excavation level by means of wells outside the cofferdam. In such cases very little or no pumping may be necessary within the cofferdam. In addition, effects a) 1) and a) 2) are eliminated, and cut-off should be determined as in b). If the excavation level is below the lowered groundwater level, then when applying Table 9,  $H$  is the height from lowered water level to excavation level.

In uniform sand the vibration caused by driving piles from excavation level may help to induce boiling if pumping is being carried out from sumps. In this case, the method in b) or c) should be adopted, or all the piles should be driven before excavation.

The effects in a) and b) can also be eliminated by carrying out final excavation under water and depositing a sufficiently thick concrete base plug underwater to ensure stability at the bottom before dewatering the cofferdam (see 8.2). This procedure will also have the effect of decreasing the number and size of loads on the bracing frames.

#### 5.3.3.7.4 Instability of the base, other than boiling

**5.3.3.7.4.1 Lifting of a clay stratum.** If the cofferdam contains a clay or silt stratum having under it a permeable stratum under a high water head, the water pressure will tend to lift the clay or silt stratum and any soil over it at some stage of the excavation. The total pressure is resisted by the weight of soil within the confines of the cofferdam. The friction or adhesion between the soil and the sheet piling should be taken into account only if there is no danger of the plug being broken up or deformed. If the margin of safety, is in doubt, it will be necessary:

- a) to take out the final excavation in narrow widths transverse to the cofferdam and replace by concrete; or
- b) to employ relief wells within the cofferdam extending into the lower permeable stratum (Ward 1957); or
- c) to employ borehole pumps in deep wells in the same manner; or
- d) to carry out final excavation under water and then deposit an adequate base plug of concrete to ensure stability at the bottom under water before dewatering.

**5.3.3.7.4.2 Upward movement of soft clay at the base.** If the sheet piling does not penetrate to any hard stratum and soft clay or silt exists at and below excavation level, it may be forced upwards by the pressure of the surrounding ground (Cooling 1948; Skempton 1951). Important factors in this connection are the shear strength of the soil and the excavation depth.

**5.3.3.7.4.3 Loss of strength in soft clay or silt.** If a soft, sensitive clay or silt stratum exists at and below excavation level, the driving of piles into or adjacent to it will cause loss of strength owing to remoulding, and this effect should be allowed for in the design.

#### **5.3.3.8 Design of bracing**

**5.3.3.8.1 General.** Cofferdams may be braced by one or more settings of walings and struts which may be of timber, steel, reinforced concrete or a combination of these materials.

In cofferdams where prefabricated bracing frames are to be used, it is essential to fabricate the frames smaller than the theoretical internal dimensions of the cofferdam to allow for the deflection of sheet piling as excavation proceeds and for the possibility of the piling not being absolutely vertical.

The struts should be secured to the walings by cleats, packings, stretchers or liners, preferably with bolts. The weight of the bracing should be supported by the sheeting or, in suitable cases, by the ground.

In cofferdams in water, tie-rods across the dam should be provided immediately above the top waling and should preferably be passed through a waling fixed to the outside of the dam. Diagonal bracing, in the horizontal and vertical planes, should be provided to ensure the rigidity of the cofferdam as a whole.

Allowance should be made in the design of the bracing for any unbalanced or excess pressures which may be imposed on any side of a cofferdam. These should be dealt with by making provision for side and end support, e.g. by the tie-rods fixed to independent anchorages, internal or external raking struts or other effective means.

If the bracing is to be used to support equipment or materials, provision should be made in the design.

Temporary conditions during construction of a cofferdam will usually produce heavier loads in the bracing than will occur in the completed cofferdam. This will need to be taken into account in the design of walings, struts and other members, and their connections.

**5.3.3.8.2 Cofferdams in built-up areas.** In the construction of land cofferdams in built-up areas, and in other special circumstances, it is often necessary to reduce as far as possible the movements of the adjoining ground. For this purpose the walls of the cofferdam should be stiff and continuous, and should be constructed in the ground before the main excavation is made. The bracing frames should be stiff and inserted over the whole area of the cofferdam as quickly as possible at frequent intervals of depth as the cofferdam excavation proceeds.

The cofferdam walls and the adjoining ground deflect to some extent as the excavation deepens. If this movement is to be limited, horizontal loads should be jacked into the frames. The jacks should have provision for measuring the load they carry, and the loads applied and maintained should be determined on the basis of observations of the movements of the cofferdam wall and of the adjoining ground.

In exceptional cases it may be necessary to push the walls of the cofferdam outwards by means of the jacks. The forces required in such an operation can be very large, approaching the passive resistance of the soil surrounding the cofferdam and much greater than the earth pressure at rest; the bracing, sheeting or wall should be designed accordingly.

Even with all these precautions to limit horizontal movement, it is not possible to eliminate entirely the settlement of the adjoining ground.

**5.3.3.8.3 Materials and stresses.** The walings, struts and other structural members comprising the bracing of cofferdams should be designed to withstand the earth, water and other loads transmitted to them without exceeding the safe stresses in the material of the bracing and without causing excessive deflection. Account should be taken of possible eccentric or vertical loading of the members. Frames should be protected against accidental damage and precautions taken to prevent their slippage.

The qualities of the materials and the working stresses should be as given in 5.2.

**5.3.3.8.4 Walings.** Walings should be designed as simply supported beams unless they are continuous over at least two spans or unless the joints are designed to develop the necessary shear and bending strength. In circular cofferdams, walings may be designed as ribs subjected to radial thrust. The ribs should be checked for buckling collapse. The resistance to collapse is less in the case of water cofferdams than in land cofferdams where the passive resistance of the soil helps to resist the buckling forces.

Walings that also act as struts to other walings, or that receive the load of diagonal struts, should be designed to withstand the stresses due to combined axial loading and bending.

**5.3.3.8.5 Struts.** When the spacing of struts is being decided, consideration should be given to the method of excavation; for example, if a grab is to be used, its size will dictate the minimum permissible clear spacing of struts.

The effective length of a strut should be taken as the distance between the walings unless the strut is adequately supported at an intermediate point, both horizontally and vertically, by means of secondary struts, vertical piles, bracing or other means. The connection between struts and walings has to be capable of transferring the imposed loads, particularly in the case of the diagonal struts. The connection needs to be such that, should the strut compression be removed or released, the strut will remain in position and not be easily dislodged. Where timber waling and struts are used, the bearing pressure on the waling across the grain will need to be considered.

**5.3.3.8.6 Anchors.** Where it is desirable to have a clear excavation or where struts would need to be of considerable length, it may be expedient to employ some form of ground anchor in place of struts. A variety of methods is available for anchoring in both rock and soils, most of which incorporate a cement grouted anchorage. Each method should be considered on its merits in relation to the ground conditions and the ground movements that would be permissible in a particular location.

It is important to consider the existence of underground services and the acquisition of wayleaves if anchors are likely to lie wholly or partly outside the site boundary.

Anchors should be designed to have their anchored length beyond the assumed slip plane given by the wedge theory (see Civil Engineering Code of Practice No. 2).

The anchors should be so spaced as to have sufficient soil or rock around them to develop simultaneously the required resistance of all the anchors.

It is normal to incline anchors at 30° to 45° below the horizontal. Due allowance should be made for this inclination in assessing the working load of each anchor. The vertical component of this load should not be neglected when assessing the vertical loads on the cofferdam walls. The capacity of the anchor to carry the designed loads should be demonstrated by site tests on actual anchors.

All individual anchors should be test loaded after curing of the grout to a load of at least 25 % in excess of the working load. The walings should be designed for the test load. The displacement under the working load should be acceptable for the purpose required.

**5.3.3.8.7 Vertical or king piles.** Vertical or king piles may be used within the cofferdam to reduce the effective length of struts, to carry part of the weight of the bracing, to resist the upward buckling of the bracing and to facilitate assembly.

### **5.3.3.9 Construction methods**

**5.3.3.9.1 Driving sheet piles.** Sheet piling for cofferdams should be driven with proper equipment and under experienced supervision, care being taken to avoid damage to the piles and to maintain them plumb and true to line. Damage to the heads of sheet piles may occur, and a small margin in length should be allowed for cutting off the damaged part.

The choice of the most suitable type of plant depends mainly on the site conditions. A pile frame or hanging leaders may be used with any suitable type of hammer. The hammer or other driving equipment may be suspended from a crane and should be provided with leg guides or grips to ensure axial driving of the piles.

There is always a tendency for sheet piling to get out of plumb in either direction. The tendency to lean across the line of the wall may be controlled by the use of two pairs of walings, spaced as far apart vertically as practicable. The walings should be substantial and held rigidly in position, and may be used for controlling the piles by wedges and packings.

The tendency of sheet piles to lean in the direction of advance of the wall may be controlled by pitching and driving in panels. The position of the piles should be marked on the walings before driving is commenced, and checks should be made at frequent intervals for any variations from the designed length of the wall.

To drive steel piles in panels the first pair of piles should be pitched and partly driven before any other pitching is done, every precaution being taken to ensure true verticality. The next 6 to 12 pairs of piles should be pitched and interlocked as a panel. The last pair of this panel should be driven to a firm penetration. The remaining piles in this panel should then be driven to their full penetration, working backwards to the first pair pitched. This method should be continued round the whole perimeter of the cofferdam. In a rectangular cofferdam it is usually convenient to arrange the closing panel to include a corner pile.

In the case of a small cofferdam it is advisable to pitch the whole of the sheet piling before driving is started. In tidal waters opened sluice valves should be included before pitching is complete.

If the piles are long and are subjected to hard driving, or if extreme accuracy is required, the panels should be driven to part depth only and the driving completed by working round the cofferdam in short stages of depth.

In circular cofferdams it is necessary to pitch the whole of the sheet piling before driving and to go round driving in 1 m to 1.5 m stages.

Straight web sheet piling requires special attention in handling, guiding and driving on account of the flexibility of the section.

**5.3.3.9.4 Pumping and watertightness.** Ample pumping capacity should be provided, particularly when dewatering is commenced and especially in tidal waters, as the joints of the sheet piles are not very watertight until a difference of head has been

**5.3.3.9.2 Fishplating, welding and cutting.** Where headroom is insufficient for piles to be pitched and driven in one length, they may be made up in two or more lengths, connected by butt welding or by fishplates designed to develop the necessary strength.

The fishplates may be riveted, welded or bolted to the piles. Where the upper lengths of pile are to be withdrawn, the fishplates should be bolted with nuts on the inside of the cofferdam, using square shank bolts or similar, to enable the nut to be removed and the bolt to be driven out of the joint from inside the cofferdam.

If the lower part of the sheeting to a cofferdam is to remain in position, for example as cut-off sheeting or to prevent scour, and where fishplating is not made necessary by restricted headroom, the sheet piling may be burned off.

If the cutting is to be done underwater, special oxy-hydrogen gas or oxy-arc cutting equipment is required.

**5.3.3.9.3 Excavation in cofferdams.** The provisions of **9.1** should be followed where applicable.

In the design of bracing for cofferdams, consideration should be given to the methods of excavation which may be employed. Steel or reinforced concrete bracing can be used to provide larger working bays and greater headroom than can normally be obtained with timber bracing.

The permissible level of excavation and/or water within a cofferdam, in relation to each frame of the bracing as it is successively fixed, is an essential part of the design of the cofferdam. This information should be clearly indicated on drawings prepared for the cofferdam construction. At no stage of the work should excavation be taken out to a greater depth than that for which the bracing and sheeting have been designed.

Sub-drains and pumping sumps should be planned to avoid increasing the designed loading of bracing and sheeting or reducing the designed depth of cut-off (see also **6.4**).

If extra excavation below the designed formation level is carried out, for instance, to remove unsuitable soil from the subgrade, the amount taken out and replaced at one time should be limited to avoid overstressing the cofferdam structure and to prevent disturbance of the ground. It may be necessary to provide additional bracing while this is being done.

created between the two sides of the sheeting.



Interlocked steel sheet piling is substantially watertight, but any leakage can generally be reduced by dropping ashes or similar material outside the cofferdam, so that it is drawn into the leaking interlocks and thereby effects a seal. In more serious cases caulking or welding of joints may be required. In tidal waters this process may have to be repeated on more than one tide. Any water percolating through the sheet piling should be intercepted, collected and led away to a sump in such a manner that the leaks do not affect the operations inside the dam.

The provision of one or more sluice gates or valves of ample proportions is advisable in large cofferdams to enable them to be flooded as a safety measure against collapse under unexpected strain, particularly with tidal waters or rivers liable to sudden floods.

Arrangements should be made to enable the sluice valves to be operated promptly in the event of an emergency. Sluice valves may also be used in the initial stages of dewatering a water cofferdam.

#### **5.3.3.10** *Extraction and removal* (see also 2.4)

**5.3.3.10.1** *Removal of bracing.* The erection of a structure inside a braced cofferdam usually necessitates the removal of the bracing which supports the sheet piling. Provision will therefore need to be made for transferring the load from the bracing to the new structure.

If the structure occupies the full width of the dam and the design permits, construction may be continued up to the underside of a bracing frame; this frame may then be removed when the concrete has acquired the necessary strength. If the permanent structure consists of two separated walls, these should be strutted sufficiently to take the temporary external thrusts. Alternatively, the frame may be made of reinforced concrete and incorporated in the structure.

If there is a space between the structure and the inside of the cofferdam, it may be filled with suitably compacted hardcore, sand and gravel, or other relatively incompressible material. The filling is carried to the underside of a bracing frame and this frame is then removed. However, if the face of the new structure is to be above ground level after completion, it is recommended that one of the following methods may be employed.

a) The structure is built up to a level slightly higher than that of the frame to be removed, a slot being left at each strut so that it is not disturbed. The waling is then wedged off the new structure by means of short struts or packings between the original struts so that the load is carried by the new “relieving” struts bearing on the higher portions of the new structure. The original struts can then be removed, the slots filled in and the next frame dealt with in a similar manner. If the structure is of masonry or has a facing of masonry, the level of the frames should be designed to correspond to the courses of masonry, and the spacing of the struts should be selected so that they correspond to arrangement of the stones in the course. Usually the omission of one stone in each face should be sufficient to accommodate a strut.

b) If it is impossible or undesirable to box out the original struts, a new waling strutted off the new structure may be inserted as close as possible under the original waling, which may then be removed. This procedure increases the span of the sheet piling between walings for which allowance should be made in the design of the dam.

c) The load from any waling may be transferred to vertical beams or soldiers (see 1.2.97). These are held at the lower end in slots formed in the new structure or by short struts, and at the upper end by new cross struts or by clamps, to an existing frame. If clamps are used, the design of the upper frame will need to provide for the additional load.

d) The walings may be braced by means of reinforced concrete struts which are incorporated in the structure and the projecting ends may be cut off after completion.

**5.3.3.10.2** *Pile extraction.* Piles may be withdrawn from the ground by the action of a hydraulic jack or by the use of a power-driven extracting hammer or vibrating extractor. The first is very effective but slow. In most circumstances the extracting hammer will give the best results. Driven piles should only be lifted by a direct pull from a crane after first having been withdrawn by one of the above methods.

#### **5.3.4** *Caissons*

NOTE The various types of caisson are defined in 1.2.

**5.3.4.1** *Choice of type.* Each deep foundation is a special case and no fixed rules can be laid down for guidance in the choice of the type of structure to be used. Frequently, the factors of time and cost will govern the choice of type as much as conditions of ground and water.



A caisson in free air or an open caisson may be employed where it is necessary to establish the foundation at a considerable depth below surface water or ground level, e.g. to depths which may in exceptional cases exceed 45 m. Excavation will usually be done by grabbing. Caissons are not commonly employed if the ground contains a large proportion of very stiff clay, owing to the great weight of the structure required to overcome skin friction in such ground, but skin friction may be reduced by lubrication with water or clay grouts. Open caissons are difficult to pass through rock or ground containing large boulders.

Compressed air or pneumatic caissons may be used for a wide variety of ground conditions. These include the following:

- a) in water-bearing ground where it is required to inspect the soil at foundation level, and where alternative means of dewatering are impracticable;
- b) where the caisson is to be sunk through water-bearing strata containing rock either as beds or as large boulders;
- c) where the caisson is to be used as a shaft in which an opening is to be made in a water-bearing zone, for the purpose of driving or receiving a tunnel;
- d) where it is necessary to avoid subsidence of adjacent ground or structures due to inflow of soil into the caisson, as may happen with open caissons sunk by grabbing.

The depth to which a compressed air caisson may be taken depends chiefly upon the effects of compressed air upon workmen exposed to it (see 6.5.1).

A caisson in free air may be converted to a pneumatic caisson if a change in ground or water conditions requires it and the conversion has been allowed for in the design. Conversely, a caisson that is commenced in compressed air may be converted to a free air caisson.

**5.3.4.2 Choice of material.** Caissons, whether sunk in free or compressed air, may be constructed of steel, cast iron segments or reinforced concrete, or a combination of these.

Reinforced concrete caissons (monoliths) may be preferable to steel caissons during sinking because of their greater weight.

The area of the caisson in relation to that of the superstructure should be sufficient to provide for some deviation from its correct position in any direction during sinking.

Steel caissons usually present fewer problems during their construction than reinforced concrete caissons.

### 5.3.4.3 Design

**5.3.4.3.1 General.** The design of a caisson should be governed mainly by the method proposed for sinking, the skin friction that may be expected during sinking and the nature and dimensions of the permanent structure required. In some cases caissons for sinking through water are designed to be built on shore and floated to the site.

The caisson as a whole should be of robust construction. It may be of framed steel plate, cast iron or reinforced concrete segments, or of in situ reinforced concrete or mass concrete. The sides of the caisson should be strictly parallel and free from bulges, and constructed so as not to lose their shape during sinking.

In designing the caisson the forces that will act upon it at various stages of its installation should be considered. These forces may include those due to construction, launching, towing, pitching, sinking, ground pressure, water pressure, compressed air pressure, kentledge loads, uneven sinking and the means used to correct uneven sinking.

In some cases the walls may be strengthened, permanently or temporarily, by concrete lining or additional bracing.

All caissons should have a cutting edge of steel or cast iron, projecting 25 mm to 50 mm beyond the face of the caisson, depending on the nature of the ground.

The roof of a pneumatic caisson should be of sufficient strength to take the air pressure and the superimposed load, and to transmit the latter to the side walls and thence to the cutting edge. No allowance should be made in the design of the roof for the air pressure carrying part of the load.

It is desirable that the height of the working chamber in a pneumatic caisson should be not less than 2.5 m to provide sufficient headroom when the cutting edge is embedded a short distance below the excavated level, and in particular to allow for blowing down. The permitted points of support for temporary jacking or packing should be clearly indicated on the underside of the roof of the working chamber. The roof should be designed accordingly.

**5.3.4.3.2 Skin friction.** Values of skin friction are not directly related to the shear strength of the surrounding soil but may vary over a wide range. For purposes of design it is usual to assume a figure of  $10 \text{ kN/m}^2$  to  $25 \text{ kN/m}^2$  ( $0.1 \text{ kgf/cm}^2$  to  $0.25 \text{ kgf/cm}^2$ ) when in movement and considerably more to start movement when the cutting edge is completely undercut. These figures may be considerably exceeded if the caisson loses its shape. Skin friction will be reduced by the leakage of air or water between the caisson and the surrounding ground.

The skin friction may be artificially reduced by water jetting or the introduction of clay suspensions or other lubricating substances.

**5.3.4.3.3 Workmanship and materials.** A caisson is generally intended to be a watertight box or shell and it is important that workmanship as well as materials should be of good quality. Materials and workmanship for steel caissons should generally be in accordance with BS 449. Other materials should be in accordance with relevant British Standards.

#### **5.3.4.4 Sinking the caisson**

**5.3.4.4.1 Open caissons.** The site should be prepared by levelling and clearing all obstructions. It may be necessary to support the lower part of a caisson during erection by building it on a bed of hardcore, concrete or timber. If the caisson is to be sunk through water or mud, it may be slung and lowered from a temporary fixed or floating structure. The caisson should be built up as high as practicable before excavation is commenced within it.

Excavation should be as uniform as possible; if the caisson is out of plumb, excavation on the high side should be carried below that on the low side, without, however, ceasing excavation on the low side entirely, as it is impossible to plumb the caisson without lowering it as a whole. Excavation should not be carried further below the cutting edge than is necessary to keep the caisson moving uniformly. Sinking should be reasonably steady and uncontrolled movements should be avoided, particularly those that may lead to tilt.

If water jets are employed, they should be used with caution to avoid sudden drops in sinking the caisson.

Kentledge may be used to increase the weight of the caisson to overcome the resistance to sinking.

**5.3.4.4.2 Compressed air or pneumatic caissons.** The recommendations made for open caissons also apply to pneumatic caissons, in general, a caisson should be sufficiently heavy during sinking that it will sink as excavation proceeds, overcoming air pressure and frictional resistance.

If it is not possible to make the caisson heavy enough during excavation, blowing down may be used. The men should be withdrawn and the air pressure reduced. The caisson should then begin to move with quite a small reduction in pressure. No general rules can be set down covering the relationship of movement and pressure reduction. Blowing down should only be used where the ground is such that it will not rise into the chamber when the air pressure is reduced. If movement does not begin when the air pressure has been reduced by one-quarter of the gauge reading, it may be taken that the caisson is too light and kentledge should be added.

Blowing down a caisson should be done in short stages and the drop should not exceed 0.5 m. Sinking may be controlled by the use of hydraulic jacks under the walls of the working chamber.

#### **5.3.4.5 Sealing of caissons**

**5.3.4.5.1 Open caissons.** All loose material should be removed from the bottom of the excavation, which should be reasonably level. As a rule a grab will not reach the portions under the haunching adjacent to the cutting edge, and these will therefore need to be cleared by hand. If the caisson is sunk in water-bearing ground, this will have to be done by a diver, and it will be necessary to place the first layer of concrete under water (see section 8). Concrete deposited under water should preferably be poured continuously to the full thickness to avoid laminations and should have a cement content of not less than  $450 \text{ kg/m}^3$ .

Before placing the structural plug, a preliminary layer of concrete should be placed and levelled; when set it should be cleaned before placing any further concrete or reinforced concrete. Sufficient concrete or reinforced concrete should then be placed in the bottom of the caisson, or in each well of a multi-well caisson, to resist the upward water pressure when it is pumped out. Before pumping out, the caisson should be sufficiently loaded to prevent flotation.

**5.3.4.5.2 Compressed air or pneumatic caissons.** The floor of the working chamber should be levelled off approximately to that of the cutting edge. In suitable ground the area of the base may be increased by undercutting the outside face of the cutting edge. Undercutting and pinning-up of the cutting edge should be carried out in short sections.

Where the roof of the working chamber is to be part of the permanent structure, the concrete filling should be placed in such a manner as to ensure complete filling of the chamber. To facilitate the packing of concrete filling, the roof of the working chamber may be cambered to rise upwards from the periphery to the access shaft. Filling should be carried out progressively from the periphery towards the access shaft, forming the filling in layers and benches to enable concrete to be packed tightly to the underside of the roof of the working chamber over the whole area.

Grout pipes should be provided at the underside of the roof of the working chamber leading to the periphery from the access shaft, to enable cement grout to be injected under an effective pressure of about  $400 \text{ kN/m}^2$  ( $4 \text{ kgf/cm}^2$ ) to fill any spaces left by shrinkage or incomplete packing at the underside of the roof.

#### 5.3.4.6 Air supply

NOTE For further information regarding work in compressed air see 6.5.1.

Air should be introduced to the caisson directly through the roof of the working chamber rather than by way of the shaft. In porous soil, the air will pass into the working chamber and out under the cutting edge, circulation thus being ensured. When the cutting edge is passing through clay, the escape of air under the cutting edge may become reduced. The circulation of fresh air for the benefit of workmen may then be maintained by opening a valve to the atmosphere.

A safety valve should be fitted to the air supply line to limit the air pressure. Instructions should be given in writing to the person in immediate charge of the work stating the air pressure to be used at each stage of the work.

**5.3.4.7 Position of air locks.** Air locks should be located at such a height that during sinking they will not descend so low as the highest possible water level. If the depth of sinking is great, it may be necessary to add lengths of air shaft at intermediate stages of sinking.

**5.3.4.8 Use of explosives.** Explosives may be used in the working chamber for breaking obstacles that hinder sinking, such as boulders, or for levelling the formation when sinking onto a rock foundation.

The following precautions should be taken.

- a) One person, experienced in the use of explosives, should be appointed as the shot firer. The key of the exploder should be in his possession before the loading of the shot holes is started.

When charging the explosive, all men except those laying the charges should be removed from the working chamber.

- b) Charges should be as light as practicable; damage may be caused to the caisson if charges are in excess of  $0.7 \text{ kg}$  of gelignite per hole. In exceptional circumstances, under expert direction, larger charges may be fired. The burden of the charge should be limited to  $1 \text{ m}$ , and a pattern of charges should be arranged with delay detonators to reduce the number of charges fired at any one time. The pattern of charges will normally be spaced at  $0.5 \text{ m}$  to  $0.6 \text{ m}$  centres, with about 24 holes per round.

In soft rock, the charge per hole will be about  $0.4 \text{ kg/m}$ ; for hard rock the charge per hole may be up to  $0.75 \text{ kg/m}$  depth of shot hole and will depend on the nature and formation of the rock, but the round should be limited to a total of  $5 \text{ kg}$ .

- c) Before detonation, all men will have to be withdrawn from the working chamber, and danger notices should be hung on airlock doors. Men may remain under pressure in the airlock with the inside door closed.

- d) The charge should be fired electrically from outside the working chamber. Firing cables should enter the working chamber by a duct used for no other purpose. Charging should be carried out by the light of battery-operated hand lamps.

Before the charge is connected, all electric supply to the working chamber should be disconnected.

- e) After detonation, the fumes and dust will need to be cleared by opening a relief valve to withdraw air from the working chamber at a point remote from the air supply inlet. The rate of air exchange will depend upon the size of the working chamber, the capacity of the air compressors and the pressure drop in the working chamber which can be permitted, usually  $15 \text{ kN/m}^2$  to  $20 \text{ kN/m}^2$  ( $0.15 \text{ kgf/cm}^2$  to  $0.2 \text{ kgf/cm}^2$ ). Approximately 20 min should be allowed for fume clearance before allowing men to re-enter the working chamber, but tests should be made with a safety lamp and silica gel tube for the presence of carbon dioxide and carbon monoxide.

- f) In addition to the foregoing recommendations, statutory regulations and customary precautions in connection with the use and handling of explosives have to be observed (see BS 5607). Booklets are issued on application by suppliers of explosives and blasting accessories.

#### **5.3.4.9 *Welding and cutting in caissons***

**5.3.4.9.1** Gas welding and cutting may be carried out in a compressed air caisson, but frequent intervals will need to be allowed for the clearance of fumes. Good general ventilation should be provided and in certain circumstances local exhaust ventilation and gas respirators may be needed (see 11.1.4). Gas respirators may be advantageous.

Blow pipes may be used; these will give adequate results up to a pressure of 100 kN/m<sup>2</sup> (1 kgf/cm<sup>2</sup>). Above this pressure propane, butane or similar fuel gas has to be used; with these gases an injector type blow pipe is required.

It is recommended, but not essential, that the gas supplies to the working chamber should be carried from pressure bottles outside through metal pipes with control valves outside and inside.

Hydrogen, methane and coal gas should not be used as fuel gases in working chambers of compressed air caissons.

**5.3.4.9.2** Welding and cutting may be carried out with the aid of electrical apparatus operating on either direct or alternating current. The metal being cut or welded should be properly earthed.

Fumes arising from the point of contact are frequently dense. Fumes cause discomfort to the eyes and can be injurious to health.

### **5.4 Safety precautions**

Recommendations for the safety of personnel constructing cofferdams and caissons are given in 11.2.



# Section 6. Geotechnical processes: groundwater lowering, grouting and other methods of changing

## the ground characteristics in situ

### 6.1 General

This section covers the application of a number of established processes to facilitate foundation construction in difficult ground conditions. The methods used are often known as geotechnical processes; they include such techniques as groundwater lowering, injection of grouts, compaction, use of compressed air and freezing the groundwater. Although these methods require widely different techniques, they all demand an adequate knowledge of the ground and water conditions to a scale of detail relevant to the processes envisaged. This information will need to be obtained before a choice is made.

Before considering any specific expedient, a full appraisal of the overall requirement to be solved has to be made. The various expedients and devices available should be considered at the design stage in relation to the general pattern of the engineering work and the soil information available. This is necessary because dewatering and ground treatment are usually carried out in the early stages of an engineering project. If consideration of these techniques is left until difficulties are encountered during the construction work, appreciable delays may result whilst the necessary geotechnical process is being carried out.

**NOTE** It has to be emphasized that this section is complementary to many of the other sections, and should be read in conjunction with those that are relevant to the particular problem and method of construction under consideration.

### 6.2 Site investigations

#### 6.2.1 Preliminary investigations

The preliminary stages of a site investigation are mainly concerned with establishing the nature and general stratification of soils underlying the site and should follow the lines laid down in BS 5930. If the preliminary results indicate that geotechnical processes will be needed, they should be considered in the preliminary schemes for the foundation design and construction. If any of the schemes involve the use of geotechnical processes, more detailed information will be required as outlined in 6.2.2.

#### 6.2.2 Detailed investigations

**6.2.2.1 General.** This stage of the investigation work should have the specific aim of studying in full detail the soil profiles, groundwater conditions and general foundation construction history in the area, to the extent that these factors are relevant to the particular problem under consideration. Aspects requiring more detailed consideration are given in 6.2.2.2 to 6.2.2.6.

**6.2.2.2 Detailed soil profiles, particle size analysis and soil conditions.** It is important to obtain not only a representative soil profile but, particularly when fine grain soils are being considered, to undertake continuous or nearly continuous sampling. Special techniques are necessary to obtain representative samples of granular soils and, if silts and soft clays are present, the use of a piston or continuous sampling device should be considered so as to retain all fractions of the sample. Particle size analyses should be made on typical samples from all silt, sand and gravel layers; in the case of silts and fine sands, the particle size analyses should be made on typical portions of undisturbed samples which have been cut open for inspection to determine the soil structure and, in particular, whether or not laminations are present. In uniform non-laminated strata, particle size distribution curves can be used to obtain an estimate of permeability of sands containing less than 5 % of clay, provided that the density of the soil is known. For complex strata or for works where the geotechnical treatment is a substantial part of the cost, it is prudent to cross-check permeability by in situ tests appropriate to the process envisaged (see 6.2.2.6 and BS 5930).

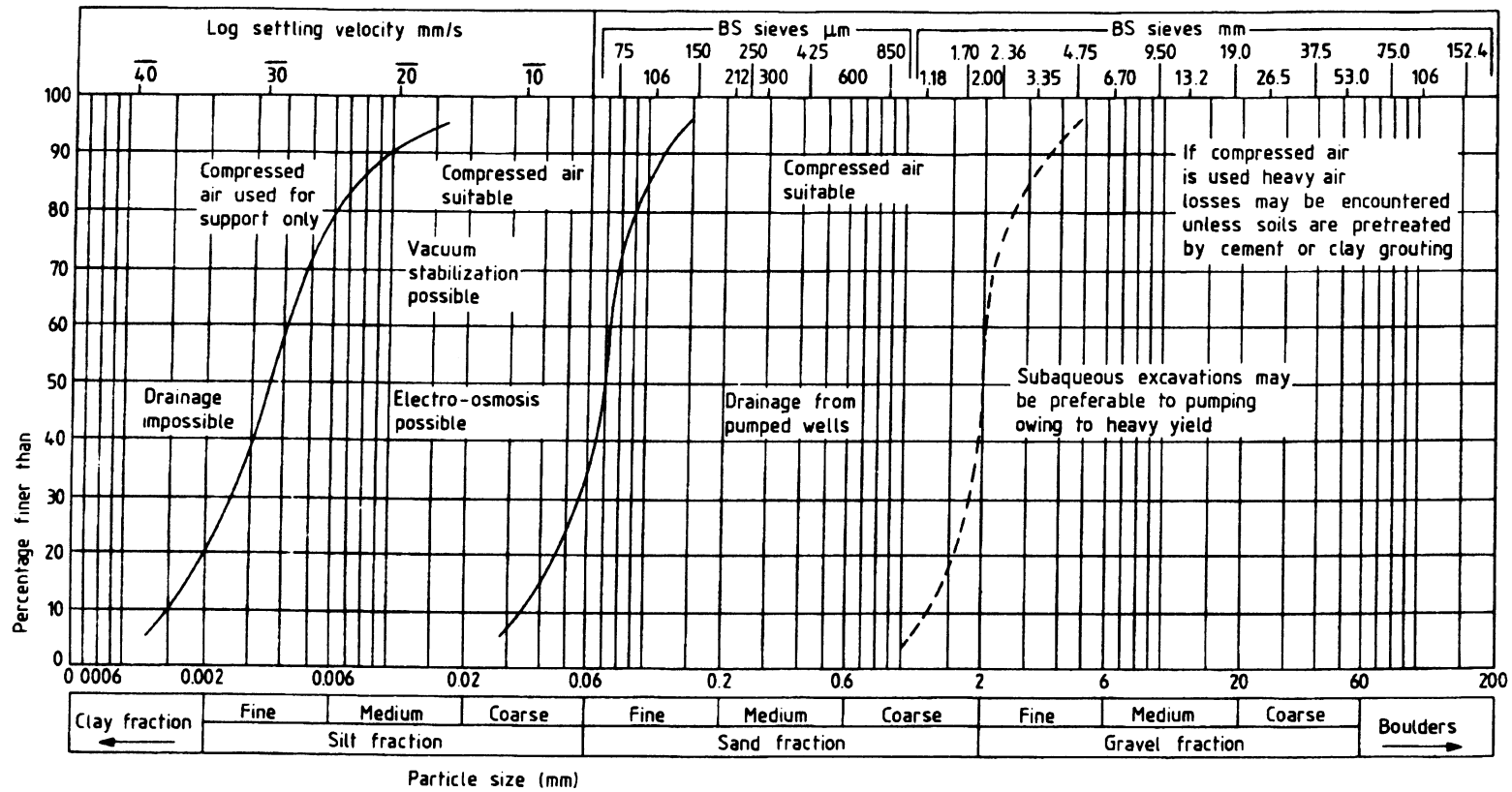
In the vicinity of existing structures the compressibilities of the various strata are required in order to ascertain whether excessive settlements due to pumping are likely to cause damage to structures within the area of influence of the pumping. (It is pointed out that damage is not an inevitable consequence of groundwater lowering.) Soil classification tests are also of considerable assistance when deciding which of a number of possible geotechnical processes can be selected for use at a particular site. Tentative particle size distribution ranges for some of these are given in Figure 4(a) and Figure 4(b) (Glossop and Skempton 1945). It is emphasized that range boundaries depend on many factors peculiar to each site and ranges overlap to some degree; most processes work well in the middle range and less well near the boundaries.



### 6.2.2.3 *Groundwater hydrology*

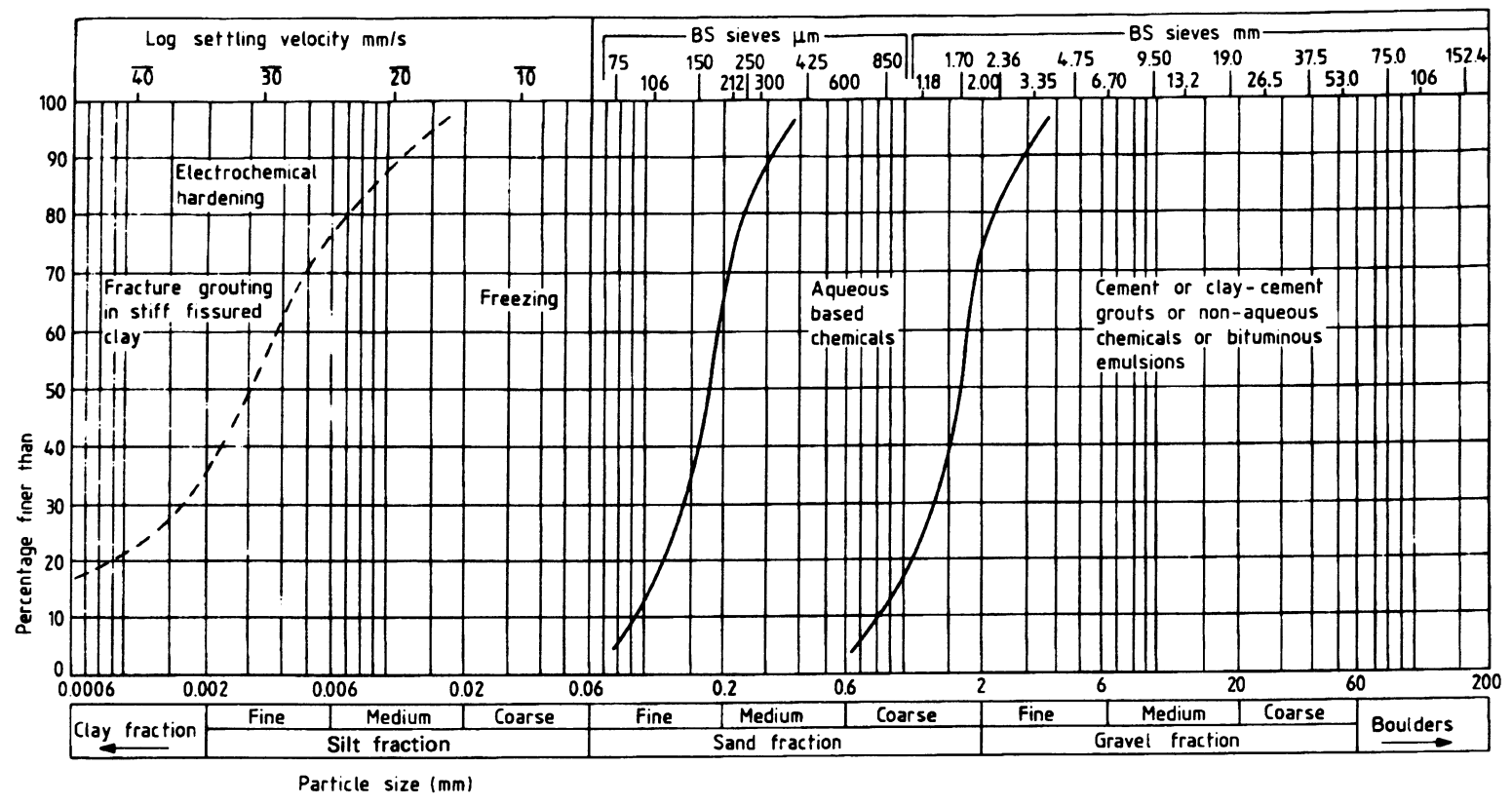
**6.2.2.3.1 *General.*** A general appreciation of the pattern of groundwater flow can be of considerable value when considering such aspects as the location of structures, the layout of groundwater lowering installations, the penetration of sheet piling in cofferdams or the thickness and composition of grout curtains. The object of such a study is to establish the sources of water and deduce the way in which it flows in the region of the site and the possible influence of the engineering work and completed structure on this flow system. It should be recognized that groundwater flow may be very different from theoretical patterns because of ground variation.

**6.2.2.3.2 *Sources of groundwater.*** Groundwater in a pervious soil stratum may be replenished directly from rain falling on the ground surface or by percolation from run-offs, streams or nearby rivers. The water pressures in pervious layers will usually change as a result of variations in seasonal rainfall, river and sea level in cases where there is direct connection between the pervious stratum and these sources.



NOTE. This figure is based on 'Particle size in silts and sands' (Glossop and Skempton 1945).  
 (a) Groundwater lowering and compressed air

Figure 4 — Approximate guide to application of various geotechnical processes to soils



NOTE. This figure is based on 'Particle size in silts and sands' (Glossop and Skempton 1945).

(b) Artificial cementing

Figure 4 — Approximate guide to application of various geotechnical processes to soils (concluded)

**6.2.2.3.3** *Groundwater flow.* For the purposes of this standard, groundwater problems can be divided into two main groups.

a) Problems in which the water is confined by a layer of low permeability along the upper boundary in the area of interest. This is shown in Figure 5(a). When the source of replenishment is or has been at a higher level than the ground surface at the point of interest, the water head in confined pervious strata may be higher than ground level. Such a condition is said to be artesian.

b) Cases where the upper flow boundary is not confined by a layer of low permeability and in consequence the pressure on this boundary is equal to atmospheric pressure. This type of flow is shown in Figure 5(b) by the case of unconfined flow to a well.

**6.2.2.3.4** *Measurement of water pressure in the ground*

**6.2.2.3.4.1** *General.* In order to obtain accurate measurements of groundwater pressures it is necessary to install special measuring devices called piezometers.

Water conditions in aquifers will need to be measured by means appropriate to the strata. In some cases a simple standpipe placed in the ground will seem to suffice, but as the accurate measurement of the true groundwater level, with due regard to the response time, is essential for engineering requirements it can be a false economy not to install a reliable piezometer.

In all cases it is necessary to make periodic checks on the readings of water pressure, as water levels can be influenced by tides, nearby water courses, canals, rainfall, broken water pipes and sewer pipes. It is necessary to give the aquifer an opportunity to respond to the installation of the piezometer, which causes local disturbance of the soil surrounding it. Frequent checks are necessary on water level measurement made in a partly sunk borehole where aquifer changes can affect the readings.

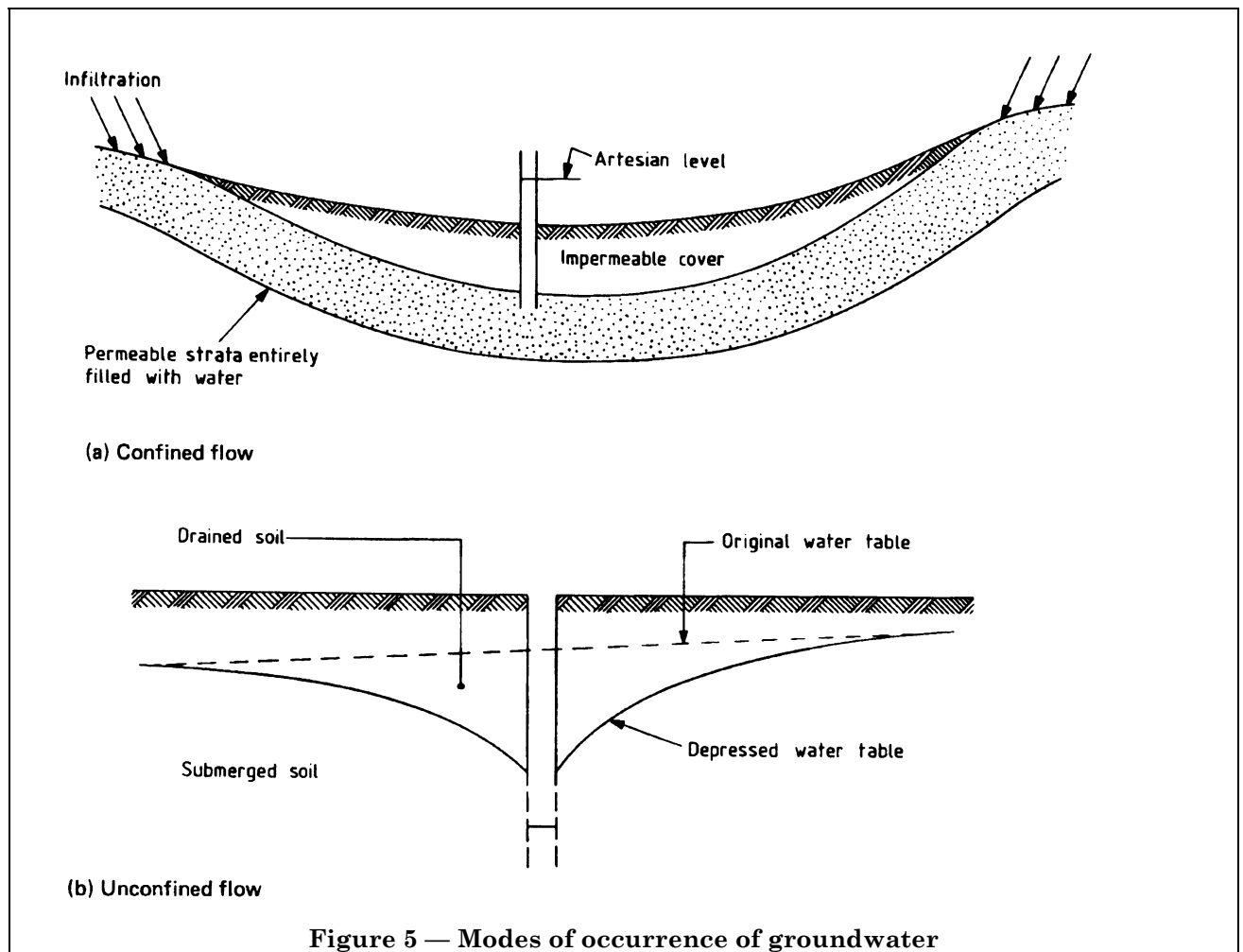


Figure 5 — Modes of occurrence of groundwater

It is necessary to measure the water pressures in the pervious strata at a number of typical positions in or around the constructional site in order to:

- ascertain the maximum water pressures to be met during construction;
- build up a picture of the direction of the groundwater flows within or through the area;
- measure changes of piezometric level produced by pumping tests and groundwater lowering installations;
- ensure that water pressures in the ground do not exceed values which would cause damage to the permanent or temporary works both during and subsequent to construction, for example, pressures due to hydrostatic uplift.

**6.2.2.3.4.2 Choice of installation of piezometers.** A piezometer is any suitable device for measuring water pressure at a chosen elevation in the soil strata and can consist of an open-ended steel pipe, a porous pot or disc, or devices containing a diaphragm giving electric responses to changes in pressure.

Generally, the more open and permeable the soil the simpler is the piezometer; fine-grained soils such as fine sands, silts and clays require special knowledge for selection of suitable piezometers.

The type of piezometer used will then depend on the permeability of the soil strata, the response time of the instrument to variations of water pressure in the soil and also on the magnitude of the water pressures at various positions in the ground relative to the elevation of the measuring station.

In permeable ground where there is no stratification and therefore, little or no departure from a straight line increase of head with depth, a simple method of measuring the groundwater pressure is to use a vertical standpipe consisting of a plain steel or plastics tube of diameter, for example 12.5 mm to 25 mm, fitted at the foot with a perforated length related to the thickness of the aquifer. In some cases the steel pipe can be driven or jetted into the ground; otherwise, when the ground conditions make it difficult to jet or drive, it can be inserted in a borehole.



When a standpipe or any other type of piezometer is installed in a borehole, it is necessary for the perforated length of piezometer tip to be surrounded with a clean sand filter. This filter should preferably be carefully sealed above and below the piezometer to enable the water pressure at that horizon to be measured with the minimum influence from the strata which may contain water at different pressures.

Where leads are taken from the piezometer to the surface they need to be protected against damage and surrounded in a sealing grout. Where taken over the ground, they have to be put in a conduit or trench to avoid damage from surface movements, excessive sunlight, inclement weather, etc. Care should also be taken to see that the leads are kept free from blockage. The choice of material for the leads has to ensure that they are not blocked by crystallization. In rocks, the response to the piezometer will depend very much on the jointing, bedding and porosity of the rock in which the piezometer is actually installed. Measurement of water pressures in rock can be misleading if due regard is not given to the position of the piezometer in relation to the rock structure. Soundings of the water levels in standpipes should be done with care; it is preferable to employ simple measuring devices making use of an electrical contact for this purpose. The time required for the measuring system to respond to changes of water pressure in the ground depends upon the dimensions of the perforated section and its surrounding filter and on the internal diameter of the standpipe, as well as on the permeability of the soil surrounding the filter (see BS 5930). In fine sands or silts, plastics tubes of about 12.5 mm in diameter fitted with ceramic tips should be used in preference to other types. Electrical-type piezometers or twin-pipe hydraulic piezometers connected to Bourdon tubes or mercury manometers will usually be required in fine silts and clays and in positions where vertical standpipes are inconvenient, e.g. under a structure.

Electrical-type piezometers can be used only where they are permanently below the water table and where they are not likely to be affected by the emergence of air or other gases from solution in the groundwater.

Attention has to be given to the effect on piezometer readings of local tidal changes, floods, watercourse levels, cloudbursts, damaged pipes and sewers and other external forces. Water pressure should, wherever possible, be measured on a number of occasions to take into account seasonal changes. It is very important to know the maximum pressures that are likely to occur and this may require observations over several months, or inspection of records kept by others.

**6.2.2.3.5 Rate of flow and permeability.** The rate at which water passes through a cohesionless soil (silt, sand and gravel) depends upon the size of the grains, the proportion of the volume occupied by connected voids and the hydraulic gradient of the flowing water.

Although Darcy's law holds strictly for laminar flow, it is applicable to flows through pervious materials and for most conditions of groundwater flow. It can be expressed by the equation:

$$Q = AK_w i$$

where

- $Q$  is the quantity of water flowing in unit time (in  $\text{m}^3/\text{s}$ );
- $A$  is the cross-sectional area across which the flow occurs (in  $\text{m}^2$ );
- $k_w$  is the permeability of the soil to water (in  $\text{m}/\text{s}$ );
- $i$  is the hydraulic gradient which equals  $H/L$  (where  $H$  is the loss of water head over a distance  $L$  in the ground).

The range of permeability,  $k_w$ , for soils is very large, as can be seen from typical values given in Figure 6 showing permeability and drainage characteristics of soils. These values have to be used with caution, as permeability can vary appreciably within an aquifer over a very short distance either vertically or horizontally.

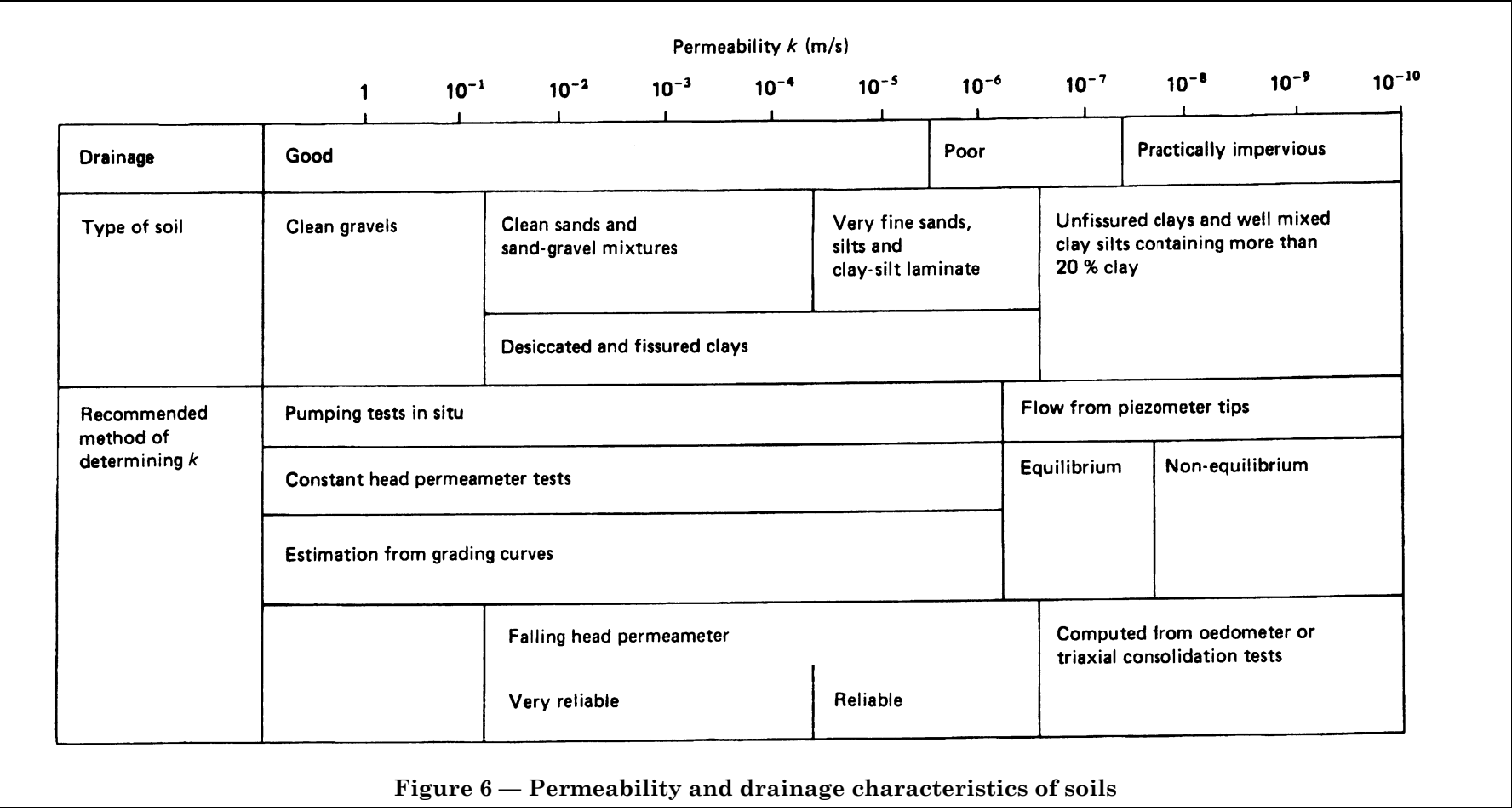


Figure 6 — Permeability and drainage characteristics of soils

The permeability of unfissured clays and rocks free from bedding and jointing is usually very small, but such conditions may be only local and realistic values may be very different. Furthermore, a large excavation may cause fissures to open and permeability to increase.

It is very desirable when considering permeability characteristics of weak and strong rocks to take advice from an engineering geologist or hydrologist, as consideration of the rock structure as a whole is likely to be more valuable than relying on inspection and testing of small rock cores.

Where it is established that rocks are likely to contain planes and areas of weakness due to bedding, jointing, fissures and faulting and in those cases where they are affected by weathering, desiccation or solution processes, appreciable quantities of water may pass through. In such cases it is the size and distribution of the cracks, voids and weaknesses, not the size of the individual grains, that will control the apparent permeability.

Darcy's law of flow may be used in those cases where the cracks, fissures, etc. are reasonably uniformly spaced and, in the case of clays, fairly narrow.

In some of the weak rocks, such as chalk, marl and certain limestones, the size and distribution of the cracks and voids can be very variable, so the quantity of water flowing will vary greatly from place to place. Thus a well in such conditions may have a very different yield, depending on the number and size of cracks intercepted.

**6.2.2.3.6 Measurement of permeability,  $k$ .** There are several recognized methods of measuring permeability either in situ in the field or in the soils laboratory. None is absolute in the sense that it is wholly accurate, and it is of practical value to assess permeability characteristics by combining the results of more than one method of measuring permeability and relating this to the particular problem under consideration.

The usual methods considered are:

- a) undertaking full-scale pumping tests in the aquifer;
- b) undertaking rising and falling head tests in a borehole;
- c) computing the value from the grading curves on the soil samples taken from the aquifer;
- d) computing values from oedometer or triaxial consolidation test results;
- e) computing values from flow conditions measured in boreholes and piezometers.

The techniques for making the tests and interpreting the results for methods a) and b) are described in BS 5930.

**6.2.2.4 Safeguarding existing structures and interests.** It is necessary to ensure that the construction operations do not affect the stability of adjacent structures, therefore consideration should be given to the following.

- a) Settlement may be caused by the reduction of pore water pressure in compressible layers of soil. Where permeable strata are overlain by or interbedded with compressible soil layers, settlement of the ground surface and any shallow foundations may occur as a result of consolidation of the compressible layers, owing to the increase in effective pressure of the over-burden soils caused by the reduction in pore water pressure. The settlements resulting from a properly designed groundwater lowering system may be only quite small and not sufficient to cause appreciable damage to structures. It is, however, necessary to determine the compressibility of any layers of soil which may be affected and it is advisable to take settlement observations during the pumping test and for a reasonable response time afterwards. This information, together with estimates of the reduction in groundwater pressures which are likely to occur, can be used to calculate the order of settlement to be expected. It may be possible to limit the risk of settlement due to lowering the water table by injecting water beneath or alongside any structures which may be affected. The injections through these "recharge wells" should be carefully controlled to ensure that the water table does not fluctuate appreciably above or below its natural level.
- b) Movement may be caused by the restoration of the groundwater table at the end of construction or by raising the groundwater level by the construction of impervious barriers such as grout curtains or concrete walls. This may produce settlement in beds of loose sand or in loosely compacted backfills, or it may result in swelling of desiccated clays.
- c) Heave may be caused by the excessive spread of grout through very permeable horizons or along interplanes or successive strata, sometimes owing to using too high injection pressures and rates.
- d) In very large dewatering installations, it may be necessary to take into account the influence of pumping on adjacent water abstraction facilities. Precautions have to be taken to prevent contamination of fresh water supplies.
- e) Heave caused by ground freezing installations may damage adjacent structures, road surfaces or drainage works.

- f) Vibrations from surface or deep ground compaction processes may damage adjacent structures.
- g) Materials used in grout injection or slurry trench processes may contaminate water courses or underground water resources.

**6.2.2.5** *Chemical properties of the groundwater and soil.* When groundwater lowering is undertaken using bored wells, encrustation of the filter materials can occur owing to the presence of soluble salts in the water and flow reduction caused by algae and bacterial slimes.

If electric submersible pumps are used to pump groundwater containing salts, sometimes stray leakage currents from the motor can accelerate corrosion of any metal lining to the wells. When wells have to be in use over a period of months, it is advisable to use well linings treated with a protective coating against corrosion. Linings fabricated from non-corrosive material are desirable for long-term pumping.

If grouting is to be used, the soluble salts in the groundwater may have undesirable effects on some grouts. It is therefore necessary to know the chemical composition and pH values of the groundwater before choosing the type of grout to be used.

**6.2.2.6** *Change in properties of the ground as the result of ground treatment.* It is practical in many cases to measure improvements in strength of the ground that has been injected; sometimes representative samples of the soils to be treated are subjected to injection by suitable grouts in a laboratory. Such samples can be tested for their strength characteristics which can be related to those prior to treatment. However, the results should be regarded with some caution as the injection process in the laboratory does not accurately simulate the full scale condition.

Permeability tests or water level observations can be undertaken in treated ground to measure the reduction in permeability due to the treatment, although a large number of tests is required to obtain a meaningful result.

In some cases improvement in strength characteristics of rocks as a result of grouting can be measured in situ by using special instruments placed in boreholes in the rock.

The effectiveness of treatment can be assessed either in situ or on samples of soils taken from treated areas. Visual inspection of good quality cores of treated ground can provide a reliable indication of the spread of ground treatment.

## 6.3 Choice of geotechnical process for control of groundwater and ground deformation

### 6.3.1 General

The method finally adopted will depend upon such factors as the dimensions of the excavation; the thickness and type of soil strata; the position of the excavation and permanent structure relative to the soil strata; the magnitude of the water pressures in the various strata; the prevention of damage to adjacent structures; the length of time for which the excavation has to be open and the overall economics of any particular solutions. An additional factor that should receive more consideration is the relationship between the geotechnical process and the constructional sequence.

Ground conditions are not the only factors which govern the selection of a particular process. A method that would be suitable for a given type of soil, e.g. in open ground conditions, might be quite unsuitable for adoption in an urban environment. Generally, the wide range and complexity of present-day techniques necessitate advice from engineers specializing and experienced in such processes.

In arriving at a final solution, the following possibilities should be considered.

### 6.3.2 Avoidance of groundwater

In circumstances where extensive temporary work is required to facilitate the construction of foundations or where permanent work is necessary to render the ground capable of supporting the foundation pressures, it is sometimes possible and more economical to avoid these constructional difficulties instead of combating them. For example, to avoid working in waterlogged ground, a shallow raft foundation, either solid or cellular and founded above the groundwater level, can often be used. Alternatively, the loads can often be transferred to underlying firmer strata by using piles. In some cases it may be possible to reduce construction difficulties by rearranging the location of individual structures so that the deeper excavations are made in more favourable ground.



### 6.3.3 Exclusion of water from excavations

#### 6.3.3.1 *Use of open drains to keep out surface water.*

Where an impermeable bed occurs at or near the surface, it is advisable to deal with the surface water as a separate problem, even though it is anticipated that water-bearing strata will be encountered at a greater depth in the excavation. This may be accomplished by surrounding the excavation by open ditches or open jointed drains taken down into the top of the impervious bed. The water should be led to a pumped sump or allowed to drain naturally to a watercourse. In some cases it may be convenient in excavating for a foundation to form a benching at the level of the top of an impervious bed and provide an open drainage system to collect the water from overlying pervious layers.

#### 6.3.3.2 *Use of cut-offs to exclude water.*

Water may be excluded from the area by surrounding it by an almost watertight box or cofferdam. Where the nature of the ground permits sheet piles to be driven down to an underlying impermeable layer, this will generally be the surest means of reducing seepage to an excavation. In conditions unsuitable for sheet piling, continuous in situ diaphragms of concrete or other impervious materials or the injection of grouts to form a curtain may be appropriate. Alternatively, the groundwater may be excluded by freezing the ground immediately adjacent to the excavation. In very loose coarse or laminated soils horizontal permeability has sometimes been reduced simply by compaction.

#### 6.3.3.3 *Use of compressed air.*

In some cases compressed air provides a more suitable means of excluding water from an excavation, and the pressure also helps to stabilize the ground in the face or bottom of the excavation (see 6.5.1).

### 6.3.4 Removal of water

When the soil conditions are such that the sides or base of the excavation will remain stable, usually the least expensive method is to pump the water after it has entered the excavation. If, however, there is a possibility of instability of unsupported slopes or bottom of excavation, it is advisable to remove the water from the ground before it reaches the excavation. Even when it is possible to dewater the excavation by pumping from internal sumps, it may be desirable, and possibly more economic, to use an external groundwater lowering installation, since both congestion within the excavation and the amount of internal support may be reduced. When large quantities of water have to be removed, an external system can also provide drier working conditions and less risk of erosion and damage to the work. Unstable fine silty sand may sometimes be stabilized by vacuum points which can change the seepage pattern to produce stability.

### 6.3.5 Ground treatment to change the physical properties of the ground

#### 6.3.5.1 *General.*

A number of geotechnical processes are available to change the existing properties. They may be used to improve strength and deformation characteristics, degrees of permeability, and to accelerate rates of consolidation to reduce ultimate settlements of structures supported on compressible soils.

#### 6.3.5.2 *Removal and replacement of the weak strata with strong compact material.*

It may be desirable to remove the soil strata concerned and replace it with an alternative stronger material or good imported soil compacted in place. In certain cases it may be possible to stabilize a weak soil by mixing in cement, lime or other admixtures. Alternatively, it may be more economic to provide a smaller foundation at a greater depth where the ground is stronger.

#### 6.3.5.3 *Compaction of soils in situ.*

Compaction relies either on collapse of cohesionless permeable soils from an initial loose to a dense state influenced by gravity following transient shock, or on forced volume reduction of cohesionless and fine soils; a combination of such effects occurs in some processes. Useful depths of compaction for most foundations on sands, gravels and fills can be achieved by shocks induced by exploding buried charges (Prugh 1963), repeated driving and extraction of piles, heavy tamping by dropping weights exceeding 10 t from 20 m or more, and by vertical insertion of heavy vibrating units. The dropping weight and vibratory systems are most commonly used because of their economy and the degree of control of compaction achieved. Silts and silty sands may also be compacted by dropping weights, whilst mixed or clayey soils are frequently strengthened (but not significantly compacted) by using vibrating units to form vertical holes at intervals about 1 m to 2 m into which coarse granular materials are packed with the vibrator to replace the clay, thus stiffening the resulting composite ground.

#### 6.3.5.4 *Pre-consolidation of soil by pre-loading the site.*

The stability of foundations can be increased and settlements can be reduced by pre-loading the site with earth or other suitable material. This is particularly applicable to large spread foundations built on soft clays, loose silts or uncompacted fills. Vertical sand drains or wick drains can be used in certain ground conditions to reduce the length of time that the preload has to be applied.



**6.3.5.5 Injection of grouts into the ground.** In appropriate circumstances it is possible to inject most soil and rock formations with grouts to reduce their permeability and improve their strength. There is a wide range of grouts available for injection suitable for differing types of soil or sizes of rock fissures; these are mixtures ranging from cement, cement and sand, cement and clay, clay suspensions and a range of chemical solutions to meet specific requirements for ground treatment.

**6.3.5.6 Construction of in situ diaphragms.** In situ diaphragms can be employed to retain perimeters of excavations and to allow excavation to proceed below groundwater level. Walls may be constructed in situ in the ground by excavating a narrow trench from the ground surface and maintaining it open against the collapse of its walls by keeping it filled with a mud slurry during the process of excavation. Where a retaining structure is required, a reinforcing cage is placed in the slurry-filled trench and concrete is then placed in the trench to displace the slurry as described in 6.5.3. Most commonly, thick reinforced concrete walls are used to retain earth and water pressures during construction. Such walls are usually incorporated as a permanent part of the substructure. In that case they can be designed to carry loads as part of the foundations. The diaphragms are frequently tied by earth or rock anchorages to the retained ground to provide clear working space within the contained excavation.

If a water cut-off is the only requirement, the filling need only be sufficiently impermeable to reduce or eliminate seepage into the excavation and in some cases the filling is required to be plastic to permit movement of the wall without cracking. Such walls can be used to enclose excavations and so act as cofferdams. There are also other techniques for forming thin diaphragms (see 6.5.4).

**6.3.5.7 Electro-osmotic consolidation and electrochemical hardening.** Homogeneous clay soils have been strengthened by reducing their moisture content by electro-osmosis so consolidating them and improving their strength for foundation (Casagrande 1949; Chappell and Burton 1975) In favourable circumstances it has also been possible to induce chemical hardening by electro-osmosis around piles in clay soils to enable them to carry higher loads, whilst introduction of chemical solutions by electro-osmotic means can also have a stabilizing effect (Farmer 1975). However, these processes are at present of very limited application and require particularly favourable soil conditions for successful and economic employment.

## 6.4 Methods of dewatering excavations

### 6.4.1 Introduction

Any scheme for the removal of the groundwater should be based on a thorough site investigation specifically for that purpose, in order that the temporary and permanent work can be designed to use the known conditions to the most economical advantage.

Water may be removed from excavations by gravity drainage or by pumping from sumps, well points or deep wells.

The method adopted will depend upon:

a) soil conditions, such as the permeability of the more porous layers, the sequence of the soil strata and local variations of permeability and pressure within the soil profile;

NOTE For simplicity, Figure 7 to Figure 12 are drawn on the assumption of uniform ground conditions within each stratum.

b) the depth of excavation below groundwater level or relative to piezometric levels in underlying strata;

c) the method of supporting the sides of the excavation, i.e. open or sheeted excavations;

d) the necessity or otherwise of safeguarding existing structures in the vicinity of excavations;

e) the time available to complete the work.

Good practice requires that the following conditions are fulfilled when dewatering excavations.

1) The lowered groundwater level should be kept under full control at all times, to avoid fluctuations which could affect the stability of the excavation.

2) The method adopted should be chosen so that the excavation remains stable at all times, i.e. slips do not occur in the sides of the excavation and excessive heaving of the base does not arise.

3) When the aquifer to be drained consists of a fairly uniformly graded granular material, it can establish itself as a natural filter to prevent loss of ground as a result of the pumping. If this is not the case, or there is a doubt, adequate filters need to be provided around the sumps or wells to ensure that there is no transportation of soils, fine-grained in particular, with the pumped water.

It is necessary to collect pumped water in a tank and to check if any soil settles to the bottom of the container. Such a test should be undertaken on all groundwater lowering installations, particularly in built-up areas and near structures, services, etc., that can be affected by loss of ground, the test being carried out at the beginning of the pumping. If some soil is being removed, an estimate should be made of the ground loss over the period of the works and a decision taken on the adequacy of the filter and measures to be taken to mitigate any undesirable effects.

It is prudent to carry out such checks frequently to ensure that the filters around the well are working properly.

- 4) There should be an adequate margin of pumping capacity standby power and plant should be available in case of breakdown and to facilitate maintenance.
- 5) Water removed by pumps should be discharged well clear of the excavation area in a manner that does not cause erosion, silting or contamination of existing drains and watercourses.
- 6) The pumping methods adopted for groundwater lowering should not lead to damage of adjacent structures.
- 7) Apart from economy in pumping, the water level should not be lowered to depths greater than necessary to keep the excavation clear of water at all times.
- 8) The method adopted should avoid excessive loss of ground by seepage from the sides or base of the excavation.

#### 6.4.2 Gravity drainage

Where site conditions permit, water can be drained by gravity from an excavation; this may be possible on a sloping site where groundwater can be collected in a sump in one corner on the downhill side of the excavation, or a gravity drain can be installed to a discharge point farther down the slope.

Where an excavation overlies either a dry pervious stratum or one containing a head below that of the base of the excavation, it is sometimes possible to drain the water off the excavation through vertical drains sunk below it to connect with the underlying permeable strata.

#### 6.4.3 Pumping from inside the excavation

**6.4.3.1 General.** It has long been the custom to pump from sumps within the main excavation. However, where large quantities of water have to be pumped, it may be more convenient to form the sump just outside the main excavation area. Sinking the sump outside the main excavation will also avoid the risk of damage by erosion of the soil at formation level. Sumps should be dug to the full depth required to drain the main excavation before the latter enters water-bearing ground, and should be maintained in their original form during the whole period of construction. This method will permit installation of any necessary filtering media to prevent loss of ground; the groundwater can be kept below excavation level at all stages of the work, and any difficulties caused by the soil or groundwater can be properly assessed during construction of the sump, so allowing changes to be made in the construction scheme for the main excavation. In all but the smallest excavations it is desirable to have at least two sumps.

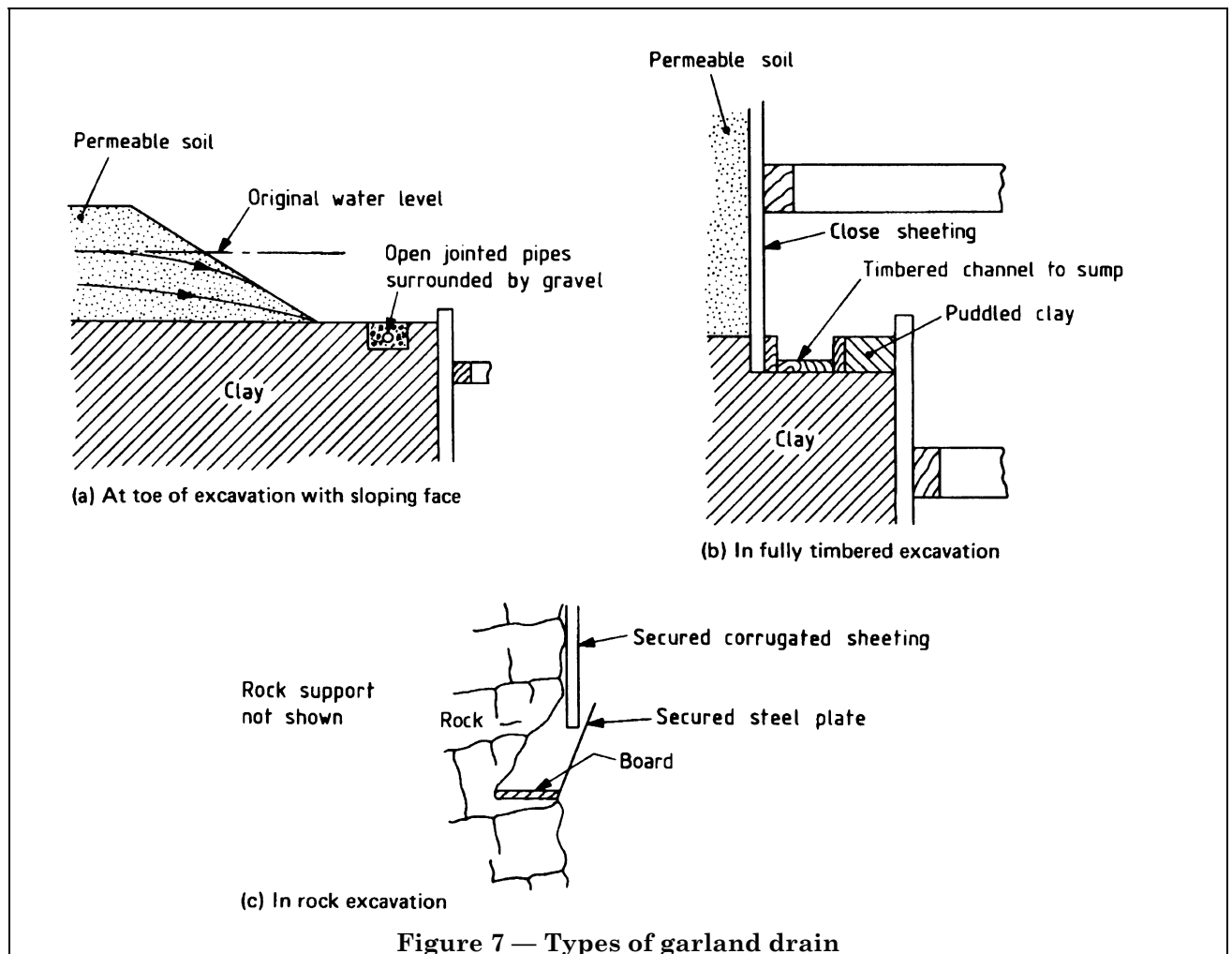
The location of drainage channels leading to the sumps should be a matter for careful consideration in order to ensure that the whole of the area is drained at all stages. The efficient design and maintenance of drainage ditches are particularly important where water seeps down a timbered or sloping face and is intercepted by the ditches. The falls in the ditches should be sufficient to avoid silting up due to soil carried into them, but they should not be so steep that erosion occurs. It is often convenient to pipe the drainage ditches using open-jointed earthenware or perforated plastics pipes surrounded by graded gravel filter material or plastics filter fabric.

The need for a sufficiency of pumping plant in each sump cannot be stressed too highly. Usually a higher pumping capacity is needed to dewater an excavation than is required to maintain the water level at a steady state in its finally lowered position. The pumping plant should be installed in multiple units so that the additional units required to give the increased capacity in pumping down the excavation can be shut down as soon as the required levels are reached, but left in position to act as a standby in case of breakdown.

Where excavation is carried through permeable strata on to impervious ground below and the bedding is horizontal or nearly so, it is difficult to arrange the positions of the sumps to drain the surface of the impervious layer completely.

There will be a risk of water bypassing the sumps, i.e. flowing between them into the main excavation. This trouble can be avoided by stepping in the excavation at the level of the impervious stratum and forming a drainage ditch (garland drain) at this level connected to a sump or sumps (see Figure 7). This method has the advantage of avoiding pumping from an unnecessarily low level and of preventing water collecting in and softening the impervious stratum at the formation level.

Adequate steps should be taken to avoid loss of ground from around the sump. The best method is to insert some filter medium between the ground and the sump. This can be done after the steel piling or timbering to the sumps has reached its final level. A cage of perforated metal is then installed and the space between the cage and the ground is filled with graded gravel filter material, the sheet piling or timbering being withdrawn as the filter material is placed. The method should not be used if upward flow of water from the base of the excavation causes unstable soil conditions. Also if only a thin impermeable stratum exists at the base of the excavation with further permeable beds below, there is a risk of upheaval of the base of the excavation due to the pressure of the water in the deeper permeable beds. In this case, the water pressure beneath the impermeable stratum will need to be relieved. This can be achieved by pumping from wells or well points inside or outside the cofferdam, or by vertical drains sunk from within the excavation.



Safeguards against loss of ground are particularly necessary when pumping from large timbered sumps. Other filtering methods can be used, such as plastics filter fabric, perforated steel piles or plates (the filter being formed by allowing the movement of some fines adjacent to the piles or plates) or a natural filter in certain naturally graded soils. This usually can only be done where the ground is of varying grain size, the majority of the grains being larger than the perforations, even then local disturbances of the surrounding ground may be caused until a natural filter is established.

**6.4.3.2 Pumping from untimbered or timbered excavations.** Where untimbered or timbered excavations are in stable rock formations, the groundwater will seep down the face of the excavations where it can be collected by drainage ditches and led to a sump without trouble arising from instability of the face. However, where faces of excavations are in permeable soil, the velocity of the water seeping into the excavation may be sufficient to cause movement of soil particles followed by collapse of the sides (see Figure 8). To avoid this difficulty, the face of the excavation should be cut back to a stable slope (see Figure 9). The water level is lowered by pumping and the water emerging at or only a small distance above the toe of the slope can be prevented from doing damage by covering the affected area with a graded gravel or stone filter or by plastics filter fabrics; alternatively, the face can be lined with timbering with small gaps between the lagging. If a natural filter does not readily form behind these gaps, then graded filter media, or permeable packing can be placed behind the timbers.

In addition to preventing surface erosion by the provision of a filter, it is also necessary to ensure that the internal seepage forces are not sufficient to endanger the overall stability of the slope.

**6.4.3.3 Pumping from cofferdams.** If a cofferdam can be constructed of jointed sheet piling or in the form of an in situ impermeable diaphragm taken down into a thick impermeable stratum, the flow of water in the overlying impervious ground will be completely cut off and the dewatering of the area enclosed by the cofferdam will be a simple and straightforward matter. Relief of excess hydrostatic pressure in permeable soil layers below the base of the excavation may need to be considered.

If an impermeable stratum cannot be reached economically by the cofferdam sheeting or diaphragm, it is often still possible to dewater the excavation using internal sumps, provided the upward hydraulic gradient can be kept sufficiently low to ensure the stability of the base of the excavation and the toe of the sheet piling. The hydraulic gradient can be reduced by:

- a) lengthening the seepage path by increasing the penetration of the cofferdam sheeting below excavation level;
- b) reducing the head of water flowing towards the excavation by pumping from wells or well points outside the cofferdam;
- c) lengthening the seepage path by using berms within the cofferdam or placing impermeable blankets outside the cofferdam.

When a naturally occurring impermeable stratum does not occur at a depth considered economic to allow the termination of cut-off within it, so forming a completely sealed surround to the excavation, it is necessary to prevent the entry of groundwater through the bottom of the excavation either by pumping or forming a seal across the bottom by injecting suitable grouts into the ground below the excavation.

In such circumstances a check has to be made that uplift pressures against the treated floor to be excavated do not cause failure. Such pressures can be relieved by pumping or maintaining a balancing head against hydrostatic uplift using a sufficient depth of soil above the treated floor.

It should, however, be pointed out that the excavation will not be so stable as it would have been if an external groundwater lowering system were used to remove all the water.

#### **6.4.4 Lowering the groundwater level using sumps, wells or well points outside the excavation**

**6.4.4.1 General.** The object of an external groundwater lowering system is to lower the water table below the level at which work is to be carried out, or to reduce the pressures in underlying pervious layers so that the stability of the excavation is ensured at all times.

This is generally achieved by installing a groundwater lowering system to cause the water to flow out of and away from the excavation, thus stabilizing the sides, rather than allowing it to flow into the excavation with risks of instability of the sides and bottom, and to lower the water level below the level at which the work is to be carried out before the main excavation is commenced.

The methods used for lowering the groundwater level outside an excavation are:

- a) excavated wells or sumps with independent pumps;
- b) a number of small diameter well points (the well point system);
- c) a buried horizontal filter (horizontal well point system);

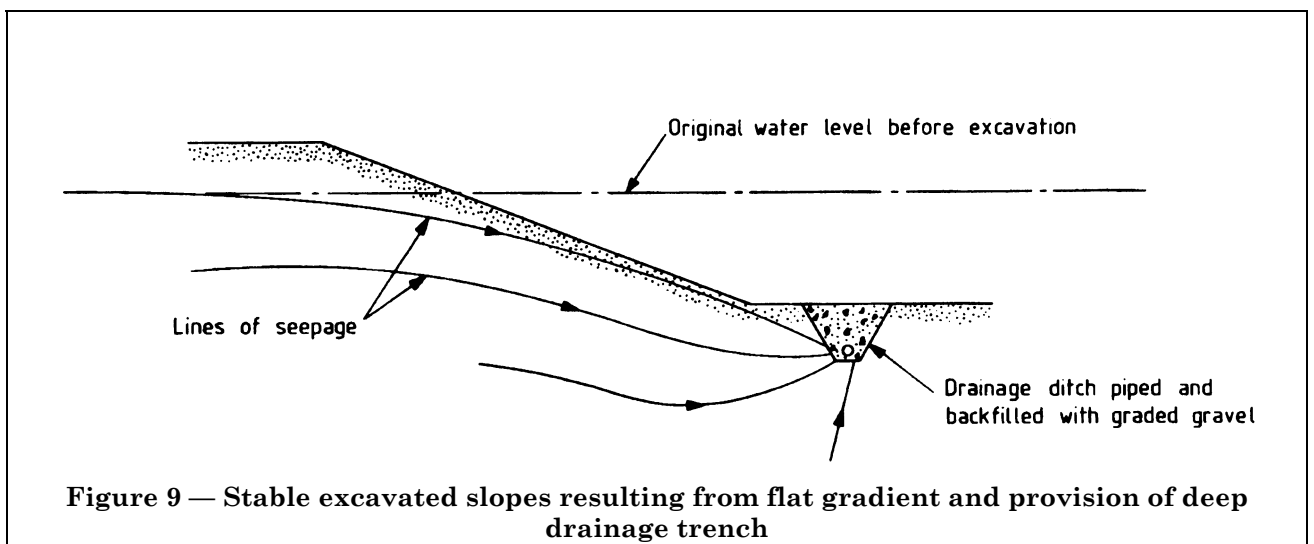
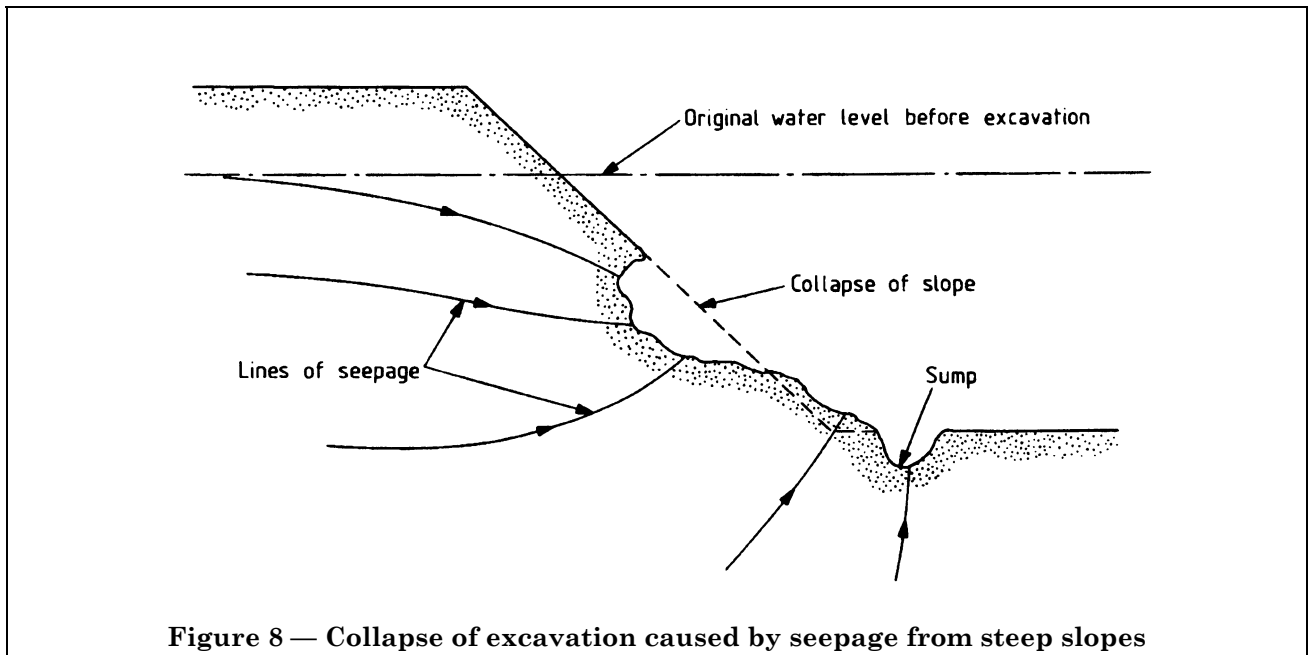
d) multiple bored filter wells with independent or common pumps at the surface (the shallow well system);

e) multiple bored filter wells with independent submersible pumps in each well (the deep well system);

f) multi-stage installations of b) and d);

g) vacuum well systems.

In all methods loss or disturbance of the ground should be prevented by the use of filters.





When the water is pumped from a well, the quantity pumped depends on the level to which the water immediately outside the well screens is lowered, on the radius of the well and on the permeability of the ground. Pumping causes the water table around the well to take the form of an inverted cone known as the cone of depression. When water is pumped simultaneously from a number of wells the cones of depression intersect. The reduction in level of the enclosed water table (see Figure 10) depends upon the spacing and size of the wells as well as the reduction in the water table immediately adjacent to the wells. The fact that the cones of depression of the wells intersect means that the yield of water pumped from any one of the wells is considerably less than that of a single isolated well for the same reduction in water level.

#### 6.4.4.2 Description of groundwater lowering systems

**6.4.4.2.1 Excavated wells or sumps with independent pumps.** One or more sumps outside the main excavation may prove adequate to lower the water table to the desired level.

**6.4.4.2.2 Well point system.** A well point is a suction device used as a small well that can be readily installed in the ground and withdrawn. It usually consists of a long 50 mm diameter steel or plastics pipe with a perforated length at its foot. At the base of the well point there is an orifice that permits water to be forced down the well point into the ground whilst jetting the point into the ground during its installation.

When the well point is in use, water is sucked in through the perforated length and not through the orifice, which is sealed against entry of soil by a simple ball valve. The perforated length is covered with a fine metal gauze or plastics filter fabric to prevent entry of fine soil into the well point.

When the well point is being installed by jetting it into the ground, the water pressure and flow are reduced as it nears its final depth and sand of coarser grading than that in the ground to be dewatered can be poured into the annular space around the well point. This acts as an outer filter round the mesh and also provides a vertical drain around the well point so facilitating downward drainage. The jetting water is then cut off and the riser pipe joining the well point to the surface is connected to a ring main which is in turn connected to the pump which should maintain a strong and regular vacuum. In some gravels and sands this procedure of forming a surrounding filter to the well point is not necessary.

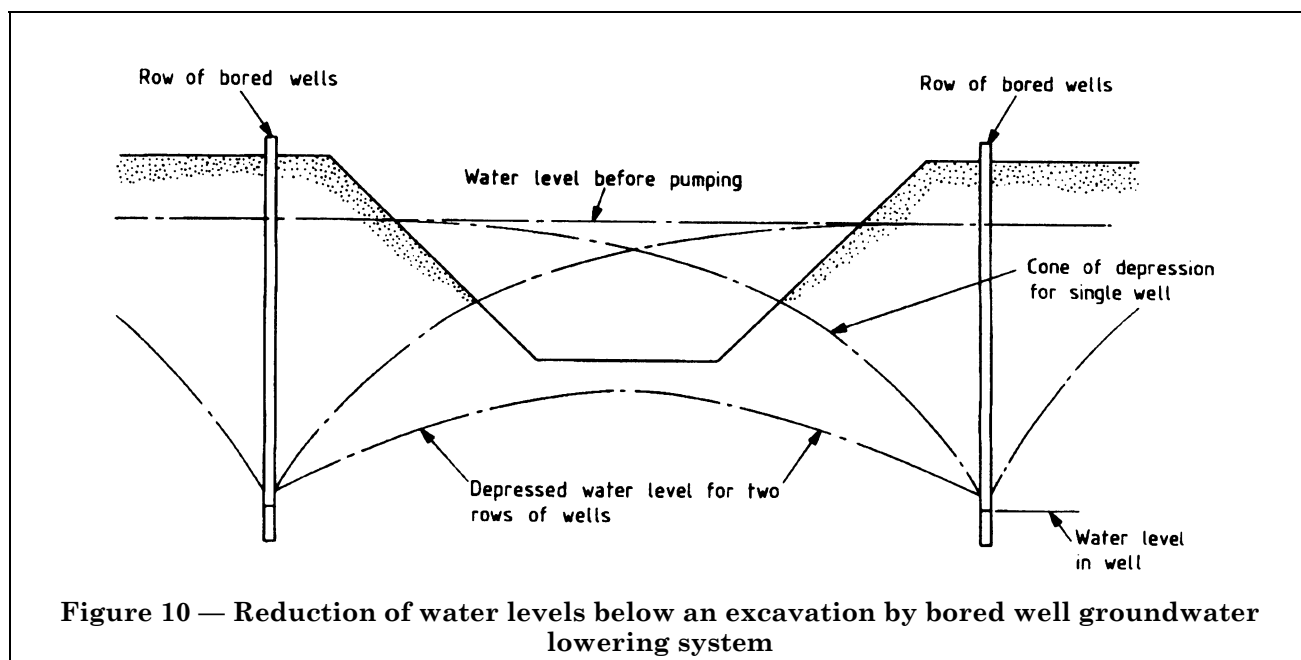
The spacing of well points depends on the permeability of the soil. Highly permeable soils require close spacing, e.g. 0.5 m, whereas wider spacing, e.g. 0.75 m to 1.5 m, can be used for fine sands or silty sands.

In the horizontal well point system a tractor-drawn chainbucket excavator is used to cut a narrow vertical slot in the ground to the required depth which can be up to 6 m. Flexible perforated plastics pipe is fed into the base of the slot through a guide at the rear of the excavator to form the horizontal filter drain. Vertical connections are made at intervals from the drain to the vacuum pump. This system is suitable for lowering the groundwater for long trench excavations, or for surrounding large excavations, but the horizontal filter drains are not recoverable.

Because of small air leaks in pipe connections and air being drawn into the system through the well points themselves it is not possible to maintain a perfect vacuum. Consequently, for all practical purposes, the maximum depth of lowering of the water table is about 5 m below the level of the pump suction inlet, although a somewhat deeper lowering may be achieved in exceptionally favourable conditions.

The well point system has the advantage of low capital costs. It is quickly installed and can readily be moved from one position to another. It is particularly suitable for trench work and, for narrow trenches, a single row of well points along one side of the trench is often sufficient; for wider trenches, two parallel rows are used. The well points are jetted down before the trench excavation is commenced and are withdrawn after backfilling has been completed. The operations of jetting down before excavating, pumping from the well points and withdrawal after or during backfilling move progressively along the trench as the excavation proceeds.

The disadvantages of the vertical and horizontal well point systems are the limited depth of lowering of the water table, requiring two or more stages for deep excavations, the difficulty in jetting or driving down well points in ground containing cobbles or boulders, the lack of control in placing the filter around the well points and the disturbance of the ground which may occur during jetting the well points. If the filters surrounding the wells are not properly formed, layers of an impervious soil may prevent water from a water-bearing pervious stratum not directly connected to the filter of the wells from reaching the wells.



**6.4.4.2.3 Shallow well system (multiple bored) filter wells with a common pump.** Bored wells, mainly of diameter 300 mm or upwards, are bored to a depth, usually 10 m below the axis of the pump, in order to ensure that the suction is always submerged. Mesh or filter fabric covered filter tubes of diameter 150 mm are lowered into the borehole. The lowest tube is unperforated over its last few metres and is provided with a blank end to form a sump to trap fine soil particles which may initially pass through the filter. Filter gravel graded to suit the particle size distribution of the ground is carefully placed in the space between the filter tube and the bored casing as the latter is gradually withdrawn. In this way, the filter gravel surrounds the filter tube for the full depth of the borehole over which the groundwater is to be lowered. After flushing the filters and removing any fine material and debris from the sump, the suction main is lowered down the filter tube and connected to the header main, which is in turn connected to a self-priming pump capable of maintaining a strong and regular vacuum, or individual wells can each have their own self-priming pump.

In the eductor well system, the wells consist of a riser pipe attached to a parallel vertical pressure pipe. Water under pressure flows down the pressure pipe from a ring main connecting all wells, to a venturi orifice at the base of the riser pipe. The reduction in pressure caused by flow through the orifice draws water from the surrounding ground through a mesh filter and up the riser pipe to a header main laid parallel to the pressure ring main, and thence to the discharge point clear of the excavation.

The shallow well system has the advantage that obstructions in the ground can be removed during boring for the filter tubes; the nature of the ground is fully explored while boring is taking place and any modifications to the filter arrangements or well-spacing can be made. As the wells are fewer and more widely spaced than the well point system, there is less general disturbance of the ground. However, the installation for multiple wells is more costly and takes longer to install than well pointing. The depth of lowering of the water table is somewhat deeper than with well pointing, but it is still limited by the suction lift of the pumps and multi-stage installations are needed for deep excavations as described in 6.4.4.2.5.

**6.4.4.2.4 Deep well system.** This system is primarily devised for use in connection with deep excavations and is of special value where artesian water is present below an impermeable stratum, even when the latter is at a considerable depth below formation level.

Wells of a diameter large enough to accommodate submersible pumps are bored to the required depth and filter tubes surrounded by single- or multi-layer gravel filters are installed as described in 6.4.4.2.3. Each well is provided with its own pump.

The filtering length of the well tube may consist of a slotted screen, perforated casing or a proprietary well screen. These screens have saw-cut, bridge or louvered slots in the casing or they may consist of wire wound in a spiral form on to a cylinder formed by longitudinal rods. The width between the wires or the width of the slotted openings is selected to conform to the particle size distribution of the surrounding ground, thus acting as a filter to prevent fine material from being drawn into the well. In favourable ground conditions a proprietary slotted well screen need not be surrounded by a gravel filter.

**6.4.4.2.5 Multi-stage installations.** If the excavation is to be taken to a greater depth than the level of the water table as lowered by a well point system or by a shallow well system using pumps at ground level, one or more deeper stages can be installed to achieve the desired lowering of the water table, as shown in Figure 11.

In marginal cases, the effective lowering by the first stage can be increased by excavating down to the level of the natural water table before installation of the system, as shown in Figure 12.

**6.4.4.2.6 Vacuum wells.** The systems described in 6.4.4.2.1 to 6.4.4.2.5 are only effective in gravels and sands. When more than about 10 % of silt is present, it is usually necessary to increase artificially the flow towards the wells. This can be done in bored wells having independent submersible pumps by applying a vacuum inside the perforated section, care being taken to seal the top of the well against entry of air into the well installations. When well points are used as vacuum wells, the sand filter around the top of the well point riser should be sealed with clay.

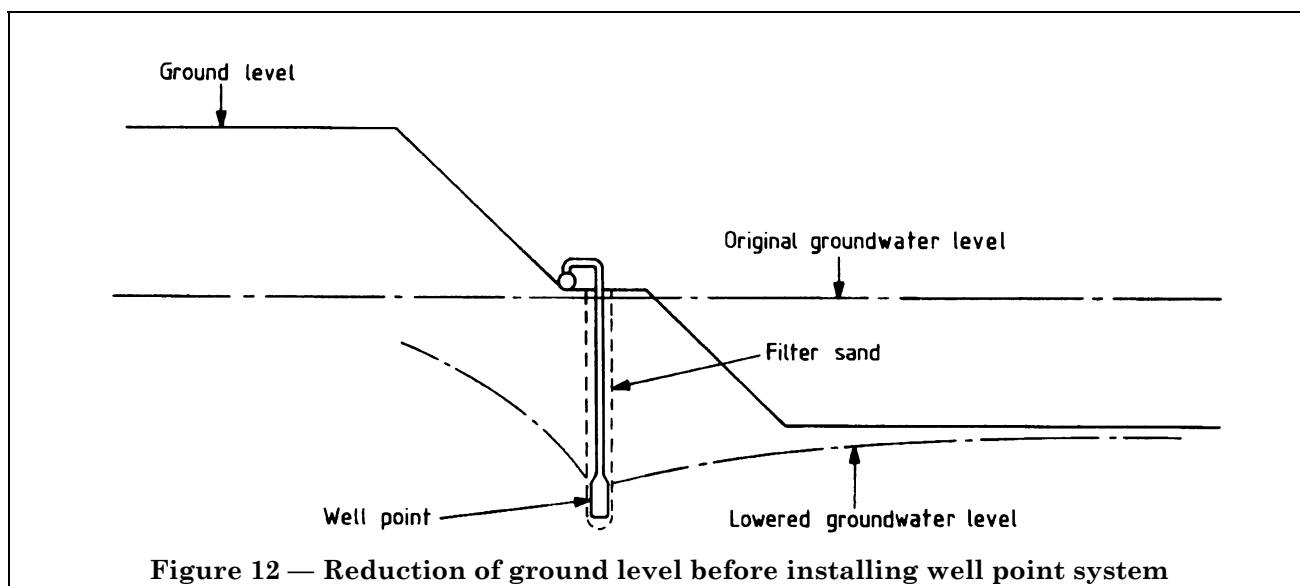
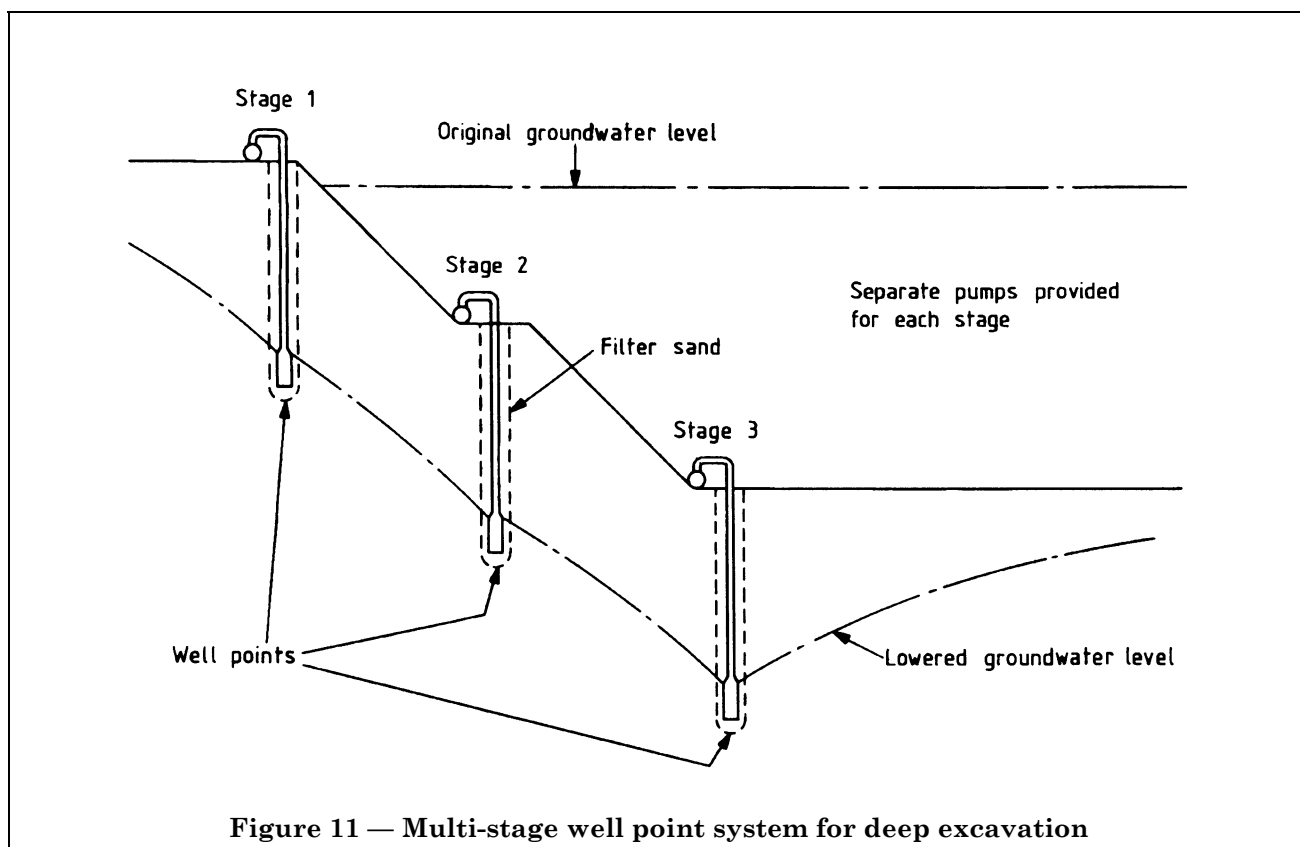
**6.4.4.2.7 Electro-osmosis.** An alternative method is to use an electrical potential to drive the water to negative electrodes at the wells, using expendable metal rods as the positive electrodes (Casagrande 1949).

Voltages of 40 V to 180 V have been used with the electrodes spaced 4.5 m apart. D.C. current of the order of 15 A to 25 A per well is required, and the process can be very expensive. The most successful applications of this technique have been made in uniform beds of fine silt.

**6.4.4.3 Precautions.** Duplication of pumping plant should be provided for the well point, shallow well or deep well systems in all cases where breakdown of the plant would cause costly delays in construction or damage to partially completed work or danger to operatives or the public. Where feasible it is useful to stop dewatering for a short period, under careful supervision and control, after the stable condition has been reached and before excavation is started in order to gain useful information on the speed at which the depressed water table would rise should a breakdown in the system occur. As previously stated, the reserve pump capacity can be usefully employed in the initial stages of lowering the groundwater table when the quantity of water to be pumped is a maximum. Where electrical pumps are used, it is a wise precaution to provide an alternative source of power.

Regular checks should be made on the level of the groundwater table around the excavation and on the water pressures given by piezometers installed at various positions in the water-bearing strata which affect the stability of the excavation or partially completed structure.

When allowing the groundwater to return to its original level, care should be taken to do it as slowly as possible by progressively shutting down wells or well points. A rapid shutdown may cause a major collapse of the sides of a deep excavation or result in the collapse of soils having a sensitive structure, e.g. loose sands.



When constructing basements, tanks or other hollow foundations in deep water, flotation of the structure may occur in the event of pump breakdown, and it is desirable to provide drainage facilities through the substructure to allow balancing water to enter as the groundwater rises, thus preventing excessive external pressures or flotation.

#### 6.4.4.4 *Settlements during groundwater lowering.*

Where partially saturated or saturated compressible soil, not yet fully consolidated, overlies an aquifer to be dewatered, then the removal of the water will accelerate the natural consolidation process of the overlying compressible soils. This consolidation may or may not be uniform and may cause settlements of any structure supported in such ground.

Damage to such structures is not by any means inevitable. It is prudent to undertake a survey of any structures that could be affected by the removal of groundwater to see their condition before dewatering operations.

**6.4.4.5 *Design of filters.*** Filters used to surround the inner tubes of shallow or deep well pumping installations, pumping sumps or piped drains, or used as blankets to control seepage, should be designed to conform to the grading of the soil surrounding the filters. Design rules commonly used are as follows.

- a) The 15 % size of the filter should be not greater than 4 times the 85 % size of the natural soil surrounding the filter (the “protected material”), as shown by point A in Figure 13.
- b) The 15 % size of the filter should be not less than 4 times the 15 % size of the protected material (point B in Figure 13).
- c) Filters should not contain more than 5 % material passing through a 75  $\mu\text{m}$  sieve complying with BS 410 (point C in Figure 13) and such material should be cohesionless. Where the size given by rule b) is less than that given by rule c), the latter should apply as in Figure 13.
- d) The grading curve of the filter material should roughly follow the same shape as the grading curve of the protected material.
- e) The 50 % size of the filter should not exceed 25 times the 50 % size of the protected material (point E in Figure 13).
- f) Where the protected material contains a large proportion of gravel or coarser material, the filter should be designed on the basis of the grading of that proportion of the protected material finer than the 19 mm sieve.

g) Where the soil is gap graded (e.g. a silty fine sand with some gravel) the coarser particles cannot prevent the fine particles from migrating through the large pore spaces in the filter, if the latter is designed on the complete grading curve of the soil. Therefore, the coarse particles should be ignored and the grading limits for the filter should be selected on the grading curve of the finer soil.

h) Where a filter is placed against a variable soil, the filter has to be designed to protect the finest soil. Generally, rule a) should be applied to the finest soil and rule b) to the coarsest.

i) Filter material should be well graded within the limits permitted by the range of particle size to avoid segregation when placing.

j) The maximum size of the filter material should be not more than about 80 mm.

k) Where pipes are embedded in filters and drains, the filter material in contact with the pipe is not to be so fine that it enters the joints or perforations in the pipes. A commonly used criterion is that the 85 % size of the filter should be not less than twice the maximum opening in the pipe or joint (point D in Figure 13).

l) In fine-grain soils multi-stage filters are sometimes necessary to ensure stability of the filter against ingress of such soils. The above rules should be applied for their design.

In practice the engineer may use his judgement to adopt filters of a grading somewhat coarser than that given by the rules. The use of coarser graded filters results in some loss of soil through them during the early stages of draining until the packing of the soil particles adjusts itself to act as a combined filter. Whilst the filter has to be effective, it should not be so fine as to prevent the ingress of water into the well.

## 6.5 Special methods for excluding water from excavations

### 6.5.1 Use of compressed air

**6.5.1.1 *General.*** When an excavation in water-bearing ground can be undertaken in a confined chamber, it is possible by increasing the air pressure within the chamber to remove or exclude the water from the workings by forcing it away through the face of ground exposed.

This method can be used in any confined chamber, e.g. tunnels, shafts, caissons and cofferdams, provided that the design allows for the retention of the air and that the chamber is capable of withstanding the applied pressures whilst allowing air to escape only against the face requiring support.



The use of compressed air is a traditional method of undertaking underground construction work, particularly where the soil conditions are difficult. It has been of particular value in the construction of tunnels and caissons, and the sinking of shafts of large diameter to form bridge and pier foundations for heavily loaded structures.

In certain conditions the use of compressed air is a more suitable means of removing or excluding water from the ground during construction work than other geotechnical processes. It is especially suitable for work in fine silts or soft clays when these are difficult or expensive to treat by injection processes or are too fine to yield their groundwater readily. This is particularly the case when the extent of the work can justify the expense of the initial installation of the compressed air equipment. It can also be used when excavating through water-bearing rock, particularly when there is too much water present to be dealt with by pumping.

There are some cases where the use of compressed air may be combined with other techniques; for example lowering the groundwater in the surrounding ground may enable air pressures to be reduced, or injection treatment of the ground ahead of the excavation may reduce air losses from the face or permit lower air pressures to be used.

The use of compressed air has its limitations. In particular, as the air pressure is raised to balance increasing water head, the health risk to men working in the compressed-air chamber increases. Special precautions always need to be exercised by rigid control of decompression times and medical supervision of the workers to prevent damage to health.

Reference should be made to published work on the health hazards of work in compressed air, e.g. The Factories Act 1961; CIRIA Report 44, which draw attention to the incidence of “bends” and the other more severe forms of decompression sickness such as paralysis or loss of consciousness (see 11.3).

**6.5.1.2 Regulations applicable to work in compressed air.** For regulations applicable to work in compressed air see 11.3.1.2.

**6.5.1.3 Design and working of compressed air installations**

NOTE Attention is drawn to Regulation 7 of Statutory Instrument 1958 No. 61 (see Appendix D).

The design and operation of compressed air installations should be undertaken under the supervision of an engineer qualified and experienced in the use of compressed air for civil engineering work (see 11.3.1.3).

**6.5.1.4 influence of ground conditions on use of compressed air.** It is imperative that a competent and thorough investigation be undertaken of the ground conditions influencing the use of compressed air and through the zone where construction is to proceed.

In the case of tunnels, the nature of the stratification above invert level will need to be carefully established and related to that of the engineering geology in the vicinity of the work. Accurate knowledge of the water conditions is essential, as these dictate the air pressure to be used.

Interconnection of the strata and presence of man-made weaknesses in the ground will have to be ascertained, as the air under pressure will endeavour to escape through such zones of weakness; if these are connected to residual air at lower pressures or the atmosphere, large losses of air can occur over a very short period, with serious consequences.

There will need to be sufficient cover of ground above the tunnel to balance the exhaust pressure from the excavation. This is particularly important when tunnelling is being done below or adjacent to free water. The presence of buried channels, old piles and foundations can produce difficulties due to sudden loss of air. The gradual escape of air from a working area may cause erosion in the overlying soil and set up passages of weakness in the soil, leading to a greater loss of air to the surface.

Consideration should be given to the effect of ground cover on the stability of compressed air workings. For example, if water is encountered under higher pressure than that of the soil and water above the roof of the workings, then an increase in air pressure to counterbalance the increased water inflow may cause a “blow” in the roof of the workings or uplift in a caisson. Loading of the ground over the workings by stone, soil or kentledge may be considered as a precaution against a “blow”.

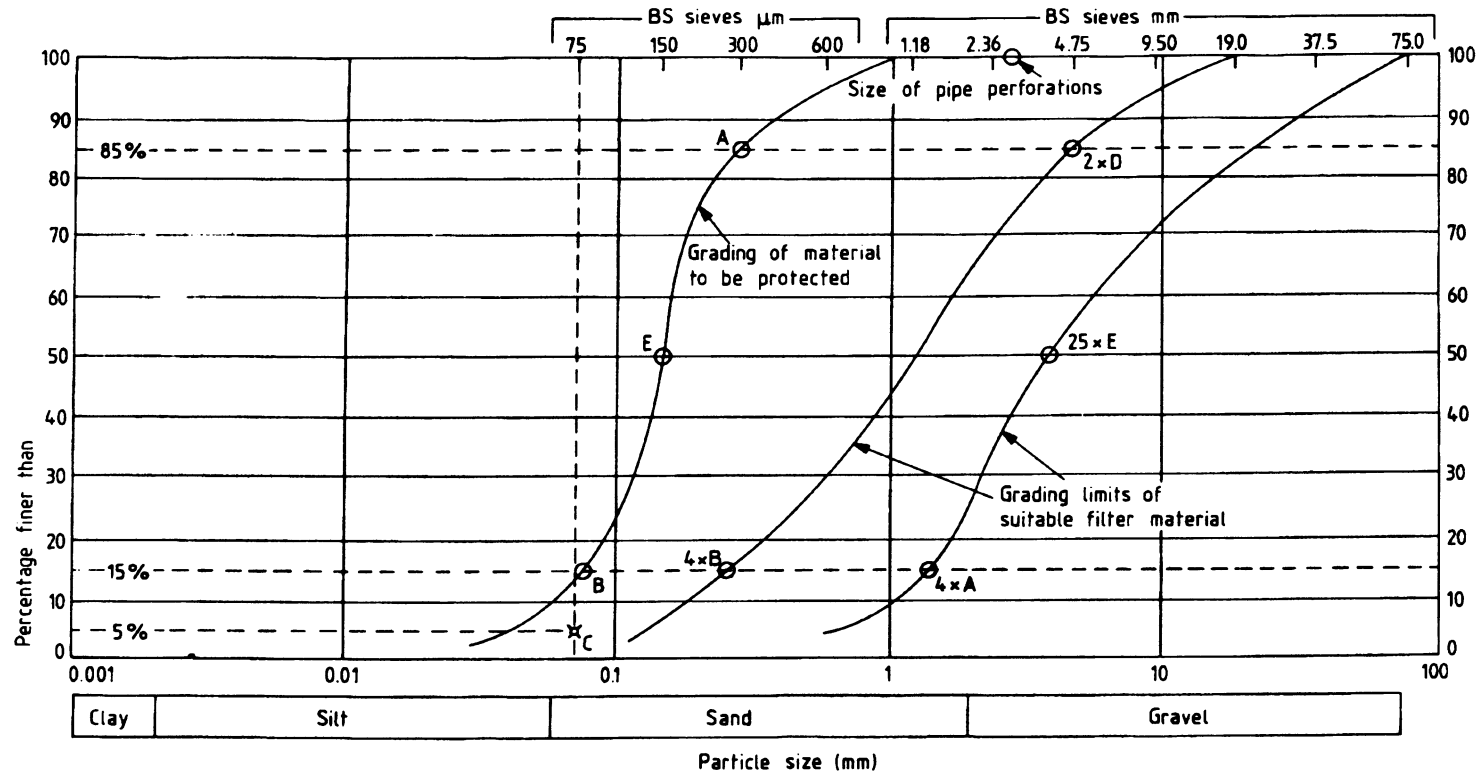


Figure 13 — Design rules for filters

**6.5.1.5 Use of compressed air in conjunction with other geotechnical processes.** In certain cases the use of compressed air is a more certain and convenient method of excluding water from excavations than the various geotechnical processes described in 6.4, 6.5.2, 6.5.3 and 6.7. However, there can be advantages in using compressed air in conjunction with these processes. The additional cost may be justified if the cost of the compressed air work can be correspondingly reduced, or if compressed air work is thereby made feasible in ground conditions where the use of compressed air alone exceeds certain limiting conditions.

The limiting conditions for the use of compressed air by itself are:

- a) the maximum working pressure that is economical while ensuring for the safety and health of the workmen;
- b) the estimated air losses through the working face, which will vary with the area of exposed ground and the permeability of the ground through which the air attempts to escape.

If air pressure can be reduced, there will be an improvement in working conditions and reduction in air losses. Consideration should therefore be given to methods of lowering the air pressure required by such expedients as reducing the groundwater pressure at the excavated face by means of:

- a) a suitable groundwater installation;
- b) reducing the inflow of water by treating the ground to form diaphragms around the excavation;
- c) in situ wall techniques (see 6.5.3).

When air can escape through pervious soils of high permeability such as gravel, these can be treated by injection to reduce the air losses. When excavating shafts and tunnels through difficult ground, protection can be given to the workings by forming a partial or complete peripheral surround to the working face by injecting suitable grouts into the ground.

**6.5.1.6 General precautions.** The use of compressed air to facilitate excavation work through difficult ground is a well tried expedient but, as has to be repeatedly emphasized, is one which is only to be undertaken under the supervision of experienced persons, as failure to understand the problems involved and inexperience in the use of the plant required can lead to disastrous consequences.

### **6.5.2 Excluding water from excavations by freezing the surrounding ground**

**6.5.2.1 General.** Freezing the water contained in the ground so as to interpose a continuous impermeable ice wall between the water-bearing ground and the foundation can provide a useful alternative to the use of compressed air or sheet piling, especially when these techniques are being carried close to the limits of safe or feasible operation (Maishman 1975). The advantage of freezing techniques increases with the depth of the excavation, and it is particularly useful for sinking shafts through deep water-bearing ground. Freezing temporarily increases the strength of nearly all soils, especially those which are water-bearing. For example, saturated sands are as strong as weak concrete when frozen, but clays, although rendered harder by freezing, may retain some of their plasticity. Because of the increased strength of the frozen soil, it is usually possible to make the excavation without temporary lining, although it is generally advisable to insulate the exposed ice wall at shallow levels, which are subject to the influence of atmospheric temperature.

Groundwater may be drawn into a frozen zone where layered clays and silts occur causing expansion of the soil structure. This leads to a reduction in soil strength and an increase in compressibility on thawing.

**6.5.2.2 Description of method.** The excavation is first surrounded by a series of parallel boreholes penetrating into impervious or self-supporting ground below the strata to be frozen. It is particularly important that these borings should be accurately aligned in a parallel direction, otherwise a local weakening or gaps will be left in the ice wall. Groundwater passing through such gaps will thaw the ice, thus enlarging the gaps and causing rapid deterioration in the stability of the ground. Freezing tubes with a closed bottom, containing a tube of smaller diameter, are introduced into each borehole. The outer and inner tubes are independently coupled at ground level to two ring mains connected to the refrigeration plant. Cooled non-freezing liquid such as brine is pumped into the inner tubes to the bottom of the larger tube, whence it ascends through the annular space, extracting heat from the ground during its passage and returning through the collector mains to be re-cooled in the plant. A cylinder of ice is gradually formed around each freezing tube until an unbroken wall of frozen ground is formed around the excavation. Excavation is then carried out inside the ice wall, care being taken to guard against the risk of collapse of the upper edges of the excavation, until a permanent lining is formed within the excavation. On completion of the permanent work essential to the stability of the excavation, either the ground is allowed to thaw naturally or thawing is assisted by the circulation of hot brine through the tubes.

As an alternative to the brine method of freezing the ground liquid nitrogen can be used. This coolant is circulated through pipes installed by probes driven into the face of the excavation and the gas is exhausted to the atmosphere.

This method is advantageous where rapid freezing is required.

**6.5.2.3 Precautions.** All materials that are normally used for permanent construction can be successfully employed in frozen ground. When concreting in frozen ground, the aggregates and the mixing water may need to be preheated.

It is desirable to dry compressed air lines delivered to pneumatic tools.

Standby machines should always be provided. It is recommended that cooling tanks be insulated to conserve cold should a breakdown occur. Most of the coolants used in freezing plant dissolve ice, and it is therefore essential that the joints in the freezing tubes be watertight in order to prevent holes being formed in the ice wall as a result of the leakage of such coolants.

Men working in excavations in frozen ground should be provided with adequate warm clothing and leather boots. After use, suitable footwear needs to be stored overnight in a warm place in order that it is sufficiently pliable for use the next day.

Where grouting is being undertaken behind the lining of shafts previously excavated in frozen ground, or when concreting is being undertaken after shutting down the freezing plant, it is desirable to monitor the temperature of the ground during the stage of thawing to determine the most favourable time for injection of grout and for any regrouting operations, or for the placing of concrete. The time for commencement of these operations should take into account the possibility of subsidence of the ground which has heaved during the freezing stage and the presence of voids in the ground caused by the thawing of lenses of ice.

The effects of saline groundwater on the creation and closure of an ice wall and on the strength of the wall should be taken into account.

The use of liquid nitrogen for ground freezing may involve a risk of asphyxiation when using the process in a confined space. The very low temperatures also have a deleterious effect on some constructional materials and on human skin.

When ground freezing is being undertaken close to existing structures, the levels of these structures and adjacent ground surfaces should be monitored at frequent intervals to ensure that the appropriate action is taken in case damaging movements resulting in frost heave become evident.

### **6.5.3 Cast-in-situ diaphragm walls**

**6.5.3.1 General.** Water can be prevented from entering excavations by surrounding them with a relatively impermeable barrier that is installed by an in situ process. The barrier may consist of a reinforced concrete wall where it is required to perform a structural function such as providing a permanent retaining wall to a basement or other underground structure (see 4.3).

Alternatively, the barrier may consist of a flexible wall provided for the main purpose of excluding water, or where a non-rigid wall is required (e.g. for seismic protection).

If possible the barrier should be taken down to an impermeable stratum to provide an exclusion to the entry of water. Where an impermeable stratum lies at too great a depth for economic construction, water seeping beneath the barrier and upwards from the base of the excavation can be removed by pumping. Alternatively, in favourable soil conditions, the base of the excavation can be sealed against the entry of water by forming a plug of grouted soil below foundation level.

**6.5.3.2 Cast-in-situ diaphragm walls used as retaining walls.** Diaphragm walls can be used economically in situations where they combine the functions of a barrier to water entering the excavation and a permanent retaining wall to an underground structure. In certain cases diaphragm walls may act as a support to vertical loading. The first stage consists of constructing a pair of guide walls along the line of the future trench in which the diaphragm wall is to be constructed. The guide walls generally should have a minimum thickness of 200 mm and a depth of not less than 1.5 m. The distance between the parallel guide walls should be about 100 mm greater than the design width of the permanent wall, and particular attention to this should be paid when the wall alignment is curved in order to avoid catching the grab. The function of the guide walls is to serve as a template for the trench excavation and to contain the bentonite slurry used to support the side of the trench. To ensure stability of the trench sides the hydrostatic pressure of the slurry has to exceed the lateral pressure of the soil and ground water. The slurry should always be maintained at a minimum height of 1 m above the highest ground water level. Where it is necessary to minimize the settlement of the ground surface adjacent to the trench excavation, the head of bentonite needs to be at least 1 m above ground water level to be effective. This may necessitate raising the elevation of the top of the guide walls above the original ground level. The depth of the guide walls should be sufficient to prevent loss of bentonite slurry into any existing cellars or drainage systems. The guide walls should also be strong enough to withstand the weight of any vertical loads acting during wall installation, e.g. tremie pipes. Excavation for the trench is performed by grabbing with rope-suspended clam shells or by grabs mounted on telescopic masts (purpose-made Kelly bars). Chiselling may be necessary for excavating through hard deposits or to form a key trench into a rock formation. Special rotating cutting machines can also be used to advance walls through certain types of rock.

Excavation for the trench is performed in alternate panels each about 4 m to 6 m long, depending upon the length of the grab. Throughout the excavation process the walls of the trench are supported by a bentonite slurry that is circulated by pumping from the trench to remove the contaminated and diluted slurry. The latter is fed through cleaning and reconditioning tanks before it is returned to the trench. Alternatively, the slurry can be pumped to waste and replaced by a freshly mixed material. It is essential that there should be a sufficient reservoir of slurry available to top up the level of the slurry in the trench should there be a loss of head in the drainage. It is important to maintain verticality in the excavation within a permitted tolerance required for the satisfactory completion of the work, and this requires careful control by measurement as the grab works downwards.

The stage of placing the concrete in the completed trench is preceded by removing all solid material which has settled at the base of the excavation when the grabbing may be supplemented by an airlift pump. The contaminated slurry is then removed and replaced by a clean slurry. It is essential for the satisfactory placing of concrete to maintain sufficient difference between the density of the slurry and the concrete to effect the process. A reinforcement cage that has been prefabricated at ground level is lowered into the trench and suspended from the guide walls. The cage can incorporate boxing-out arrangements to form recesses in the wall with provision for bending-out bars for subsequent connections to other structural works. The spacing of the bars should permit easy flow of concrete between the bars and into the space between the bottom of the cage and the base of the trench. The cage dimensions should be such as to ensure adequate clearance between the cage and the sides of the trench.

Concrete mixes should be designed to displace the slurry and any sediment remaining at the bottom of the trench. Slumps in the range of 150 mm to 200 mm are desirable and they should be checked at frequent intervals during the placing operation. The concrete is placed through a tremie pipe in one continuous operation to avoid horizontal joints. The rate of supply of concrete and the time of its setting should be considered so that an integral number of panels can be constructed within the working period. Plasticizers may be beneficial in reducing the water content of the mix.



Keyed joints are provided between adjacent panels. These are formed by appropriately shaped stop ends placed at the ends of each excavated length of trench. These joints are usually watertight but minor seepage through leaking joints can be dealt with by grouting or may even be tolerable in certain classes of structure. Wherever possible re-entrant corners to retaining walls should be avoided because panel joints at the corners are susceptible to opening and leakage. Methods exist to provide steel shear or tensile connections between panels, but their implementation is onerous and costly.

Post-tensioned precast concrete slabs can be used to form retaining walls as an alternative to in situ concrete construction. It is also possible to construct rigid walls in T- or Y-shaped trenches to form counterforts or buttresses to retaining walls.

Clutching arrangements between panels can be provided for structural continuity.

Control of the properties of the bentonite slurry used in diaphragm wall construction is described in 6.5.3.8.

**6.5.3.3 Thick cast-in-situ diaphragm walls used only as impermeable barriers.** Diaphragm walls used only to exclude water from excavations or as a cut-off to prevent water seeping through or beneath a dam can be constructed by trenching methods in conjunction with support by a bentonite slurry or another form of clay suspension.

Structural rigidity is not required, support to the flow reducing barrier being provided by the natural soil or fill (Little 1975).

The trench is excavated by grabbing as described in 6.5.3.2 but traversing percussive or rotary drills with reversed circulation of the slurry to remove cuttings can be used for deep trench excavations. In the softer rocks a milling type of cutter can be used.

Where space is available, a long continuous trench to form an impermeable cut-off beneath a dam or surrounding a large excavation can be excavated by a dragline bucket, special care being taken to avoid collapse of the trench during excavation and concreting. This method of trenching using a dragline bucket is appropriate for temporary works. The excavation may be backfilled with conventional concrete (including reinforcement, if required) or with so called plastic concrete (a low cement content concrete with some bentonite added); cement with a retarder is sometimes added to the slurry to form a semi-rigid clay-cement diaphragm. Erosion can be prevented by attention to the grading of the aggregates and by adding a small proportion of cement to the clay-aggregate mixture to impart some strength without impairing the ability of the fill material to yield plastically.

Reference should be made to 6.5.3.8 for information on the control of the properties of the bentonite slurry used for the trench support.

**6.5.3.4 Thin cast-in-situ diaphragm walls used as impermeable barriers.** There are several methods of mechanically forming a thin “slot” in the ground and filling it with suitable impermeable material to form a diaphragm. In one process a slot is made in the ground by means of driving a steel H or box section to the required depth for forming the diaphragm. A slurry of cement, clay or chemicals is pumped down a pipe or channel to the bottom of the steel section while the latter is being withdrawn, thus filling the slot with an impermeable material. In another process thin plastics sheeting is fed into a narrow slot formed by a rotating cutter, the slot being supported by a bentonite slurry.

**6.5.3.5 Contiguous bored piles.** Bored piles are constructed in single or double rows on a line in plan as close as possible to each other. Retaining walls constructed from contiguous bored piles can be used as a form of permanent lining and a facing wall can be constructed as a second stage operation to provide the required standard of finish. The retaining walls are conveniently constructed in conjunction with support by ground anchors to give clear working space within the excavation.

The diameter of the piles can vary from, for example, 375 mm upwards, depending on the resistance that they have to provide against hydrostatic and earth pressure when they are exposed during the excavation. It may be possible to seal any gaps between the piles by injecting grouts behind them but care should be taken to avoid build-up of pressure on one face of the wall being treated.

**6.5.3.6 Secant bored piles.** Bored piles are constructed in a single row on a line in plan; adjoining piles are interlocked to prevent the entry of water into the excavation. Interlocking is achieved by first installing the piles in alternate positions, followed by boring for the piles in the intermediate spaces between the first stage piling. A boring tool is used to cut grooves down the sides of these first stage piles, and after completion of drilling each intermediate pile, reinforcement is fixed and concrete is placed to fill the borehole including the grooves cut in the first stage piles. The diameter of the secant piles can vary from 500 mm upwards, depending on the resistance that they have to provide against hydrostatic and active earth pressures on the sides of the excavation supported by the interlocked piles. Secant bored piles can be used as a permanent retaining wall with or without support by ground anchors as described in 6.5.3.2.

**6.5.3.7 Mix-in-place walls.** The soil on the line of the wall is mixed in place with additives such as cement, clay or chemicals during the process of sinking a series of holes close to one another, using the drilling tool to mix soils and additives. In some methods the mixing is achieved by pumping a stabilizing slurry through the drilling bit as the latter is withdrawn on completion of the boring; in others an adaptation of an auger is used.

Generally, cementitious materials are used for stabilizing sandy soils, whilst soft clays with high moisture content can be stiffened with quicklime. Contiguous columns to a depth of 10 m or more form stable or impervious blocks of treated soil, so permitting safe excavation or bearing for foundations (Broms and Boman 1976).

**6.5.3.8 Preparation and control of bentonite and other clay slurries used in diaphragm wall construction**

**6.5.3.8.1 General.** Bentonite is a naturally-occurring or manufactured sodium montmorillonite. When wetted the mineral swells to form a suspension that is fluid when stirred, but thixotropic action causes the fluid to gel when the suspension is at rest. If this action does not occur, impurities may have been included in the materials making up the suspension. For further information on preparing suspensions for use in diaphragm wall construction, reference should be made to ICE (1978).

When the suspension is placed against a permeable surface, a cake of hydrated bentonite particles will be formed. This acts as a barrier to further loss of water through the surface, unless the ground is too open to enable the bentonite to form a seal.

The viscosity of the fluid enabling soil particles to be held and transported in the suspension, its thixotropic properties, and the ability to form a filter cake make a bentonite slurry uniquely suitable for supporting deep excavations in unstable and water-bearing soils. A bentonite slurry can be used for ground support in bored pile installations (7.4.5).

It is possible to form slurries from clays other than sodium bentonite but their properties to form a gel or filter cake are inferior to those of sodium bentonite. Types of clay other than bentonite may be required if the mineral content of the available mixing water causes the suspension to flocculate with consequent fluid loss and loss of thixotropy. For example, a suspension of attapulgitite is used if fresh water is not available for mixing and sea water is used.

It is essential to control the preparation, storage, handling and cleaning of the bentonite slurry if its particular properties are to be maintained unimpaired throughout all the construction processes until no further use can be made of the material.

**6.5.3.8.2 Bentonite.** The bentonite supplied to the site should be in accordance with Specification No. DFCP 4<sup>3)</sup> of the Oil Companies Materials Association or the equivalent.

The bentonite should be mixed with clean fresh water until it is fully hydrated to form a slurry having a density, viscosity, shear strength and pH value suitable for the particular excavation and concrete placing conditions. The temperature of the water should be not less than 5 °C.

The density and hence the viscosity of the slurry controls the ability of the fluid to carry away particles of material in suspension and also controls the stability of the gel when the fluid is at rest. If the shear strength of the gel is too high, the concrete flowing from the tremie pipe will not displace the gelled slurry, and excessive sediment may remain trapped beneath the concrete at the base of the excavation.

It is usual to state that the density of the bentonite slurry should be less than 1.10 g/mL; the viscosity as measured by the Marsh Cone should be within a range of 30 s to 90 s, and the 10 min gel strength to be in the range of 1.4 N/m<sup>2</sup> to 10 N/m<sup>2</sup>.

A check on the density and viscosity of the slurry at a level of about 0.2 m above the base of the excavation should be made before placing concrete. If the density is greater than 1.5 g/mL, it should be replaced by a lighter fluid which can be provided by adequately desanding the bentonite before concreting.

Contamination of the slurry by clay and silt from the excavation or by cement from the concreting operations can cause excessive fluid loss and the consequent build-up of a thick filter cake on the sides of the excavation. An excessively thick filter cake can be detrimental to the mobilization of skin friction between the concrete and the soil. This could be particularly significant in bored pile construction.

<sup>3)</sup> Available from the Oil Companies Materials Association, 86 The Strand, London WC2P 0DX.

Contamination with clay and silt can be avoided by controlling the density and viscosity of the circulating fluid. Contamination by cement can be detected by an increase in the pH value of the slurry. The pH value should be maintained within a range of 9.5 to 12. It may be necessary to add alkaline substances such as soda ash to the freshly-mixed slurry to achieve the lower limit of this range.

Excessive spillage of bentonite in the area of diaphragm wall construction should be avoided for reasons of safety and site cleanliness. Proper arrangements should be made with the statutory authorities for the disposal of discarded and surplus slurry.

#### **6.5.4 Use of injection methods to form an impervious wall around the excavation**

Injection of setting grouts can be employed to form an impervious wall around an excavation. Where the ground is of varying permeability, it is possible to limit the treatment to those aquifers whose permeability requires to be reduced and so save expenditure (see 6.7). However, total cut-off can be difficult to achieve because of variability of soils and sensitivity of injection techniques to soil permeability variations.

### **6.6 Methods of improving the physical properties of the ground**

#### **6.6.1 General**

Several methods are available to decrease the permeability, increase the strength or decrease the compressibility of the ground. These techniques may be used to meet temporary requirements, such as reducing the water flow towards excavations, or to provide a permanent improvement such as increasing the strength of the soil under a foundation. The methods used to improve the ground are preloading compaction, installation of vertical drains, injection of grouts and electrochemical and thermal hardening of the ground.

#### **6.6.2 Shallow compaction**

When the soil is loose or wet, simple surface treatment by compaction or drainage can often be employed to improve its supporting power to a limited depth. Where the groundwater table is above foundation level, the first consideration will be to improve the drainage of the ground, thus lowering the water table below foundation level and facilitating construction on stable ground.

Where drainage is effective in sandy or gravelly soils, the loose soil may be compacted by ramming or rolling; with the size and type of equipment that can be used in normal foundation excavations, the improvement given by such methods of surface compaction will not normally extend to depths of more than about 0.5 m below ground level. However, the method may be useful in restoring stable conditions in granular materials which have been loosened by construction operations.

Compaction in clays or silts is usually ineffective and may even cause softening of the ground due to puddling in wet conditions. It may, however, be desirable to roll or ram the base of an excavation in stiff to hard clay or soft rock which has become loosened or disturbed by mechanical excavation.

Stabilization of granular soils with cement or lime may be considered as an economical means of preventing erosion by wind or water and loosening during construction operations. Such treatment could be considered as an alternative to the provision of a layer of lean-mix blinding concrete beneath foundation structures. Where stabilization is used to prevent erosion beneath shallow foundations or as a base to the foundation structure in granular soils, the treatment should extend sufficiently beyond the area to be loaded to prevent undermining and lateral movement of the ground at the edges.

#### **6.6.3 Deep compaction by vibration**

The settlement due to loading of loose non-cohesive granular soils or fill materials above or below the water table can be improved by deep compaction, achieved by introducing large tubular vibrators into the ground to depths of up to 25 m. This process may be an economical alternative to piling or other methods of improving the bearing capacity of such soils. Liquefaction risk of loose soils under earthquake shocks can also be reduced by compaction.

The depth to which compaction can be achieved is normally limited by considerations of withdrawal of the vibrator after reaching the required depth. However, since granular soils usually decrease in compressibility with depth the treatment need not extend to their full depth.

The equipment comprises a tubular vibrator 300 mm to 400 mm in diameter and 2 m or more in length actuated by an eccentrically-mounted axially rotating weight inside the lower tip of the tube which generates essentially horizontal vibration below ground surface. The vibrator is usually handled by a mobile crane which lowers the tube with the vibrator working to enable it to sink under its own weight. Water jetting through the machine is usually employed to reduce effective stresses in dry soil and to assist penetration of resistant layers. Vibratory pile drivers with contrarotating eccentric weights mounted at the top of the casing can be used as an alternative.

Cohesionless soils undergo compaction if they exist at less than maximum density. This will be evident by the formation of a crater around the vibrator at ground surface. As compaction proceeds, gravel or sand of selected grading may be filled into the crater to make up the ground to its original level. The range of compaction will depend on the energy of the vibrator and the granular structure of the soil. It is usually in the order of 1 m to 2 m from the vibration source.

The pattern of treatment is by single compactions, groups, rows or multiple rows to cover any plan shape required. The main objective of treatment is uniformity of compaction to minimize differential foundation settlements. Allowable bearing pressures ranging up to 500 kN/m<sup>2</sup> can be achieved on the compacted ground. Choice of compaction spacing for a design bearing pressure is determined empirically and subject to site tests during and after treatment; this is more economical and reliable than calculations based on soil dynamics. In cohesionless soils layered with cohesive or low permeability bands, the material backfilled around the poker should be a medium to coarse gravel to form stiffening columns through these layers which are less susceptible to compaction by vibration. Such granular columns have the added benefit of acting as drains in soils of lower permeability which is particularly important when treating fine-grained soils in earthquake areas.

#### **6.6.4 Ground improvement by vibro-displacement and vibro-replacement**

Large vibrators are also used to form columns of coarse granular material in soft silts or clays or weak compressible fills with the primary objective of reducing their compressibility but also to improve shearing resistance.

Foundations can then be constructed directly on groups or rows of these columns. The stiffening effect is most marked where independent or narrow strip footings are placed on a small group or single row of columns for which the bearing capacity of the ground may be increased about two or more times. Under rafts and widespread loads such as tanks or embankments, the remaining effect is most marked at low average stresses and is governed by column spacing.

As columns consist of uncemented aggregate, their strength depends on the radial support available from the natural soil and is critically determined by the column diameter. They are designed as friction piles with the additional requirement that they should not bulge significantly.

Clays strong enough to permit a stable unlined bore may be displaced by penetration of the vibrator which is then removed to allow gravel or stone to be tipped and compacted by the vibrator in about 1 m stages to fill the hole. Some forced compaction of the ground surrounding the column is achieved; this is known as vibro-displacement.

In soft unstable soils, water circulation is used to support the bore as in washboring. Gravel or stone backfill is dropped against up-flowing water in the annular space around the vibrator which then compacts the backfill in place; this is known as vibro-replacement and does not have a significant compactive effect on the soil between the columns. The method will be ineffective if the ground has insufficient strength to provide lateral support to the columns.

Whenever deep compaction is contemplated, preliminary soil investigations are necessary to determine particle size distribution and in situ density, preferably supplemented by estimates of permeability. Where strengthening by stone columns is proposed, plasticity index, undrained cohesion and compressibility characteristics of the soil are important for design. The effectiveness of treatment may be checked by any of the standard methods of in situ soil testing at selected points before and after treatment. In cohesive soils, time should be allowed for dissipation of pore pressures induced by treatment before making final tests.

Deep compaction or vibro-replacement techniques should not be used in soils or fill materials above ground water level where these contain degradable or soluble constituents.

Biological degradation or solution of cementing bonds around mineral particles due to random water seepages may cause collapse of the ground with loss of lateral support to the stone columns.



### 6.6.5 Deep compaction by heavy tamping

This process is for compacting granular soils and silts or mixtures of these including a large variety of fills. It may also be employed to displace soft organic soils with a stronger fill.

It involves simply dropping a free-falling weight, usually of 10 t to 15 t, from heights varying between 5 m and 25 m or more onto the ground surface. Forced compaction results from dissipation of energy of successive impulses inducing a degree of irreversible compression of void spaces and correspondingly elevated pore fluid pressures which have to dissipate before improved soil strength is realized. At impact, some hydraulic fracture may occur where the impulsive stress is high.

Tamping is usually carried out continuously across the area to be treated or by a number of blows at points spread 5 m to 10 m apart on a regular pattern. This treatment is designated a pass, and the complete treatment may consist of three to five passes undertaken with intervals sufficient to allow for pore pressure dissipation between passes. Where appropriate, this can be accelerated by providing buried drainage.

The area of the tamper in relation to its weight is chosen according to the strength of the upper soil to limit penetration and surface shearing. It is usually necessary to cover fine-grained soils exposed at surface with a 1 m thick layer of granular fill. For heavy compaction to compact the soil to depths of the order of 10 m, large weights and heights of drop are necessary. The weights are normally lifted and dropped by means of large crawler cranes, but winches have been used in conjunction with tripod masts. This and the time needed for pore pressure dissipation make the method most economic for comparatively large areas of treatment.

The sequence of operations and of impulse levels required for treatment depends strongly on the soil type and profile. For this reason the soil characteristics have to be established in considerable detail before treatment and checked after each pass.

It is convenient and economic to do this by in situ tests with pressure-meters or penetrometers, or by tests to monitor change of dynamic properties of the soil. Piezometers are sometimes used for monitoring pore water pressure at critical points. After each pass the ground surface is smoothed and levels taken to check the enforced settlement. This empiric control of compaction is very important to detect soil variations of a scale sufficient to influence the proposed foundations. It forms a major part of the whole technique and should not be neglected or minimized in scope.

### 6.6.6 Use of vertical drains

The natural process of consolidation of compressible soils can be accelerated by improving the drainage conditions within the soils, so assisting in the outward migration of the water.

If such soils are loaded on their surface by a temporary surcharge or permanent work such as an embankment, the water pressure in the underlying soil will increase. If the water cannot escape sufficiently quickly a dangerous stability condition could arise, which in some cases can be relieved by a reduced rate of forming the surcharge.

Alternatively, this condition can be anticipated and exploited to accelerate drainage of the compressible soils by installing vertical columns of sand or strips of preformed permeable material within the ground to enable the excess water to escape more rapidly. These columns are called sand drains, vertical drains or wick drains, and are frequently considered for improving the strength of the soil.

Sand drains can be formed by one of several methods. In small installations, the borings can be made using bored pile equipment using a casing to line the hole. For larger schemes the holes can be formed by driving a steel tube fitted with an expendable shoe to displace the soil away from the tube, or a special casing can be jetted down to the required depth. (Care should be taken not to reduce the permeability by disturbance.) The holes so formed are then backfilled with a selected sand as the borehole lining is removed. At the groundwater surface a horizontal blanket is placed to connect the sand drain heads so that the expelled water can be drained away.

More usually machine driven mandrels are used to force down thin strips or wicks from perforated plastics or cardboard. These proprietary systems have the advantage that they displace a lesser volume of soil than sand drains and so minimize ground heave and the development of excess pore pressure at the time of installation.

The soil to be strengthened by these methods requires investigation of its strength characteristics before and after sand draining. Piezometers should be installed in the ground to measure the change in water pressures as drainage proceeds. The extent of sand draining required can be established by calculation, and if the scheme is large a small pilot test scheme will provide valuable data on drain size centres.

The economics of vertical drain installations require careful consideration since in layered or laminated soils the permeability of the soil in a horizontal direction is often sufficiently high to permit drainage to be achieved more effectively and at a faster rate than can be achieved by sand drains.



The permeability of the soil should be measured in situ to confirm the feasibility or economic desirability of a vertical drain installation, because experience has shown that values of permeability coefficients obtained from laboratory tests can be wildly misleading.

#### **6.6.7 Electro-osmosis**

The electro-osmosis system (see 6.4.4.2.7) can be used to reduce the moisture content of a very soft silty clay or clayey silt and thus increase its shear strength and reduce its compressibility. Settlement is also induced in the compressible soil layers. Electro-osmosis may be used in this way to permit an excavation to be made in conditions where base heave would occur without this treatment, or to arrest movement in an unstable slope.

### **6.7 Ground treatment by injection of grouts**

#### **6.7.1 General**

Certain civil engineering problems call for treatment of the ground by the injection of suitable grouts in order that its engineering properties and characteristics may be altered to permit design and construction conditions to be met on a temporary or permanent basis. Such treatment is undertaken to reduce the permeability of the ground or to improve its strength, or to do both.

Ground treatment by injections may often take one of several alternative forms to suit the requirements of the problem but, whichever is selected, it will need to be able to effect treatment reliably and economically in relation to the result required.

The geology of the ground will influence the choice of method, and it is axiomatic that no treatment can be properly considered until an adequate site investigation of the relevant ground and water conditions has been undertaken. Reference should be made to 6.2 which deals with such site investigation, and a check should be made on the suitability of other geotechnical processes appropriate to the problem before deciding upon the use of injection of grouts.

This method of ground treatment is to inject suitable grouts, being fluids with or without fillers which, in their ultimate position in the ground, will be stable and, in combination with the ground, will meet specific engineering requirements. These fluids are required to travel through the ground to controllable limits and, in so doing, should not be affected by the ground or the groundwater to the final detriment of the work.

Grout under pressure will attempt to infiltrate through voids or zones of weakness in the soil structure; the extent of penetration of the grout will depend on its fluidity and on the nature and extent of the voids which it is attempting to fill. Generally, the more open the ground the easier it is for grout to penetrate, and the consistency of the grout can be correspondingly thicker; in the other extreme in fine-grained soils, such as silts and fine sands, the grout has, of necessity, to be of low viscosity and also should not be broken down during the time in which it is moving through the soil. When rocks are being grouted, the main considerations are to fill voids due to jointing, bedding and cavities, and when these are large a grout of higher viscosity containing fillers is normally suitable.

Possible variations in the nature of the ground on any particular site may call for the use of more than one grout. For example, saving may be effected by using cheaper coarse grouts to fill the larger voids, followed by a more expensive penetrative grout to fill the remaining fine voids. The volume of grout used and the acceptance rate will much depend on the extent and nature of the voids to be filled and these cannot always be predetermined. In assessing any grouting proposals regard should be given to the possibility of there being a large variation in the quantity of grout actually used compared with that predicted.

The selection of grouts has also to be considered in relation to setting up short-term and long-term effects, not only within the vicinity of the work but also in the surrounding area, e.g. disturbance to adjoining structures, ground heave and alteration to the groundwater conditions.

#### **6.7.2 Investigation of the ground prior to injection treatment**

Attention is drawn to 6.2 on site investigation.

It is advisable to take levels of the ground surface in the area of grouting so that any movement of the ground, such as heave, can be noticed, and in some cases it is advisable to install indicators at depths which will give an indication of variation in ground movement. A survey should be undertaken, local to the work, of any structures which it might be claimed could be affected adversely by the grouting. A check should also be made in the vicinity for the presence of any sewers or public services through which grout might infiltrate, particularly through any cracks in the lining of sewers.

### 6.7.3 General features of grouting

The main features of grouting are as follows.

- a) Grouting consists of injecting fluid compositions into the ground to render it less permeable or stronger or both. The object of grouting is to fill pores and fissures with solid deposits.
- b) The treatment may be of a temporary or permanent nature.
- c) Thinner grouts, i.e. grouts of lower viscosity, are required in ground of lower permeability, but this may be modified if extremely long periods of injection are possible with the grout chosen.
- d) If grouts of the suspension type are used, the size of the suspended particles will need to be small enough not to cause premature filtration blockages in the pores or fissures treated.
- e) Injected grouts should have sufficient stability to prevent them from being displaced from the ground by groundwater flow or hydrostatic pressures during injection and after hardening.
- f) In general, grouts based upon suspensions should be sufficiently stable to prevent premature sedimentation of solid matter.
- g) Grouts based upon gel formations should produce a gel which, in the maximum size voids to be treated, is stable for a period long enough to suit the engineering requirements.
- h) Grouts based upon the emulsion type will need to have emulsion breaking conditions controlled to ensure that the required radius from the points of injection is adequately treated.
- i) Where there is a wide distribution of void sizes, it is usual to treat the larger voids first with a thicker grout and then to fill smaller voids or fissures with thinner grouts.
- j) The criterion for stopping grouting may be determined on a pressure or volume basis.

### 6.7.4 Methods of grouting

Methods of grouting will vary to meet the engineering requirements of the works, ground conditions, resources and techniques of the individual specialist firms undertaking the grouting, and the method may well vary as the work proceeds. Usually, grouting by injection is achieved by pumping a prepared grout through to the head of a steel or plastics pipe placed into the ground that has to be treated; the fluid passing down this pipe has to discharge through an orifice in the end, usually in the form of a perforated length at the tip of the pipe. Each job will require an installation suited to the magnitude of the scheme, but will generally call for an adequately equipped station for storing grout materials, preparing the grout using suitable volume and weight batching equipment, and pumping plant capable of delivering the grout at controllable rates and pressures.

Instruments are required to measure pressure, volumes of grout placed, with facilities to plot the information so obtained during the operation.

In some cases ancillary plant and equipment is needed between the base station and the point of working, e.g. to boost pumping pressures or to re-mix grouts affected by the length of pump lines. Generally, a single fluid is used, but there are certain processes requiring two separate fluids which are injected separately to mix within the ground.

Usually, the purpose of the grouting operation is to force the grout into voids and fissures within the ground, and care will need to be taken to ensure that the grout travels where it is required to infiltrate and does not simply escape from the point of discharge from the pipe outlet or seep back along the pipe and flow out at ground level.

The method of grouting should take into account that the ground is generally more permeable in the horizontal than the vertical direction.

Three established operational techniques are described in **6.7.5** and two main philosophies for the treatment of differing ground conditions in relation to job requirements are described in **6.7.6**.

### 6.7.5 Operational techniques

**6.7.5.1 Open-ended type grouting.** This is the simplest form of grouting and the equipment normally consists of an open-ended injection pipe which, whilst being installed, is sealed at the lower end by means of a suitable plug which is expendable once grouting is commenced. This type of grouting, although very simple, is subject to the main criticism that grout may flow back around the outside of the pipe to ground surface in preference to infiltrating into the adjoining ground, particularly if high pressures are needed. However, in very open gravel and larger material it is frequently used as part of an overall grouting procedure, often as a preliminary to the use of thinner grouts injected by other techniques.

**6.7.5.2 Stage grouting.** Generally, stage grouting is used in treating rock; in this process a borehole is sunk into the rock usually using some form of drill. After the drilling has been taken, for example, 1.5 m to 3 m into the rock, grout is injected into the hole and, after it has slightly set off, the hole is drilled out and a further length is drilled. This is then similarly grouted up, and so progressively it is possible to proceed with the grouting stage by stage. Alternatively, the borehole drilling can be taken right down to its full depth, and then it is grouted up in lengths, using a device called a packer to seal off the top of the length to be grouted, so that the grout can be contained under pressure over the length to be treated.

**6.7.5.3 Sleeve grouting.** This form of grouting is historically based on the use of a sleeve device called a tube-à-manchette; the object of this form of grouting is to ensure a proper control of the placing of the grout, and it also enables variations in grout sleeve types to be used. Briefly it consists of boring a hole of diameter, for example, 100 mm through the ground, and then placing within this a smaller pipe (the tube-à-manchette) having groups of peripheral holes spaced at approximately 300 mm apart, these holes being covered with a sleeve of rubber which acts as a valve. The annulus between the steel pipe and the borehole is then filled with a plastic type of sleeve grout which is then allowed to set. Grout is introduced through a pipe, usually 20 mm in diameter, the end of which has packers on either side of orifices which permit the grout outflow from such a pipe to be discharged through any selected sleeve in the tube-à-manchette pipe.

The grout, in passing through the sleeve, forces its way through the peripheral plastic grout and breaks out into the adjoining ground in a horizontal direction. The presence of the plastic grout round the tube-à-manchette pipe prevents grout leaking vertically along the pipe. It is possible by this technique to localize the point of entry of the grout into the ground and consequently to vary the type of grout to suit the soil being treated. Furthermore, by washing out the grout pipe it is possible to return at a later stage and use either more of the same grout or a different type if needed.

**6.7.5.4 Types of grout.** Table 10 shows types and examples of grout currently in use in treating various ground conditions. Improvement in existing grouts and introduction of new grouts can be expected as a result of research in this field; therefore it is important to keep this table under review. The type of grout used will influence the criteria on which a decision to stop grouting is based.

### 6.7.6 Grouting philosophies

**6.7.6.1 Permeation grouting.** The aim is to fill the pore spaces of the ground to be treated as uniformly as possible without significant disturbance to the soil structure. Limited injection pressures only can be employed and need to be carefully monitored. Permeation is used where it is necessary to obtain maximum void filling and where the zone to be grouted is comparatively narrow so that the essential aim of a high degree of control of grout distribution can be achieved. This process is particularly necessary when the main aim of the grouting is to strengthen the soil which requires the majority of voids to be filled for effective treatment. The method requires detailed knowledge of soil conditions for design of treatment and a high degree of supervision.

Table 10 — Types of grout

Ground	Typical grouts used	Examples
<i>Alluvials</i> Open gravels Gravels Coarse sands Medium jointed	Suspension	Cement suspensions with particle size of about 50 µm Cement clay, clay treated with reagents Separated clay and reagents, montmorillonite clays with sodium silicate and deflocculants (clay gels) Two-shot sodium silicate based systems for conferring strength Bituminous emulsions with fillers and emulsion breaker
Coarse sands Medium sands		Colloidal solutions Single-shot silicate based systems for strength (silicate – organic ester) Single-shot lignin based grouts for moderate strength and impermeability Silicate – metal salt single-shot system, e.g. sodium silicate – sodium aluminate; sodium silicate – sodium bicarbonate Water soluble precondensates, e.g. urea-formaldehyde Oil based elastomers (high viscosity)
Fine sands Silts	Solutions	Water soluble polysaccharides with metal salt to give insoluble precipitate Water soluble acrylamide, water soluble phenoplasts
<i>Morainic</i>	Any of the above	Any of the above. Choice of grout dependent on grain size and content of moraine
<i>Fissured rocks</i> Open jointed Medium jointed	Suspensions	Cement – sand, cement, cement clay
Medium jointed Fine jointed		Solutions Oil based elastomers, non-water soluble polyesters, epoxides and range of water soluble polymer systems given above Hair cracks in concrete would be treated with a high strength low viscosity polyester or epoxy resin
<i>Clays</i>	Suspensions	Cement, cement clay

**6.7.6.2 Consolidation or compaction grouting.** In contrast to permeation, this approach forces grout into the soil under high pressures, locally disturbing the soil structure. The hydrofracture (or claquage) grouting technique uses fluids that cut in the form of tongues through planes and zones of weakness in the soil. It is used most frequently in heterogeneous and lensoidal soils with fine-grained inclusions. The resulting reticulation and interconnection of veins of grout permeating coarse lenses and compacting finer zones can lead to significant reductions in overall permeability. This technique allows a comparatively widely spaced array of injection holes leading to good economy where the volume of soil to be treated is large. Significant ground movements can accompany this kind of grouting in superficial soils depending on the treated volume and depth; due consideration has to be given to the effect of this disturbance on nearby structures.

When confined by substantial overburden or in deep tunnels, hydrofracture grouting can be applied using repeated very high pressure injections to achieve effective consolidation of clayey or silt sized soils by successively squeezing out moisture to stabilize them sufficiently to allow safe excavations.

Compaction grouting is a technique involving deliberate displacement of the soil by extrusion of bulbs of comparatively stiff grout from open-ended injection tubes. The grouts of near mortar consistency displace rather than fracture the soil. Hence the compactive effect is highly localized and controlled. The method is commonly used at shallow depths to correct settlements of existing buildings and in some instances to restore them to original level after settlement by directing treatment beneath the foundations.



**6.7.6.3 Open gravel containing very little fine material.** Cement or cement clay suspensions can be used. Often a rapid-setting additive is needed to limit the extent of travel of the grout. Where water flowing through open gravels at a high velocity has to be staunched, a technique can be employed whereby bituminous emulsion is injected simultaneously with cement through separate or concentric pipes to form a “flash-setting” rubbery substance which reduces water velocities, thus facilitating subsequent grouting with more conventional material.

**6.7.6.4 Fine gravel and coarse sand.** Depending upon whether strength or impermeability is required, a wide range of grouts can be used. Dispersed clay chemical grouts, soft silica gels or weak organic polymer gels can be used to give impermeability. Strength can be conferred by treatment with two-shot silicate based systems, or liquids or stronger single-shot silicate grouts.

**6.7.6.5 Medium sands.** The colloidal solution systems can be used in the medium sand range. These include those based on silicates, organic polymers or lignin. They have viscosities a little above that of water, and their economic use for rendering ground either impermeable or stronger depends largely upon the control which can be achieved over their setting time.

**6.7.6.6 Fine sands and silts.** Very low viscosity grouts of the pure solution type will need to be used in fine sands. In silts it is sometimes possible to carry out hydrofracture grouting to compress them and establish an interconnecting network of planes of rupture filled with grout of low permeability.

**6.7.6.7 Clays.** It is not possible to grout clays by permeation, but it is sometimes possible to strengthen a mass of clay by grouting using the hydrofracture principle.

**6.7.6.8 Morainic materials.** Moraine constitutes a not uncommon foundation material for dams, and as it can range from boulders to clay, the method of treatment may involve all the grouting methods mentioned above.

**6.7.6.9 Rocks.** Cement or cement-clay suspensions are generally used for grouting rock formations. The grout enters open fissures or cavities in the rock mass. Prior to grouting, flushing with water is employed to remove sand, silt or clay infilling the cavities or fissures.

**6.7.6.10 Cavity grouting.** Large cavities such as in old mine workings, swallow holes, etc. can be filled with grouts, including low cost filling agents, i.e. local clays, fly ash or foams that set. Such cavities can sometimes be filled with coarse material, i.e. handlable boulders, colliery waste and then subsequently grouted up.

Sometimes a cost benefit can accrue from using coarse grouts followed by fine grouts.

**6.7.6.11 Jet grouting.** A method of ground treatment is the injection under pressure of an appropriate mix of cement grout, either alone or with bentonite or other filler, through a rotating nozzle at the end of the injection pipe.

The application includes the improvement of the permeability and compressive characteristics of soils by a replacement technique.

### 6.7.7 Site control

**6.7.7.1 General.** Site control can be divided into two aspects, control of grouts and control of grouting operations.

**6.7.7.2 Control of grouts.** Tests to establish the suitability of a grout depend upon the type of grout and upon its function. The acceptable range of grout properties is often best established after a trial study on site, but the kind of tests and the equipment needed on site which may be regarded as appropriate are shown in Table 11.

**6.7.7.3 Control of grouting operation.** All grouting work should be supervised by an engineer experienced in grouting. Prior to commencing work, a survey should be made of adjacent structures and utilities; further surveys should be made as grouting proceeds.

The actual control of operations calls for an adequate system of recording information on grouting pressures and flow into the ground at each injection point and the sampling of the grout being used.

Permissible injection pressures should be decided after some trials. The pressure at which the ground fractures may be a function of ground strength, and, in certain ground conditions, of overburden. When pressures are likely to exceed overburden pressures, a system of check levelling points should be established and observed during the grouting programme.

Good quality cores of ground before and after treatment can often provide useful information on which to judge the results of grouting.

In situ permeability tests in sufficient numbers should also be performed before and after treatment. Trace indicators can also be used on ground treated with chemical grouts in order to assess the presence of grout.

Grouting is very much an art calling for good engineering appreciation of its effectiveness.

Injection pressures are governed very much by the size and distribution of voids in the ground to be treated.



The maximum permissible injection pressure at the point of injection is usually limited by the weight of overburden over the place of injection; it is also influenced by the extent of the area over which the grout is spreading during injection.

The volume of grout used is largely dependent upon the extent of void which can be filled, except in the case of hydrofracture grouting.

## 6.8 Safety precautions

NOTE Attention is drawn to the statutory requirements given in section 11 affecting the health and welfare of workmen on construction sites.

### 6.8.1 Work in compressed air

It is essential that conditions of work in compressed air chambers are very carefully controlled to avoid severe health hazards (see 6.5.1)

### 6.8.2 Working with toxic chemical grouts

**6.8.2.1 General.** Some of the chemicals used in grouting are toxic to varying degrees; many are also corrosive. They can give rise to chemical burns, dermatitic irritation or ulceration if splashed on the skin; others ingested as dust from the atmosphere give rise to internal complaints whilst certain vapours can affect the nervous system. Particular care has to be taken with chemicals in the atmosphere when working in tunnels or other enclosed spaces. None of the chemicals need be dangerous if properly handled. Precautions recommended by manufacturers should be followed and all people working with such chemicals should be provided with protective clothing appropriate to the circumstances. This may include gloves and splash-proof clothing, simple face masks or, in exceptional circumstances, complete breathing apparatus.

Mixing stations should be designed to minimize risks of splashing or dispersion of chemicals in the atmosphere.

**6.8.2.2 Toxic hazards to third parties.** The concept of a limit on toxicity of chemicals used or on toxicity of effluents leached from grouted ground is not generally valid. The toxic hazard depends on the circumstances of location and use of grouts in relation to their physical environment. Very toxic chemicals may sometimes be safely used without public hazard whilst in other circumstances even chemicals generally considered harmless for household use would be unacceptable.

The main hazard to third parties usually occurs during the injection contract. It arises principally from accidental spillage of chemicals seeping into gutters, pipelines or the soil. Such spillage occurs when chemical drums are broken out or when injection pipelines are disconnected or washed out. In these circumstances, relatively large volumes of chemical grout constituents in ungelled form can escape into the soil. Whether these subsequently become a hazard depends upon the scale of the operations and resulting leakage in relation to the total hydrological environment.

After injection when the chemical gels are in service, the hazards usually are negligible. Very few gels are toxic and nearly all are of very low permeability and used in circumstances where hydraulic gradients across them are small. Thus leaching of any constituent chemicals not incorporated in the gel reaction is at an extremely low rate and the volume available is comparatively small in relation to the hydrological environment.

Nevertheless some chemicals, notably acrylamides, can be dangerous unless the circumstances in which they are used are carefully considered and appropriate safety measures are taken.

Table 11 — Grout tests

Grout type	Test type	Instruments/tests
Suspensions	Consistency	Flow cone Rotational viscometer
	Bleed	1 L cylinder
	Density	Baroid mud balance
	Maximum grain size	Wet sieving
	Strength	Cylinder crushing or vane tests for weaker grouts
Colloidal solution	Consistency	Flow cone Rotational viscometer Flow channel for thick fillers
	Shrinkage	1 L cylinder
	Specific gravity	Hydrometer
	Strength	Vane tests
	Gel time	Controlled temperature baths

## Section 7. Pile foundations

### 7.1 General

The provisions of this section of the code are intended to apply to bearing piles of various materials installed by any method, including piles driven by the blows of a hammer or by vibrator, driven cast-in-place piles, bored cast-in-place piles and jacked piles. If types of piles not specifically provided for in this code are used, they should follow the general principles of the code in so far as they are applicable.

The main function of bearing piles is to transfer the load to lower levels of the ground which are capable of sustaining the load with an adequate factor of safety and without settling at the working load by an amount detrimental to the structure that they support. Piles derive their carrying capacity from a combination of friction along their sides and end bearing at the pile point or base. The former is likely to predominate for piles in clays and silts and where long sockets are formed in soft rocks. The latter applies to piles terminating in a stratum such as compact gravel, hard clay or rock.

When friction piles are installed in a deep deposit of fairly uniform consistency in order to transfer the foundation pressure to the lower levels, they should be long enough to ensure a substantial advantage over a shallow foundation. In these circumstances it should be borne in mind that for the same superficial area of pile surface, a few long piles forming a piled foundation are more effective and will support the load with smaller settlements than many short piles.

Piles should be installed to the prescribed depth, resistance or set per blow without damage to the pile shafts or the bearing stratum and records of the installation process should be maintained. The piles should be able to carry their design loads without exceeding the permissible working stresses in the material of the pile, but the stresses during driving may exceed these. The stresses during pitching and handling should be within the safe bending stresses prescribed in the design.

The load should be applied concentrically with the axis of the pile or the centre of gravity of the pile group. Allowance should be made in the design for inaccuracies in positioning the piles, particularly for isolated single piles or pairs of piles. Such piles should be designed to accommodate the resulting moments or should be restrained in accordance with 7.3. Where permanent eccentric loads have to be carried, due allowance should be made in the design.

In certain situations vertical piles may be subjected to horizontal or eccentric loads. If the ground is unable to resist the resultant forces without excessive lateral movement, additional resistance should be provided. This can be achieved by increasing the number of vertical piles or increasing their stiffness, by the use of raking piles or ground beams or by replacing the upper soil layer with soil more capable of resisting horizontal forces.

Special consideration should be given to effects such as ground heave due to frost or the behaviour of expansive clays, temperature variations in the ground and alterations in groundwater level, in case they are detrimental to the bearing capacity or effectiveness of the piles. Temperature changes may also affect the horizontal dimensions of the superstructure, thereby distorting the piles and causing additional stresses.

### 7.2 Preliminary investigations

#### 7.2.1 Ground investigations

An investigation of the ground should be carried out by competent and experienced persons in accordance with BS 5930. Borings should reach depths adequate to explore the nature of the soil both around and beneath the proposed piles, including all strata likely to contribute significantly to settlement. In cohesive soils undisturbed samples should be taken from the borings and tested for strength, compressibility and other characteristics to provide information on the carrying capacities of the soils at various depths, so that a preliminary estimate of the length and spacing of the piles may be made. Penetration tests and tests of disturbed samples are of value in assessing the variations in the ground conditions when boring through granular soils.

It is important that the nature and occurrence of groundwater should be investigated. If the standing levels vary from stratum to stratum or if there is a water table gradient between boreholes this should be noted. Groundwater or soil may contain harmful constituents in amounts sufficient to cause damage to Portland cement concrete or buried metals. Chemical analysis of samples of the groundwater and soil should be undertaken to assess the necessity for special precautions.

On sites where the soil conditions do not permit point bearing at economical depth, the available point resistance and skin friction should be determined separately at various depths before the pile foundation is designed.

### 7.2.2 Nearby structures

The preliminary investigation should include a careful appraisal of nearby structures and substructures, including the types and layout of all services near and through the site. The choice of pile may be influenced by the effects which its installation would produce on these structures and services. Safety precautions relating to underground services and overhead power lines are referred to in section 11. The appraisal should include examination from records or by trial holes of the nature of nearby foundations and the noting of any evidence of past settlement, subsidence or slips.

### 7.2.3 Preliminary piles

**7.2.3.1 General.** Preliminary test piles should be installed as part of the preliminary investigations in all cases except where in the judgement of the engineer they are unnecessary because of extensive local experience or because high factors of safety are to be employed. They should be sited near the borings and the piling records should be studied in conjunction with data from the ground investigation. The piles should be tested to determine the load settlement characteristics and the ultimate bearing capacity (ICE 1978).

Preliminary test piles should, wherever possible, be of the same material and dimensions and constructed by the same techniques as the working piles, to ensure that their behaviour under test loads is comparable with that of the working piles.

The test loading of preliminary piles should be carried out generally as described for test piles in 7.5. It is important to make careful note of every observed phenomenon, since anything unusual or unexpected may be significant in the subsequent appraisal of the investigation. Observers experienced in pile testing should be present to supervise each test. The equipment should have a reserve beyond the predicted ultimate bearing capacity.

**7.2.3.2 Driven test piles.** Where preformed piles are proposed, special preliminary piles of ample length, designed to withstand very hard driving, may be necessary in the exploratory stages.

A driving record showing the number of blows per unit of linear measurement of penetration should be plotted against the penetration of the pile below ground level, to show the variations of soil resistance with depth.

When an acceptable resistance or set appears to have been reached, the driving of a preliminary test pile should be suspended for an interval sufficient to permit the soil to recover from the disturbance of pile driving, and then resumed to determine whether there is any increase or decrease in resistance. In soils that dilate when disturbed, e.g. silts and some shales, negative pore pressures can be set up temporarily and the driving resistance may fall as these pore pressures return to normal. In clays disturbance can cause positive pore pressures to develop and the strength of the soil may increase as these dissipate. The necessary time interval before redriving may vary from 1 h to 2 h for non-cohesive soils to 2 days or more for clays. The resistance at the start of redriving is more likely to be representative of the true bearing value of the pile, and each redriving result should be taken into consideration when deciding the factors of safety to be employed.

## 7.3 Design considerations

### 7.3.1 General

The design of pile foundations calls for specialized knowledge and requires an understanding of the ground conditions, the reaction of the ground to the various types of piles and the effects produced by the applied loads. Consideration should be given to the possibility of imperfections in the various types of pile and the consequences to the structure of an imperfect pile.

The decision to use a piled foundation should be made considering the relative costs of suitable alternative foundation designs.

The choice of the type of pile will be determined by the ground conditions, the effects which the installation of the piles may have upon nearby structures and other property and, if more than one type is suitable in the particular conditions, by the relative costs.

Where piles are installed in groups or clusters, the effect of placing a number of piles in close proximity to each other will need to be taken into account.

Every design will have to satisfy three conditions:

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

### 7.3.2 Choice of type

Piles may be divided into three main types, depending on their effect on the soil as shown in Figure 14. These are as follows.

- a) *Large displacement piles.* These include all types of solid pile, including timber and precast concrete and steel or concrete tubes closed at the lower end by a shoe or plug, which may either be left in place or extruded to form an enlarged foot.
- b) *Small displacement piles.* These include rolled steel sections, such as H piles, open-ended tubes and hollow sections if the ground enters freely during driving. However, it should be recognized that open-ended tubes and hollow sections frequently plug and become displacement piles particularly in cohesive soils. H-piles may behave similarly.
- c) *Replacement piles.* These are formed by boring or other methods of excavation; the borehole may be lined with a casing or tube that is either left in place or extracted as the hole is filled.

When considering a bored pile in soft or granular ground especially when water-bearing, particular care will need to be taken to ensure the integrity of the pile shaft (Thorburn and Thorburn 1977) and to avoid:

- 1) disturbance causing instability of the bearing stratum and the surrounding ground;
- 2) an inflow of water which could cause a disturbance to the bearing stratum or the surrounding ground;
- 3) an inflow of water or soil causing damage to the unset concrete in the pile or piles recently installed nearby (see 7.4.4 and 7.4.5).

When a displacement pile is installed, its volume has to be accommodated below the ground surface by vertical and lateral displacements of the soil. Effects of consequential heave or compaction may be unacceptable to adjacent structures.

In some soils the vertical and lateral displacement or heave resulting from subsequent piling may carry with it piles that have already been installed. This may cause end bearing piles to be lifted off their bearing stratum. If lifting is suspected, careful levelling from a datum unaffected by the piling should be made on the heads of piles already driven before and after driving subsequent piles. Risen piles which depend largely on end bearing and which are capable of being redriven should be redriven to the specified resistance. Driven cast in situ piles should not normally be redriven after concreting and should be replaced or supplemented or the required resistance developed by pre-loading the piles. The soil around a driven pile is disturbed during installation and with a pile group the volume of soil affected may be considerable. Mitigation of this effect can be obtained by the use of small displacement piles, or by pre-boring over part of the depth of displacement piles or, if the ground conditions are suitable, by the use of bored piles. Where large displacement piles are to be employed in ground liable to heave, the effects on adjacent piles and structures can usually be minimized by working outward from the centre of a group or by commencing with the piles adjacent to the structure and progressing away from it.

Where vibration or noise has to be reduced to a minimum, piles formed by augering are generally preferable. However, piles driven under carefully controlled conditions do not necessarily cause appreciable vibration and may then be used safely under skilled supervision. When working near existing structures which are founded on loose sands or silts, particularly if these are saturated, it is essential to avoid the use of methods which cause dangerous vibrations, because these may have the effect of temporarily reducing the shear strength of the ground beneath the adjoining foundations. Rapid settlement may also occur due to consolidation of loose sands and silts. If bored piles are used in loose sands and silts, care should be taken to prevent inflow of soil into the boreholes, as this might cause loss of support by the removal of soil from beneath adjacent foundations or create unstable water-filled cavities outside the pile shaft, leading to voids in the shaft during concreting or the washing out of cement.



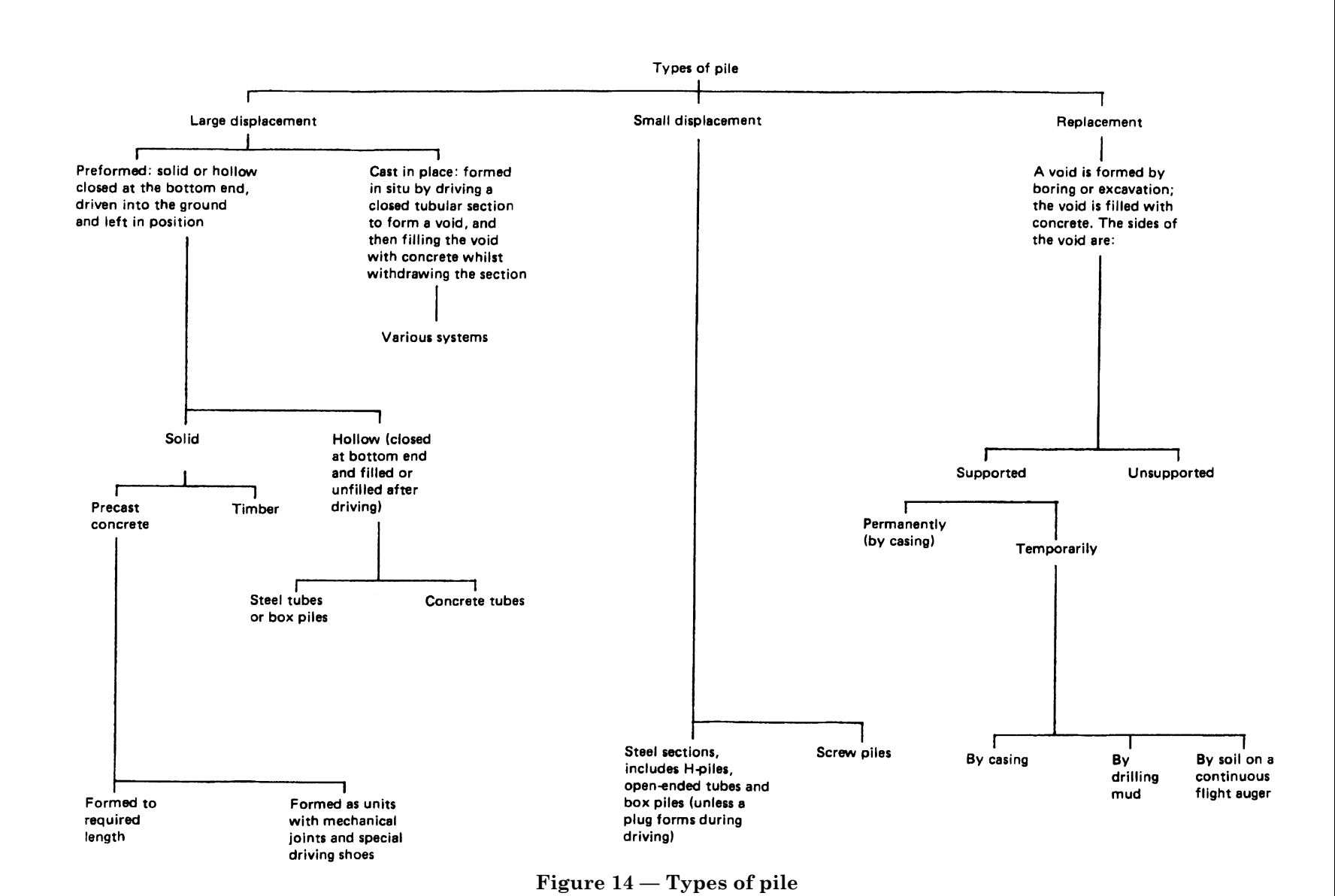


Figure 14 — Types of pile

Where headroom is limited, a pile installed in sections by jacking it into the ground may be used, for example where underpinning is to be carried out, but care is required to ensure that the reaction available is adequate. Some rigs for installing bored piles and some proprietary rigs for driven piles are also capable of working in confined headroom.

Whilst they are rarely used, screw piles are suitable for the construction of structures such as wharves, piers and jetties where there is soft alluvial soil to a considerable depth. They may also be used to withstand uplift forces.

Timber piles may be used in permanent work where the conditions are such that deterioration will not occur.

They are also used for marine structures where permanence may be sacrificed for flexibility and resistance to shock, and in temporary structures. In all cases the possibility of damage by insects, marine borers or fungi should be taken into account.

Where cast-in-place piles are to be installed in granular soils through which water moves freely, care is necessary to ensure that the movement of the groundwater does not leach out or otherwise affect the cement prior to hardening of the concrete. This may be done by providing a permanent casing.

Where the site investigation shows the soil or groundwater to contain substances that are deleterious to normal Portland cement, consideration should be given to the use of a cement of appropriate composition or other measures (see BRE Digest 250).

### 7.3.3 Strength of piles

**7.3.3.1 General.** If a pile or any part of it is preformed, e.g. a driven reinforced concrete pile, it will need to be capable of withstanding without damage the stresses arising during handling and installation. Lifting points should be clearly marked on all preformed concrete piles. Whatever type of pile is used, it will need to have the necessary strength when installed to transmit the loads imposed on it to the soil with an adequate factor of safety against failure.

**7.3.3.2 Stresses during installation.** It is part of the design process to consider the stresses which may arise during the installation of a pile. These are likely to be more critical for driven piles, particularly for those made of reinforced or prestressed concrete (see 7.4).

Stresses which may be high enough to damage the pile can be caused in several ways during driving, particularly the following.

a) If the surface of the pile head is not normal to the direction of the blow, lateral forces will be generated and bending stresses will accompany the direct stresses. Bending may also be caused by the restraint imposed by the leaders on a pile that is running out of position, or by vertical blows on the head of a pile that has become inclined.

b) If the surface area of the pile that receives the blow is not large enough, the material is likely to be crushed. This is most applicable to timber or steel piles driven without a helmet, but a badly fitting helmet or packing that does not fully cover the pile head can lead to crushing or spalling of a concrete pile.

c) If piles have to be driven through a very stiff or compact upper stratum to reach the bearing stratum at a lower level, excessive driving stresses may be caused whilst penetrating, through the upper stratum (see also 7.3.4.1).

d) Care should be taken to avoid generating excessive tensile forces by the use of too light a hammer and a large drop.

**7.3.3.3 Axially loaded piles.** Where a pile of normal dimensions is driven vertically and fully embedded there is little likelihood of its buckling in the ground. It is not therefore necessary to design the pile in accordance with the rules given for columns in BS 8110, CP 114 or BS 449. Where exceptionally long slender piles are driven through soft soils (for example, alluvial soils with a shear strength of less than 20 kN/m<sup>2</sup> (0.2 kgf/cm<sup>2</sup>)) the designer should refer to Francis et al. (1962).

Where part of the finished pile projects above ground, that length should be designed as a column in accordance with BS 8110, CP 114 or BS 449. The effective length to be taken in the calculations is dependent on the lateral loading (see 7.3.3.4) if any, and on the degree of fixity provided by the ground, by the structure which the pile supports, and by any bracing. The depth below the ground surface to the point of contraflexure varies with the type of soil. In firm ground it may be taken as about 1 m below the ground surface; in weak ground, such as soft clay or silt, it may be as much as one-half of the depth of penetration into this stratum but not necessarily more than 3 m. The degree of fixity of the position and inclination of the pile top and the restraint supplied by any bracing should be estimated as in normal structural calculations.

**7.3.3.4 Laterally loaded piles.** A vertical pile may be subjected to lateral forces from a number of causes. There is extensive literature on the design of laterally loaded piles, much of which is referred to by Elson (1984). Where a large portion of the pile is free-standing, as in a jetty, it is sometimes pulled into line after installation, thus causing initial stresses. The finished structure may be subjected to lateral forces due to tides, currents, the impact of ships, earthquakes or other causes. Piles that are fully embedded, as in the foundations of a building or a bridge, may have to resist lateral forces caused by wind on the structure, traction, braking and centrifugal forces of vehicles, displacement of soil from beneath an adjacent loaded area or earthquakes. Lateral movements in a structure caused by temperature changes and concrete shrinkage will also induce stresses in the piling (see Civil Engineering Code of Practice No. 2).

**7.3.3.5 Raking piles.** Raking piles may be provided where the ground in which the structure is embedded cannot resist the applied horizontal forces. In design the loading on a raking pile is generally considered to be axial and the distribution of load between vertical and raking piles may be determined by either graphical or analytical methods. However, the direction of movement of the top of a raking pile is seldom axial and bending can occur. This is generally the case when groups include piles raked in more than one direction or vertical piles. The magnitude of the bending depends on the settlement of the foundation, the rake of the pile and the rigidity of the connection to the pile cap. As an alternative to raking piles, consideration should be given to the use of vertical piles designed to resist the applied horizontal forces.

Whilst bored cast-in-place concrete piles can be installed at inclinations to the vertical generally up to 1 in 4, the formation of raked bored piles to the standards of similar vertical piles is difficult. Alignment tolerances may have to be increased, particularly because the inclination to the vertical of the axis of the lower part of a pile shaft can be less than that specified due to sagging of the drive rod of the boring machine. Departures from the design alignment can have an important effect on the loads carried by the individual piles of a group. In addition, bores may be damaged during the lowering of reinforcement cages and tremie pipes, and debris accumulating at the bottom of pile shafts is likely to have an adverse effect on the performance of the piles in service. Where raking piles are requested, foundation designers should consider using driven piles which can be installed at larger inclinations to the vertical but only in the rarest cases should both bored and driven piles be employed in the same foundation.

Free-standing raking piles are subject to lateral forces due to their own weight, wave forces, the weight or thrust of ice or other causes. Raking piles embedded in fill or in consolidating ground may become laterally loaded owing to the settlement of the fill.

**7.3.3.6 Eccentric loading of piles.** The load carried by a stanchion or column is seldom a simple axial load. Frequently the load on the base of the column is eccentric, or there is a small component of horizontal force or a moment accompanying the vertical force. A single pile is seldom placed with such accuracy that the axis of the pile will coincide with the point of application of the resultant force. Thus there is the likelihood of some degree of eccentric loading on the piles in a foundation consisting of only one or two piles. The piles should be designed to resist the bending which results or the pile cap should be effectively restrained from lateral or rotational movements. The restraint and the pile section or both should be sufficient to resist the moments due to eccentric loading or other causes.

Where differential settlement of a piled foundation is anticipated, and where this would lead to significant stresses in the piles or pile cap, the piles and pile cap should be designed to withstand the stresses induced or measures taken to prevent the occurrence of high stresses.

**7.3.3.7 Tension piles.** Piles may be called upon to resist either permanent or intermittent tensile forces when they form the foundations of, or provide anchorages for, structures such as towers, masts, off-shore rigs and platforms exposed to large wind or wave forces. For these conditions the piles should be designed to transmit the full range of possible loadings to the surrounding ground. Where the capacity of tension piles cannot be assessed from soil data with sufficient accuracy for design, pulling tests should be undertaken. Wherever possible the axis of the pile should be co-linear with the direction of loading. The reinforcement in concrete piles should be adequate to carry the tensile stresses. It is often cyclic loads which present the most difficult design problems.

### 7.3.4 Piles in groups

**7.3.4.1 Installation.** When piles are installed in groups, heave and lateral displacement of the soil should be limited as far as possible by the choice of a suitable type of pile and by appropriate spacing. Some types of soil, particularly loose sand and filling, are likely to be compacted by displacement piles, and the order of installing such piles in a group should avoid creating a compacted block of ground into which further piles cannot be driven. Similar driving difficulties may be experienced where a stiff clay or compact sand and gravel have to be penetrated to reach the bearing stratum. This may be overcome by driving the centre piles of a group first and working outwards, but it is frequently more convenient to begin at a selected edge and work across the group. In some cases it may be necessary to prebore through the hard upper stratum. If the group is confined by sheet piling which has already been driven, it may be preferable to drive from the perimeter inwards to avoid displacement of the sheet piling. However, bored piles or small displacement piles should then be considered.

**7.3.4.2 Spacing of piles.** The spacing of piles should be considered in relation to the nature of the ground, their behaviour in groups and the overall cost of the foundation.

The spacing should be chosen with regard to the resulting heave or compaction and should be wide enough to enable the desired number of piles to be installed to the correct penetration without damage to any adjacent construction or to the piles themselves.

The cost of a cap carrying the load from the structure to the pile heads, or the size and effective length of a ground beam, may influence the spacing, type and size of piles.

The spacing of piles will be determined by:

- a) the method of installation, e.g. driven or bored;
- b) the bearing capacity of the group.

Working rules which are generally, though not always, suitable, are as follows.

For friction piles the spacing centre to centre should be not less than the perimeter of the pile or, for circular piles, three times the diameter. The spacing of piles deriving their resistance mainly from end-bearing may be reduced but the distance between the surfaces of the shafts of adjacent piles should be not less than the least width of the piles. For piles having bases enlarged by undercutting, specific consideration should be given to the problems, such as interaction of stresses and installation tolerances, that may arise from the proximity of the bases.

**7.3.4.3 Effect of size of group.** When piles are installed in a group to form a foundation, the behaviour of the whole foundation, as well as that of the individual piles, should be considered in design. The behaviour of a pile group depends on the manner in which the loads are transferred to the soil and the soil characteristics, and the ultimate bearing capacity and settlement of the group may not be the same as those of a similar number of isolated piles. The chosen design should ensure that the settlement of the whole foundation is acceptable in relation to the type of structure and the service connections. It should also ensure that differential settlements between single piles and sub-groups of piles within the overall group are acceptable.

The depth to which the soil is stressed to an important degree below the piles in a group is dependent on the size of the group as well as on the load per pile. This is most readily appreciated by using the concept of a "bulb of pressure". The relatively small bulb of pressure associated with one pile affects only a small volume of soil, but the individual bulbs of pressure of the piles in a group merge to form a large bulb of dimensions related to the width of the group. There is no simple relationship between the characteristics of a single isolated pile and those of a group, as any relationship is dependent on the size and other features of the group and on the nature and sequence of the strata (Whitaker 1957; 1960).

**7.3.4.4 Settlement of groups.** The settlement of a large group at a certain load per pile is always greater than that of a small group, and both are greater than the settlement of an isolated pile under that load. This applies to the immediate movement under load (sometimes called the elastic movement) and also, if compressible soils exist within the bulb of pressure, to the long-term consolidation settlement. This effect will not be significant where piles are bearing on rock or other relatively incompressible strata.

Where the support is mainly derived from end bearing, the settlement may be calculated assuming that the foundation load is applied over the area covered by the group at the level of the pile toes. Where a significant proportion of the support is mobilized as friction, the settlement may be estimated assuming that the foundation load is applied over an area larger than the area of the group at the level of the pile toes. Various methods of determining the enlarged area have been proposed but it may be assumed that the foundation load is applied over the area covered by the group at a level one-third of the length of the piles above the pile toes (Peck et al. 1974; Tomlinson 1980).



For deep beds of cohesive soil, a superposition approach to the estimation of group settlement from the settlement of a single pile may also be considered (Poulos 1968b; Butterfield and Banerjee 1971a and b; Banerjee and Davies 1977; Randolph and Wroth 1979; Cooke et al. 1980; Fleming et al. 1985).

The overall settlement of a foundation formed of a number of groups of piles should be assessed considering all the groups acting together and taking into account the effects of any excavations for basements.

**7.3.4.5 Ultimate bearing capacity of groups.** Where cohesionless soils are compacted by the installation of the group, the ultimate bearing capacity of the group may exceed the ultimate bearing capacity ( $Q$ ) of a single isolated pile multiplied by the number of piles in the group ( $n$ ) (Kezdi 1957). The ultimate bearing capacity of piles end-bearing on rock, or on compact sand or gravel with equally strong material beneath, will be substantially equal to  $nQ$ . Where piles rely on skin friction in a deep bed of cohesive material, the ultimate bearing capacity of a large group may be less than  $nQ$ . The magnitude of the reduction depends on the pile spacing, the pile length, the number of piles in the group, the characteristics of the soil, the stiffness of the pile cap and the participation of the pile cap in transmitting load to the soil (Whitaker 1957; 1960). When the cap on a group of piles is clear of the ground, or the ground immediately below the cap cannot be relied on to take any load from the cap, then, with the spacings given in 7.3.4.2 and groups of nine piles or more, the reduction may be in the order of one-quarter or more. When the cap on the pile group is cast directly on the cohesive stratum which supports the piles, it may contribute to the bearing capacity of the group. The distribution of load between the piles of a group in deep cohesive soil depends on the rigidity of the pile cap and the supported structure. When the superstructure is highly rigid, the loads carried by piles around the periphery of a group may be considerably larger than the loads carried by piles in the interior of the group. This should be borne in mind when considering the stresses in the pile shafts and in the design of pile caps for large groups. The designer should refer to the literature on this special problem (Butterfield and Banerjee 1971a and b; Hooper 1973 and 1979; Hain and Lee 1978; Cooke et al. 1981; Fleming et al. 1985).

### 7.3.5 Ground conditions

**7.3.5.1 General.** Piles may be used for foundations in the following conditions:

- a) where compressible strata overlie bedrock;
- b) where compressible strata overlie sand or gravel of adequate density, there being no other beds of greater compressibility below the sand or gravel;
- c) where compressible strata overlie a bed of sand or gravel of adequate density and of sufficient thickness to reduce the stresses applied to compressible strata below;
- d) where the strata consist of cohesive soils of great depth capable of supporting the piles by friction;
- e) in the case of short bored piles, to minimize movements of a structure resulting from swelling or shrinkage of soils near the surface.

The compressible strata may be silt, peat, clay or loose sand, or any combination of these materials. The purpose of piling in a) to c) is to transmit the imposed load through the compressible strata to the more resistant beds beneath, and these will need to be capable of carrying the applied load in addition to the existing load due to the overlying strata. In all cases the possibility of negative skin friction or downdrag and the effect of the additional load which it may impose on the piles should be considered.

**7.3.5.2 Compressible strata overlying bedrock.** The site investigation should establish whether the underlying rock surface is level, inclined or irregular, and the thickness of decomposed rock through which the piles should penetrate. If the surface is inclined, driven piles may require suitable points and be driven so that embedment is achieved. For a pile with its point properly embedded in sound rock, the upper limit to loading may be the allowable compressive stress in the material of the pile. The lateral support provided by even relatively weak material through which the pile is driven is often great enough to enable slender piles to be used.

**7.3.5.3 Compressible strata overlying dense sand or gravel with no underlying compressible beds.** The conditions are similar to those described in 7.3.5.2. Where loose sand overlies the bearing stratum, the effect of displacement piles will be to compact it and, if the spacing is close enough, the block of sand in which the piles are embedded may become sufficiently compact to behave as a pier. Any other process by which compaction can be carried out might be equally effective in providing a foundation.



**7.3.5.4 Compressible strata overlying a bearing stratum of limited thickness with compressible strata beneath.** This may occur in alluvial deposits where a bed of compact sand or gravel overlies clays or silts and is in turn covered by compressible material. Care should be taken to install the piles so that the points or bases do not approach the lower boundary of the bearing stratum and the site investigation should be sufficiently detailed to achieve this with certainty. In these conditions tests on single piles will be of little value in predicting the behaviour of the group and the design should be based on an analysis of the support given by the strata underlying the bed of sand or gravel. For this purpose it may be assumed that the foundation load is applied over the area covered by the group at the level of the pile points or bases, and the analysis should include studies of both bearing capacity and settlement (Tomlinson 1980).

**7.3.5.5 Strata consisting of cohesive soils of great depth.** Both the overall bearing capacity and the settlement of the group will need to be considered. In general, the need to limit settlement will be the governing factor in design.

If the soil is sensitive to disturbance there is a risk that the remoulding resulting from the installation of piles will cause loss of strength followed by consolidation settlement. Consideration should then be given to forms of construction not dependent on piling.

**7.3.5.6 Minimizing movements due to swelling and shrinkage of soil near the surface.** Adequate support has to be developed by the pile at depths greater than those at which the soil moisture content is affected by the growth or removal of vegetation or subject to seasonal variations (see BRE Digests 240, 241 and 242).

Where large soil movements due to swelling or shrinkage can be expected, consideration should be given to sleeving the upper part of the pile shaft or to the need to provide reinforcement to resist tensile forces.

**7.3.5.7 Reducing movements in ground affected by mining operations.** Both the overall bearing capacity and the settlement of piles should be carefully assessed. Piles are used to transfer structural loads to a bearing stratum below a worked out mining seam where the superficial deposits are too weak to support the structure. Piles will often be required to penetrate rock and may require socketing to a level below which subsidence will occur. Possible adverse effects from deeper workings should be considered and also where there is a possibility of loss of lateral stability in collapsed zones or through voids. As further movement of the superficial deposits is possible, the piles should be designed to withstand any additional loading by shear and downdrag.

### **7.3.6 Negative skin friction**

When a pile is installed through a stratum which undergoes consolidation after the pile is in place, the downward movement of the consolidating soil and of any overlying soils will cause a drag on the shaft of the pile. This is termed "negative friction". The consolidation of the soil may be caused by the self-weight of the deposit, the imposition of a surcharge such as a loaded floor or fill, disturbance due to vibration or as a result of remoulding during the installation of the pile. The downward drag on the pile may throw enough additional load on the pile point or base to make the total settlement excessive (Zeevaert 1959).

When piles are driven through sensitive clays the resulting remoulding may initiate local consolidation. The negative friction force due to this consolidation may be estimated as the cohesion of the remoulded clay multiplied by the surface area of the pile shaft.

Where it is expected that the soil around the shafts of end-bearing piles will consolidate, the skin friction exerted by the downward moving soil should be estimated in accordance with the properties of the materials. The downward force will need to be taken into account when the allowable load on the pile is calculated (Fleming et al. 1985).

Consideration should be given to reducing negative friction by proprietary coatings or sleeving applied to the relevant length of pile (Bjerrum et al. 1969; Claessem and Horvat 1974).

### 7.3.7 Pile caps

Pile caps are usually made of concrete and should be of appropriate depth to resist the high shear forces and should be adequately reinforced. It is advantageous for the reinforcement of concrete piles to project into the cap for bonding. Steel piles should be fitted with cap plates or other suitable means for transferring the load from the pile to the concrete of the cap, unless the length of pile embedded is already adequate. The tops of timber piles should be cut square, treated with preservative and embedded in the concrete of the pile cap. The attachment of the pile head to the cap should be adequate for the transmission of lateral loads and the depth of cap over the piles should be adequate for the transmission of vertical load and resisting punching shear. Structural design of pile caps should be in accordance with BS 8110 or CP 114.

Where swelling of soils near the ground surface is expected, owing to the removal of vegetation or to other causes, excessive pressure should be prevented from developing on the underside of the pile caps or ground beams by forming the caps or beams clear of the soil surface. Alternatively, the concrete for the caps or ground beams may be placed on a thick layer of low density expanded polystyrene or cellular cardboard (see BRE Digest 241).

### 7.3.8 Factors of safety

When a piled foundation is being designed, a series of factors of safety are applied as the design proceeds, as follows.

- a) A factor of safety on the materials is applied in the choice of the stresses used in the design of the piles and pile caps as structural members.
- b) When piles are considered singly, it is essential that the working load be not greater than the ultimate bearing capacity divided by an appropriate factor of safety (see section 1). The ultimate bearing capacity may be assessed by applying a dynamic formula, or by using stress wave analysis, or on the basis of soils tests carried out prior to the construction of the bulk of the piles where this is practicable, or from a loading test, as described in 7.5. The factor of safety should be chosen having regard to the nature of the soil, its variability over the site and the reliability of the method by which the ultimate bearing capacity has been determined.

The ultimate bearing capacity should be obtained from a loading test whenever practicable. If a sufficient number of the piles is tested, the data obtained from the tests may be used for adjusting the coefficients used in a pile formula, and the ultimate bearing capacity determined on this basis may then be regarded as reliable.

In certain ground conditions it is found that a pile meets a reduced resistance when redriven (see 7.2). Care should then be exercised in applying any dynamic formula and it is preferable to resort to loading tests.

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

- c) In the case of a large bored pile, particularly one with an enlarged base, it is generally advisable to take account of the different resistance/settlement relationships of the shaft and base when calculating the working load by applying different load factors to the calculated ultimate resistance of the shaft and the base. Reference should be made to ICE (1966b) and Burland and Cooke (1974).
- d) It is essential that the settlement, and in particular the differential settlement, at working load be not greater than can be tolerated by the structure. The factor of safety necessary to ensure this may be larger than that required in b).
- e) The factor of safety applicable to single piles determined in b), c) and d) should be increased when large impact, cyclic or vibratory loads are expected or the properties of the soil may be expected to deteriorate with time.
- f) When piles are installed in groups, the designer should take into consideration the recommendations of 7.3.4 and 7.3.5.5. The ultimate bearing capacity of the group should take into account the bearing capacity of the pile-soil block and the total bearing capacity of the individual piles. An estimate of the settlement to be expected at the working load may be made in suitable circumstances by soil mechanics methods.
- g) Where settlement is not critical, a smaller factor of safety may be used provided there is an adequate margin against bearing failure.

## 7.4 Types of pile

### 7.4.1 Timber piles

#### 7.4.1.1 Materials

**7.4.1.1.1 Types of timber.** Certain types of softwood and hardwood are suitable for use as permanent piles. The choice will depend upon availability in suitable sizes, the expected useful life and the relative cost, including preservative treatment. Availability in the required lengths is frequently a limiting factor.

In the United Kingdom Douglas fir is the most common softwood timber in present use for piles and is imported in sections up to 400 mm square and 15 m long, or somewhat longer to special order. Pitch pine is imported in sections up to 500 mm square and also 15 m long.

In the United Kingdom greenheart is the most commonly used hardwood for permanent works and is normally imported rough-hewn in sections up to 475 mm square and up to 18 m long. Larger sections and lengths up to 24 m long are available to special order. Several other tropical hardwoods are also used and are normally imported in sections up to 9 m long.

In many countries it is customary to use straight trunks of coniferous trees as piles. In most cases it is advisable to remove the bark and pressure impregnate with wood preservative.

Piles should be ordered sufficiently long to ensure that after cut-off at the proper level the top is left clean, sound and uninjured after having been driven.

**7.4.1.1.2 Quality of timber.** The piles should be free from defects which may affect their strength and durability. Straightness of grain is important in timber piles, particularly where hard driving is anticipated.

Some of the defect limitations used in arriving at stress grades in BS 5268 are not relevant in the case of piles and suitable material will generally be obtained from SS grades and better. The centreline of a sawn pile should not deviate from the straight by more than 25 mm throughout its length, but for round piles a deviation of up to 25 mm on a 6 m chord may be permitted. All timber piles should be inspected before driving to ensure compliance with these recommendations (see also section 10).

**7.4.1.1.3 Preservative treatment.** In general, the need to treat with preservatives will depend upon the timber species and the severity of the service conditions. This is discussed in greater detail in 10.2. The heartwood of timbers rated as durable or better, e.g. ekki, greenheart, jarrah, okan, opepe, may be used without treatment. However, if sapwood forms a significant part of the timber, treatment should be carried out. All other timber species should be treated with preservatives. Only pressure impregnation with creosote (see BS 913) or copper/chromium/arsenic (see BS 4072) preservatives is deemed suitable. It should be noted that if a long service life is required and preservative treatment of a timber having inadequate natural durability is chosen, then timbers that are difficult to treat, e.g. whitewood and hemlock, should be avoided.

Recommendations for the treatment of timber piling are given in BS 5268-5. Natural durability and treatability classifications for the various timber species may be found in BRE Digest 296.

#### 7.4.1.2 Design

**7.4.1.2.1 Working stresses.** Working stresses in compression should not exceed the appropriately modified grade stresses given in BS 5268 for compression parallel to the grain for the species and grade of timber being used. In calculating the stresses account should be taken of the stresses during installation and in use (see 7.3). Allowance should be made for reduction in section by drilling or notching. It should be noted that very much larger stresses will be developed during driving.

**7.4.1.2.2 Pile heads.** Before driving, precautions to prevent brooming should be taken; this may be done by trimming the head of the pile to a cross section at right angles to the length and fitting it with a tight steel or iron ring or by an alternative method shown to be effective.

After driving, the heads of the piles should be cut off square to sound wood and treated with preservative before capping.

Where untreated softwood piles form the foundation of a permanent structure, care will need to be taken to ensure that they are cut off below the lowest anticipated ground-water level. They should then be extended, if necessary, in concrete or other suitable material.

If concrete caps are provided, the piles should be embedded for a depth sufficient to ensure transmission of the load. The concrete should be at least 150 mm thick around the piles and should be suitably reinforced to prevent splitting.

**7.4.1.2.3 Pile shoes.** A pile should be provided with a suitable shoe for protecting the point of the pile during driving, unless the driving is wholly in soft soils. The shoes should be truly concentric and firmly bedded on to the end of the pile. The contact area between shoe and timber should be sufficient to avoid overstressing during driving.

#### **7.4.1.3 Driving procedure**

**7.4.1.3.1 General.** Hard driving of timber piles is likely to broom the butts, crush the tips and cause cracking.

In addition to loss of structural strength, such damage may well nullify the effect of preservatives. The danger of injuring the pile during driving can be reduced by limiting the drop and number of blows of the hammer. It is desirable that the weight of the hammer should be equal to the weight of the pile for hard driving conditions and not less than half the weight of the pile for easy driving. To reduce the need for excessively hard driving, the working loads on timber piles are often limited to about 300 kN on a 300 mm × 300 mm pile. Stresses high enough to damage the pile during driving may be caused if the head of the pile is not central with the hammer and normal to the length of the pile, or if the pile runs out of position relative to the leaders.

When driven, piles should not deviate more than 75 mm from their designed positions; a greater tolerance may be permitted for piles driven over water.

Safety precautions relating to equipment for driving timber piles are described in section 11.

**7.4.1.3.2 Splicing.** When it is necessary to employ piles formed from two or more lengths, the butting surfaces should be cut square to ensure contact over the whole cross section of the pile. The joints of sawn piles should be secured by timber or steel splice plates, or by steel sections such as channels or angles welded together to form a box designed to develop the necessary strength and stiffness. Round piles should be secured with tubes. Splices near points of maximum bending stress should be avoided.

#### **7.4.2 Precast reinforced concrete piles**

**7.4.2.1 General.** The following recommendations apply to normally reinforced concrete piles.

Prestressed concrete piles are covered by **7.4.3**. Safety precautions relating to equipment for driving reinforced concrete piles are described in section 11. The piles should be straight and of uniform section and should be designed, cast, and cured to develop the strength necessary to withstand the transporting, handling and driving stresses without damage. For piles larger than 400 mm × 400 mm, an octagonal section is preferable to a square section.

Long piles present problems during handling, transporting and driving. Where long piles are required consideration should be given to the use of prestressed piles (**7.4.3**) or of precast jointed piles (Simcox and Buchanan 1980; Young and Reddaway 1980). Hollow piles provide stiffness and a larger perimeter for less weight.

#### **7.4.2.2 Materials**

**7.4.2.2.1 Concrete.** Generally, materials should be in accordance with those for main members in BS 8110 or CP 116, with the following special requirements.

- a) Portland cements complying with BS 12 or BS 4027 should be used unless another type is specified or sanctioned by the engineer.
- b) Where sulphates are found in soils or groundwater, concrete having a water/cement ratio of 0.55 or less should be used, and the recommendations of **10.4.2** on special cements should be followed.

**7.4.2.2.2 Reinforcement.** The reinforcement should be of rolled steel bars complying with BS 4449 and BS 4461, or such other reinforcement as may be suitable, having regard to yield stress, ductility, tensile strength and other essential properties.

#### **7.4.2.3 Design**

**7.4.2.3.1 Concrete.** The piles should be designed to withstand the loads or stresses and to meet other serviceability requirements during handling, pitching, driving and service in accordance with BS 8110 or CP 116.

In general, practical considerations arising from ground conditions and stresses during lifting off the bed, transport and driving tend to govern the design, of precast concrete piles.

It should be noted that very high stresses will be developed during driving. The maximum stresses will generally, but not necessarily, occur at the head of the pile. For the different conditions of driving and exposure the cement contents and strengths given in Table 12 should be the minimum employed.

**7.4.2.3.2 Reinforcement.** The reinforcement should comply with the provisions of BS 8110 or CP 116 for resisting stresses due to lifting, stacking and transport of the pile and any uplift or bending transmitted from the superstructure. It should also be capable of resisting tensile forces due to ground heave resulting from the installation of other piles close by or from excavations carried out after the piles have been installed.



The main longitudinal bars should be level at the top of the pile and should fit tightly into the pile shoe if one is used. Where special mechanical joints connect preformed pile segments the main reinforcement should terminate at a constant distance from the joint and be properly lapped with any jointing bars.

Shorter bars to resist local bending moments may be added but should be arranged to avoid sudden discontinuity of the steel which may lead to cracks during heavy driving. Joints in the main longitudinal bars, if unavoidable, should be made by full penetration butt welding in accordance with BS 5135.

The lateral reinforcement is of particular importance in resisting driving stresses and should be in the form of hoops, links or spirals. For a distance of about three times the least lateral dimension of the pile from each end, the lateral reinforcement should be not less than 0.6 % of the gross volume except where special mechanical joints or suitable crack rings, which in themselves distribute driving stresses, are used. In the body of the pile the lateral reinforcement should be not less than 0.2 %, spaced at not more than half the width of the pile. Transitions in spacing of lateral reinforcement should be gradual.

The cover over all reinforcement, including binding wire, should be as specified in Table 3.4 of BS 8110-1:1985.

**7.4.2.3.3 Pile shoes.** Piles should be provided with flat or pointed coaxial shoes if they are driven into or through ground such as rock, coarse gravel, clay with cobbles, and other soils liable to damage the concrete at the tip of the pile. The shoes can be of steel or cast iron. In uniform clays or sands the shoes may be omitted, and in these circumstances there may be no advantage in tapering the tip of the pile. The area of the top of the shoe should be such that the stress in the concrete in this part of the pile is within the safe limits (see 7.4.2.3.1).

#### **7.4.2.4 Manufacture, curing, transport and storage**

**7.4.2.4.1 Manufacture and curing.** The casting yard for all concrete piles should be so arranged that the piles can be lifted from their beds and transported to the piling frames with a minimum of handling.

The piles should be cast in a continuous operation. The casting yard should have a well drained surface to prevent excessive or uneven settlement due to softening during manufacture and curing. The concrete should be thoroughly compacted against the forms and around the reinforcement by means of immersion and/or shutter vibrators. The concrete should be continuous from end to end of each pile. After completion care should be taken to ensure that vibration from adjoining work does not affect the previously placed concrete during the setting period. The faces of the pile should be as dense as possible, and immediately on completion of the casting the top surface should be finished level without excessive trowelling.

Great care should be taken that the full specified cover is preserved and that the ends of binding wires are turned inwards.

Each pile should be marked with a reference number and the date of casting.

Freshly cast piles should be protected, after casting, for the periods given in Table 13, from sudden changes of temperature and humidity by shading and covering from hot sun and cold winds and by watering to ensure that the concrete remains humid. Insulating covers should be used against frost.

The time when piles may be moved is best established by making, each day that piles are being concreted, three control cubes from the same mix and cured in the same way as the piles. The piles may be moved when crushing tests on these cubes show strengths greater than the stresses to which the piles will be subjected in moving, allowing for dynamic effects and multiplied by a suitable factor of safety. The minimum periods from time of casting at which piles may be driven having regard to the driving stresses may also be ascertained from control cubes. In the absence of control cubes the times given in Table 13 may be used as a guide. The figures in the table are based on average conditions and on average air temperature of 10 °C (50 °F). The times should be increased for lower temperatures and may be reduced for higher temperatures. In the case of steam cured piles the recommendations of CP 116 should be adopted.



**Table 12 — Cement content and cube strength of concrete for precast reinforced concrete piles**

Conditions	Grade designation	Minimum quantity of cement		28 day characteristic cube strength		
		kg/m <sup>3</sup>	lb/yd <sup>3</sup>	N/mm <sup>2</sup>	kgf/cm <sup>2</sup>	lbf/in <sup>2</sup>
Hard and very hard driving conditions for all piles and in marine works	40	400	670	40	400	5 800
Normal and easy driving conditions	25	300	500	25	250	3 625

**Table 13 — Curing periods for use in the absence of control cubes or steam curing**

Type of cement	Minimum periods from time of casting			
	Strike side shutters	End wet curing	Lift from casting bed	Drive
	h	days	days	days
Ordinary Portland and sulphate resisting Portland	12	7	10	28
Rapid hardening Portland	12	7	7	10

**7.4.2.4.2 Transport.** Care should be taken at all stages of transporting, lifting and handling to ensure that the piles are not damaged. If the piles are put down temporarily after being lifted, they should be placed on trestles or blocks located at the lifting points. Considerations of speed and economy and limitations of space generally require precast concrete piles to be lifted from the casting bed to a stack and later to be driven with the least possible delay. On the other hand, as thorough curing and hardening require time, a proper balance should be struck in deciding the minimum intervals which should elapse between the operations of casting, stacking and driving. Safety precautions relating to the handling and pitching of precast concrete piles are described in section 11.

**7.4.2.4.3 Stacking and storing.** Piles should be stored on firm ground not liable to unequal settlement under the weight of the stack of piles. The piles should be placed on timber supports which are truly level and spaced so as to avoid undue bending in the piles. The supports should be vertically above one another.

Spaces should be left round the piles to enable them to be lifted without difficulty.

The order of stacking should be such that the older piles can be withdrawn for driving without disturbing newer piles. Separate stacks should be provided for different lengths of piles.

Arrangements should be provided to enable the piles to be watered, if necessary, to meet weather conditions or to provide further curing during storage; in hot weather the piles should be shaded from direct sunlight.

#### **7.4.2.5 Driving procedure**

**7.4.2.5.1 General.** The hammer blow generates a stress wave which traverses the length of the pile and failure, whether by compression or tension, may occur anywhere along the pile (Smith 1960).

a) *Compression.* Failure due to excessive compressive stress most commonly occurs at the head. Head stresses, which in general are independent of ground conditions, depend upon the weight of the hammer, its drop and the head cushion stiffness.

The maximum set for a given stress is obtained by using the heaviest hammer and the softest packing, the hammer drop being adjusted to suit the allowable stress in the concrete.

Since head packing materials increase in stiffness with repeated use, optimum driving conditions can be maintained only by regular replacement of the packing.

Failure in the lower sections of a pile can only occur in exceptionally hard driving where, in theory, the toe compressive stresses can reach twice the head stresses. In practice, however, this rarely occurs and, more often, the maximum compressive stress tends to be fairly uniform over a considerable length of the pile.

b) *Tension.* Longitudinal tension is caused by reflection of the compressive wave at a free end. Tensile stresses, therefore, may arise when the ground resistance is low and/or when the head conditions result in hammer rebound, i.e. with hard packing and a light hammer.

In addition, a relatively long length of pile unsupported above a hard stratum may encourage transverse or flexural vibrations which may be set up if the hammer blow becomes non-axial or the pile is not restrained.

**7.4.2.5.2 Hammer.** Piles may be driven with any type of hammer, provided they penetrate to the prescribed depth or attain the specific resistance without being damaged. The hammer, helmet, dolly and pile should be coaxial and should sit squarely one upon the other. The weight or power of the hammer should be sufficient to ensure a final penetration of not more than 5 mm per blow, unless rock has been reached. It is always preferable to employ the heaviest hammer practicable and to limit the stroke, so as not to damage the pile. When choosing the size of the hammer, regard should be given to whether the pile is to be driven to a given resistance or to a given depth. The stroke of a single-acting or drop hammer should be limited to 1.2 m, preferably 1 m. A shorter stroke and particular care should be used when there is a danger of damaging the pile. The following are examples of such conditions:

- a) where, in the early stages of driving a long pile, a hard layer near the ground surface has to be penetrated;
- b) where there is very soft ground to a considerable depth, so that a large penetration is achieved at each hammer blow;
- c) where the pile is expected suddenly to reach refusal on rock or other virtually impenetrable soil.

A spring helmet should be considered when these conditions are likely to occur.

When a satisfactory set for the last 10 blows has been achieved, repeat sets should only be carried out with caution, and long-continued driving after the pile has almost ceased to penetrate should be avoided, especially when a hammer of moderate weight is used. It is desirable that a full driving record be taken on one pile in every hundred driven, and on the first few piles in a new area.

Any sudden change in the rate of penetration which cannot be ascribed to normal changes in the nature of the ground should be noted and the cause ascertained, if possible, before driving is continued.

When the acceptance of piling is determined by driving to a set, the driving conditions when taking the set should be the same as those used when the sets of test piles are obtained.

**7.4.2.5.3 Protecting pile heads.** The head of a precast concrete pile should be protected with packing of resilient material, care being taken to ensure that it is evenly spread and held securely in place. A helmet should be placed over the packing and provided with a dolly of hardwood or other material not thicker than the width of the pile.

**7.4.2.5.4 Position and alignment tolerances.** The piles should be driven as accurately as possible to the vertical or the specified rake. For vertical piles, a deviation of 1 in 75 should not normally be exceeded, although in special cases a closer tolerance may be necessary. Straining the pile into position can damage it, and the leaders and driving equipment should be adjusted as much as possible to follow the position of the pile. Piles should not deviate more than 75 mm from their designed positions at the working level of the piling rig. Greater tolerance may be prescribed for piles driven over water and for raking piles. For piles to be cut off at a substantial depth, the design should provide for the worst combination of the above tolerances in position and inclination.

Any pile deviating beyond these limits and to such an extent that the resulting eccentricity cannot be taken care of by a redesign of the pile cap or pile ties should, at the discretion of the engineer, be replaced or supplemented by one or more additional piles (see 7.3).

**7.4.2.5.5 Risen piles.** In ground where there is a possibility of piles rising or being displaced laterally due to ground heave, the positions of the tops of the piles should be measured at intervals while nearby piles are being installed. Piles which have been displaced as a result of driving adjacent piles should be redriven to the original resistance, unless redriving tests on neighbouring piles or loading tests on selected risen piles have shown this to be unnecessary (Cole 1972; Cooke et al. 1979).

**7.4.2.5.6 Jetting.** Jetting may be used as a means of minimizing or eliminating the resistance at the toe; the frictional resistance along the surface of the pile shaft may also be reduced. By reducing the toe resistance very hard driving and vibration can be avoided and greater rates and depths of penetration can be achieved than by percussive methods. Jetting is effective in cohesionless soils, such as sand, gravel and fine-grained soils, provided the percentage of clay is small; it is not effective in, and should not be employed in, clay soils.

Jetting of piles should be carried out only with permission of the engineer and in such a manner as not to impair the bearing capacity of piles already in place, the stability of the ground or the safety of any adjoining buildings.

The quantity of water required for effective jetting is directly related to the cross-sectional area of the pile (including external jet pipes). Up to 2 L/min for every 100 mm<sup>2</sup> of pile cross section may be required at the pile in dense cohesionless soils; loosely compacted soils may require less water. The pressure should be from 0.5 N/mm<sup>2</sup> to 1 N/mm<sup>2</sup> (5 kgf/cm<sup>2</sup> to 10 kgf/cm<sup>2</sup>) or more. If large quantities of water are used it may be necessary to make provision for leading away the water that emerges at the ground surface so that the stability of the piling equipment is not endangered by the softening of the ground.

The arrangement of the jets should be balanced to ensure that the pile penetrates vertically. Independent pipes surged down or two pipes attached to the opposite sides of the pile may be used. To minimize the risk of blockages the nozzles should not be positioned at the point of the toe. Acceptable verticality may be achieved by the use of rigid leaders and allowing the pile to enter the ground gradually, after operating the water jets, under the weight of the pile and hammer, the rate of penetration being controlled by the pile winch. Once maximum apparent penetration is achieved by this method further penetration may generally be obtained in cohesionless soils by light driving whilst the water jets are running. When jetting is completed, the piles should be driven to the final penetration or set.

**7.4.2.6 Stripping pile heads.** The concrete should be stripped to a level such that the remaining concrete will project 50 mm to 75 mm into the pile cap. The effect of this projection on the position of any reinforcement in the pile cap should be considered in design. The pile reinforcement should be exposed for a length sufficient to permit it to be bonded into the cap. This should be done carefully to avoid shattering or otherwise damaging the rest of the pile. Any cracked or defective concrete should be cut away and made good with new concrete properly bonded to the old.

**7.4.2.7 Lengthening piles.** Where a pile is to have another length cast on to it before or during driving, the longitudinal reinforcement should preferably be joined by full penetration butt welding as described in 7.4.2.3.2. The concrete at the top of the original pile should be cut down to expose not less than 200 mm of the bars to avoid spalling the concrete by heat. The added bars have to be held accurately and rigidly in position during welding. Where facilities on site are insufficient to make good butt welding practicable, the joint may be made by lapping. The reinforcement at the head of the pile will need to be exposed for a distance of 40 times the bar diameter and the new bars overlapped for this distance.

Where special mechanical pile joints are used, these should have a strength equal in all respects to that of the pile sections they join. The design and installation of the joints should ensure that adjacent pile sections remain in true alignment at all stages of handling and driving and the abutting ends of the pile sections remain in close contact during driving (Bredenberg and Broms 1980).

### 7.4.3 Prestressed concrete piles

**7.4.3.1 General.** The main differences between prestressed concrete piles and normal reinforced concrete piles are as follows.

The stresses set up when handling prestressed piles of given length can be resisted by smaller cross sections and hence economy in materials may be achieved. The smaller cross section may permit or necessitate greater penetration. The bearing capacity may govern the cross section of a pile and could preclude the use of the smaller sizes that would be possible from strength considerations alone. The tensile stresses caused by the action of stress waves when driving can be reduced by the prestress. The reduction of tensile cracks should give greater durability to the pile, particularly if the pile is submerged. The piles are better able to resist, without cracking, any tensile forces set up by the working loads, whether direct or due to bending, or by accidental loads.

Prestressed piles require high-strength concrete and careful control during manufacture; usually this means casting in a factory where the curing conditions can be strictly regulated. Where piles have to be lengthened the procedure is more elaborate. There is some evidence to suggest that a larger ratio of hammer weight to pile weight is required to avoid damaging the pile.

Safety precautions relating to equipment for driving piles are described in section 11.

For certain situations, e.g. for an earth retaining structure, post-tensioned bored piles can be considered.

### 7.4.3.2 Materials

**7.4.3.2.1 Concrete.** The materials should in general be in accordance with BS 8110 or clauses 201 to 203 of CP 115:1969, with the special requirements set out in 7.4.2.2.1 of this code. The concrete should be designed and controlled in accordance with BS 8110 or clause 207 of CP 115:1969.

**7.4.3.2.2 Prestressing steel.** Prestressing steel should be in accordance with BS 8110 or clause 205 of CP 115:1969.

**7.4.3.2.3 Reinforcement.** Where ordinary reinforcement is introduced into prestressed piles, it should be in accordance with BS 8110 or clause 206 of CP 115:1969.

### 7.4.3.3 Design

**7.4.3.3.1 Concrete.** The maximum axial stress that may be applied to a pile acting as a short strut should be  $\frac{1}{4}$  (specified works cube strength at 28 days less the prestress after losses). If the ratio of effective length to the least lateral dimension is greater than 15 the stress should be reduced as shown in clause 322 of CP 114:1969. The effective length of the pile should be assessed from its end fixity conditions, and from the lateral support to be expected from material around the pile when in position. A guide to the effects of end fixity conditions is given in BS 8110 and clause 322 of CP 114:1969.

The static stresses produced during lifting and pitching should not exceed the values given in Table 1 and Table 2 of CP 115:1969 using in Table 2 of that code the values relating to loads of short duration. To allow for impact, the tensile stresses during transport, calculated as static stresses, should not exceed one-third of the values calculated as above.

**7.4.3.3.2 Prestress.** The prestress after allowing for losses of prestress should satisfy the following conditions.

- For prestress to cover handling, transporting and lifting conditions, it may be assumed that only 75 % of the full loss of prestress will have occurred within 2 months of casting.
- The prestress (in  $\text{N/mm}^2$ ) should be not less than 0.07 times the ratio of the length of the pile to its least lateral dimension, if measured in  $\text{kgf/cm}^2$ , the prestress should be not less than 0.7 times the ratio of the length of the pile to its least lateral dimension.
- The minimum prestress related to the ratio of effective weight of hammer to weight of pile should be as follows.

Ratio of hammer weight to pile weight not less than:		0.9	0.8	0.7	0.6
Minimum prestress for normal driving:	$\text{N/mm}^2$	2	3.5	5	6
	$\text{kgf/cm}^2$	20	35	50	60
Minimum prestress for easy driving:	$\text{N/mm}^2$	3.5	4	5	6
	$\text{kgf/cm}^2$	35	40	50	60

For diesel hammers the minimum prestress should be  $5 \text{ N/mm}^2$  ( $50 \text{ kgf/cm}^2$ ).

A considerably greater prestress may be required for raking piles, particularly if these are driven in ground which may tend to deflect the piles from their true alignments.

Loss of prestress should be calculated in accordance with BS 8110 or clause 304 of CP 115:1969.

**7.4.3.3.3 Prestressing wires and stirrups.** The prestressing wires should be evenly spaced parallel to the faces of the pile.

Mild steel stirrups of not less than 6 mm diameter should be placed at a pitch of not more than the side dimension less 50 mm. At the top and bottom, for a length of three times the side dimension, the stirrup volume should be not less than 0.6 % of the pile volume. The concrete cover to reinforcement should be in accordance with 7.4.2.3.2.

**7.4.3.3.4 Pile shoes.** If pile shoes are required they may be as described in 7.4.2.3.3 with sufficient space round them to pass the prestressing wires. Alternatively, they may be light shoes drilled to pass the ends of the prestressing wires.

### 7.4.3.4 Manufacture, curing and transfer of prestress

**7.4.3.4.1 Manufacture.** Prestressed concrete piles are normally cast by the "long-line" method in a factory under conditions of close control. Where piles are cast other than in a factory, casting should take place in an enclosed space at an air temperature of not less than  $10^\circ\text{C}$  ( $50^\circ\text{F}$ ). Piles should not be removed from the place of casting until after the transfer of prestress.

The piles should be cast in one operation using internal and external vibrators to assist compaction of the concrete.

Care should be taken to ensure that vibration from adjoining work does not affect the placed concrete during the setting period. Care should be taken that the head of the pile is finished plane and normal to the axis of the pile. Each pile should be marked with a reference number and date of casting. Curing should be carried out as described in 7.4.2.4.1 or the piles may be steam cured (see Clauses 304 g) and 501 g) of CP 115:1969.

**7.4.3.4.2 Transfer of prestress.** Whenever a batch of piles is cast, four test cubes should be cast and stored in close proximity to and under the same conditions of temperature and humidity as the piles.



The minimum cube strength of the concrete at transfer of prestress should be 2.5 times the stress in the concrete at transfer, or  $28 \text{ N/mm}^2$  ( $280 \text{ kgf/cm}^2$ ) for strand or crimped wire or  $35 \text{ N/mm}^2$  ( $350 \text{ kgf/cm}^2$ ) for plain or indented wire, whichever is the greater. The attainment of this strength may be checked either by testing the relevant test cubes or by allowing sufficient time to elapse after casting, provided this period can be shown to be adequate on the basis of previous test cube results and strictly controlled curing conditions. After transfer of prestress, the prestressing wires should be cut off flush with the face of the concrete or pile shoe.

**7.4.3.4.3 Stacking and storing.** For stacking and storing of prestressed concrete piles see **7.4.2.4.3**.

**7.4.3.5 Driving procedure.** Driving of prestressed concrete piles should follow the recommendations for reinforced concrete piles in **7.4.2.5**. Although the effect of prestressing is to reduce tension cracks induced by stress waves, such cracking may still occur, particularly when driving is "light", or if too light a hammer is used. A careful check for tension cracks should be made during the driving of the first piles and, if these occur the hammer drop should be reduced. If the cracks persist or recur when the full drop has to be used, then a heavier hammer should be substituted.

**7.4.3.6 Bonding of head of pile into pile cap.** The most satisfactory and most easily implemented method is stripping the concrete of the pile to expose the prestressing wires. The concrete should be stripped to such a level that the remaining concrete projects 50 mm to 75 mm into the pile cap. Where tension has to be developed between the cap and the pile, the exposed prestressing wires should extend at least 600 mm into the cap. An alternative method is to incorporate mild steel reinforcement in the upper part of the pile. After stripping the concrete this reinforcement should be bonded into the cap.

**7.4.3.7 Lengthening of piles.** Where piles have to be lengthened during driving, this may be done by one of the following methods.

- a) Where mild steel reinforcement is incorporated in the head of the pile, lengthening may be as described in **7.4.2.7**.
- b) By using a mild steel splicing sleeve together with a precast extension piece. The sleeves should be bedded on to the top of the pile with a dry sand/cement mortar, epoxy resin or other compound, and the extension piece similarly bedded on to the sleeve. It should be noted that piles lengthened in this way have a limited resistance to bending at the splice.
- c) By means of dowel bars inserted into drilled holes, the connection being made with grout or epoxy resin.

#### **7.4.4 Driven cast-in-place piles**

**7.4.4.1 General.** Driven cast-in-place piles are of two types, the first having a permanent steel or concrete casing and the second being without permanent casing. Bored cast-in-place piles are considered separately in **7.4.5**.

There are many proprietary systems of driven cast-in-place piles; careful supervision by the engineer and the contractor is always necessary to ensure that the piles are properly formed. This is particularly important in the case of piles not provided with a permanent casing. Safety precautions relating to the formation of driven, cast-in-place piles are described in section 11.

##### **7.4.4.2 Materials**

**7.4.4.2.1 Concrete.** In general, the recommendations for precast concrete piles in **7.4.2.2.1** also apply to driven cast-in-piles. The concrete mix should contain not less than  $300 \text{ kg/m}^3$  of cement. The workability, which is not to be confused with water content, should be adequate for the method of placing employed in the formation of the pile. Suggested slumps for typical concreting situations are given in Table 14 except for those piling systems requiring a dry mix.



**Table 14 — Suggested slump details for typical concreting situations for cast-in-place piles**

Typical conditions of use	Slump	
	Range	
	mm	in
Poured into water-free unlined bore. Widely spaced reinforcement leaving ample room for free movement between bars	75 to 125	3 to 5
Where reinforcement is not spaced widely enough to give free movement between bars.		
Where cut-off level of concrete is within casing.		
Where pile diameter is less than 600 mm	100 to 175	4 to 7
Where concrete as placed by tremie under water or bentonite suspension	150 to collapse	6 to collapse

**7.4.4.2.2 Reinforcement.** For reinforcement of driven cast-in-place piles see 7.4.2.2.2.

#### 7.4.4.3 Design

**7.4.4.3.1 Concrete.** The average compressive stress under working load should not exceed 25 % of the specified works cube strength at 28 days calculated on the total cross-sectional area of the pile. Where the casing of the pile is continuous and permanent, of adequate thickness and of suitable shape, the allowable compressive stress may be increased at the discretion of the engineer.

**7.4.4.3.2 Reinforcement.** The recommendations of 7.4.2.3.2 apply to driven cast-in-place piles where relevant. Cast-in-place piles may be reinforced over the whole of their length, over part of their length, or merely provided with short splice bars at the top for bonding into the pile cap. The extent of the reinforcement will depend on whether the pile is used to resist tensile or bending forces, on the type of foundation and on the possibility of horizontal or vertical ground movements due to the installation of other piles nearby or to moisture changes in the soil.

**7.4.4.4.4 Finishing pile heads.** Where a temporary casing is used, the concrete of the pile should be brought up sufficiently far above the required finished level to allow for slumping on withdrawal of the casing and to ensure that all laitance and weak concrete rises above cut-off level. The concrete

#### 7.4.4.4 Formation of piles

**7.4.4.4.1 General.** All plant, materials and operations employed in the formation of a pile should be such as to ensure that the completed pile is of the full cross section. Where reinforcement is used, care will need to be taken to ensure that it is not displaced or distorted during the formation of the pile. At all stages of the work every precaution should be taken to prevent waisting and the formation of voids in the concrete. These defects may be due to faulty consolidation, insufficient head of concrete during placing, concrete of inadequate workability, flow of groundwater past green concrete, or extracting the casing too late or too quickly.

The volume of the concrete should be checked at frequent intervals and steps taken to maintain a sufficient head of concrete above the bottom of the casing to prevent inflow of soil or water. In depositing concrete through a cage of reinforcement, care will need to be taken to prevent the segregation of cement mortar from the aggregate. During extraction of the casing observation should be kept on the surface of the concrete to detect any tendency towards lifting of the concrete.

When casings are driven to a prescribed resistance or set, relevant sections of 7.4.2.5.2 should be observed.

**7.4.4.4.2 Reinforcement.** When reinforcement is used, effective means should be employed to keep it in place with correct cover and alignment during concreting of the pile. When it is made up into cages these should be sufficiently rigid to enable them to be handled and lowered into the pile borehole safely and without damage or significant change in dimensions. Where cages are formed or extended on site by welding, the area and quality of the welds should be adequate for the forces applied during handling and under working conditions after the pile has been concreted and should be subject to normal inspection.

The bars should be openly spaced and lateral ties should be not closer than 150 mm centres so that the placing of concrete is not impeded.

**7.4.4.4.3 Damage to newly formed piles.** When cast-in-place piles are formed within a tube which is subsequently withdrawn, the sequence of driving the piles should be such as to prevent damage to any recently complete piles before the concrete has had time to harden sufficiently.

should be cast to a minimum of 0.3 m above groundwater level unless all water-bearing strata are effectively sealed off or the water excluded by other means.

Particular attention should be paid to the compaction of the concrete in the top metre or so of the pile. Any defective concrete in the head of the completed pile should be cut away and made good with new concrete well bonded into the old. The reinforcement in the pile should be exposed for a sufficient distance to permit it to be adequately bonded to the pile cap (see 7.4.2.6).

**7.4.4.4.5 Position and alignment tolerances.** The provisions of 7.4.2.5.4 apply where relevant.

**7.4.4.4.6 Risen piles.** In ground where there is a possibility of piles rising or being displaced laterally due to ground heave, the position of the tops of the piles should be measured at intervals while nearby piles are being installed. Driven, cast-in-place piles that have been displaced should not normally be redriven although it is permissible to redrive some types of piles in which concrete is placed in a permanent steel or precast concrete casing. All other types of driven, cast-in-place piles that have risen or been displaced laterally should either be replaced or the required resistance developed by preloading unless loading tests on selected piles demonstrate that the movements have not had a detrimental effect on pile performance (Cole 1972; Cooke et al. 1979).

#### 7.4.5 Bored cast-in-place piles

**7.4.5.1 General.** Bored cast-in-place piles are formed by boring or grabbing and subsequently filling the hole with concrete. Piles 600 mm or less in diameter are commonly known as small diameter bored piles. Large diameter bored piles are greater than 600 mm diameter.

There are several methods of constructing bored piles; many features of their construction are similar and not proprietary. Continuous supervision on site by the engineer and the contractor is always necessary to ensure that the piles are properly formed. Safety precautions relating to the formation of bored piles are described in section 11.

The pile diameters given in Table 15 are recommended. The actual sizes of piles may vary from the nominal sizes because of factors such as the use of equipment built to imperial dimensions and variations in the soil strata in which the pile is being formed. Design of the piles should be related to the actual size constructed rather than nominal dimensions.

#### 7.4.5.2 Materials

**7.4.5.2.1 Concrete.** In general, the recommendations in 7.4.2.2.1 and 7.4.4.2.1 also apply to bored cast-in-place piles. Where ready-mixed concrete is used, with the approval of the engineer, it should be in accordance with BS 5328. The concrete should be supplied in sufficient quantity to ensure that the concreting of each pile proceeds without interruption. In addition to meeting the strength requirements the concrete will need to have adequate workability so that it can flow against the walls of the shaft, and into every cavity. The slump should be as given in Table 14. The minimum characteristic strength requirement at 28 days should be 20 N/mm<sup>2</sup> (200 kgf/cm<sup>2</sup>).

Table 15 — Nominal pile diameters

mm		in	
300		12	
350		14	
400		16	
450		18	
500		20	
550		22	
600		24	
750		30	
900		36	
1 050		42	
1 200		48	
1 350		54	
1 500		60	

NOTE The pile diameters actually achieved will vary with the soil conditions but the nominal diameters given are commonly used in piling work.

Generally the concrete should contain not less than 300 kg/m<sup>3</sup> cement. To avoid segregation, honeycombing and bleeding, and other defects resulting from the high water content required for workability, the use of a water reducing/plasticizing admixture may be beneficial. However, to ensure the required cohesion of the mix the fines content of cement or aggregate may also have to be increased. The engineer should have access for inspection to the supplier's works at all times. The supplier's works should have adequate facilities for heating the aggregates in cold weather (Wigmore 1961).

**7.4.5.2.2 Reinforcement.** For reinforcement of bored cast-in-place piles see 7.4.2.2.2.

#### 7.4.5.3 Design

**7.4.5.3.1 Concrete.** For concrete for bored cast-in-place piles, see 7.4.4.3.1.

**7.4.5.3.2 Reinforcement** (see 7.4.4.3.2). If piles are required to resist tensile forces, the reinforcement should normally be carried down for the full length of the piles and into the enlarged bases, if any. Where the tensile forces are small, the reinforcement need only be of the length necessary to transmit fully the tensile forces. Bored piles are frequently installed from ground level and excavations carried out subsequently. Swelling of the unloaded ground causes tensile forces to be set up in the piles which may remain until construction of the superstructure is well advanced. Reinforcement should therefore be provided for tensile forces which are not expected to exist when the structure is completed.

#### 7.4.5.4 Formation of piles

**7.4.5.4.1 General.** The recommendations of 7.4.4.4 also apply to bored cast-in-place piles. In soils that are stable it will often be possible to drill an unlined hole and place the concrete without having to case the hole. In those cases where surface spoil can fall into the hole it is necessary to have a short lead-in tube. In soils liable to flow into the borehole, a casing is normally used to maintain stability and thereby to prevent the uncontrolled entry of soil and water into the borehole. Where practicable, the bottom of the casing should be kept sufficiently far below the boring to prevent the inflow of soil and the formation of cavities in the surrounding ground, which can release water into the pile concrete when the casing is withdrawn (Thorburn and Thorburn 1977). This is particularly important for pile borings carried out in close proximity to the foundations of adjacent buildings when loss of ground from beneath the adjacent foundations can occur. Where the casing cannot practicably be advanced below the bottom of the borehole, for example in compact sands and gravels, it will be necessary to use other expedients. For example, inflow of soil and groundwater can sometimes be prevented in relatively impermeable soils by a head of water. In granular soils it can be prevented by drilling mud such as bentonite suspension maintained within the boring. The supporting fluid inside the bore should be maintained at a level above the surrounding groundwater level sufficient to ensure the stability of the strata being penetrated throughout the boring process and maintained at that level until the pile has been concreted. Alternatively, the borehole can be kept stable by mixing the ground into a fluid column with dry bentonite through which the casing can subsequently be penetrated; this process is normally described as mudding-in.

When using shelling techniques, care should be taken to avoid over rapid withdrawal of the boring tools causing suction leading to excessive removal of soil into the casing. There should be adequate clearance between the shell and the casing to minimize the suction.

Dry bores should be clean. Where the piles are vertical, concrete may be poured through a funnel with a length of tube so that the flow is directed and does not hit reinforcement bars or the side of the hole. Shutes extending to near the base should be employed for raking piles of large diameter. For raking piles of small diameter this may not be practicable and an enriched mix should be used in the first few batches of concrete to minimize segregation. With a properly designed mix of suitable workability it should not be necessary to vibrate the concrete. Concrete within temporary casings should never be vibrated. Vibration or punning after casings have been removed to ensure good compaction of the top 1.5 m or so of the pile should be permitted only if the pile head concrete cannot be displaced into soft ground and contaminated by soil, or water.

In the case of friction piles in ground liable to deteriorate on exposure concreting should be carried out without delay. In circumstances where the cut-off level is below the groundwater level, the need to maintain a pressure on the unset concrete equal to, or greater than, the groundwater pressure should be observed.

Horizontal flows of groundwater can occur in certain highly permeable soils. In these situations care should be taken to ensure that the fines are not washed from the concrete of recently placed piles.

#### 7.4.5.4.2 Concreting underwater or drilling mud.

When concreting underwater or drilling mud, all soft or loose material will need to be removed from the bottom of the hole and concrete pumping or an efficient tremie technique should be used (Fleming and Sliwinski 1977). General recommendations for tremie concrete are given in 8.2.2.3.3 and the following procedures are applicable to the use of tremie concrete in piles.

- a) The concrete should be cohesive, rich in cement (i.e. not less than 400 kg/m<sup>3</sup>) and of slump not less than 150 mm.
- b) The sides of the borehole have to be stable. This may be achieved by maintaining an adequate head of fluid or by the provision of a temporary casing of the necessary length.
- c) The tremie should be watertight throughout its length and have a hopper attached at its head by a watertight connection.

d) The tremie pipe should be large enough in relation to the size of aggregate. For 20 mm aggregate the tremie pipe should be of diameter not less than 150 mm, and for larger aggregate tremie pipes of larger diameter are required.

e) The tremie pipe should be lowered to the bottom of the boreholes allowing ground water to rise inside it. It is essential to prevent the tremie concrete from mixing with water in the tremie pipe and to this end a plug or other device should be used.

f) The tremie pipe should always be kept full of concrete and should penetrate well into the concrete in the borehole with an adequate margin of safety against accidental withdrawal if the pipe is surged to discharge the concrete.

g) The pile should be concreted wholly by tremie and the method of deposition should not be changed part way up the pile, to prevent the laitance from being entrapped within the pile.

h) If the time taken, to form large piles is likely to be excessive, the use of set retarding admixtures should be considered, particularly in the case of high ambient temperatures.

i) All tremie tubes should be scrupulously cleaned before use.

j) When drilling muds such as bentonite suspension are used, the fluid at the pile base should be checked for contamination before concreting to ensure that it will be readily displaced by the rising concrete.

**7.4.5.4.3 Bentonite suspensions.** Where bentonite suspension is used to maintain the stability of a borehole for cast-in-situ piles, it is essential that the properties of the material be carefully controlled at stages of mixing, supply to the borehole and immediately before concrete is placed.

Reference should be made to section 6 and to specifications relating to the use of bentonite in bored, cast-in-situ piles and diaphragm walls.

NOTE For further information see FPS (1973) and ICE (1978).

**7.4.5.4.4 Continuous flight augered piles.** Bored piles may sometimes be formed in unstable ground conditions without the need for lining tubes or drilling suspensions by the use of continuous flight augers (Derbyshire 1984). The auger penetrates to the required depth in one continuous operation and little soil is removed from the borehole. The permanent filling (such as concrete or grout of a suitable workability to ensure satisfactory emplacement within the borehole) is then pumped under pressure through the hollow centre of the auger to the bottom of the pile and the soil carried by the flights is slowly lifted from the borehole. The borehole cannot be inspected at any stage during the formation of the piles by this method. For this reason it is particularly important to have a detailed site investigation; careful supervision is essential. The auger should not be lifted from the borehole before the required depth has been reached without measures being taken to safeguard the stability of the surrounding ground, such as pumping drilling mud or other suitable material down the hole or other approved method for ensuring stability.

**7.4.5.4.5 Reinforcement** (see 7.4.4.4.2). Where temporary casings are employed, the longitudinal reinforcement should extend at least 1 m below the bottom of the casing so that movement of the reinforcement during extraction of the casing is minimized.

**7.4.5.4.6 Damage to newly formed piles.** Piles should be formed in such a manner or sequence as to ensure that no damage is sustained by previously formed piles in adjacent positions.

**7.4.5.4.7 Finishing pile heads.** For finishing pile heads see 7.4.4.4.4.

**7.4.5.4.8 Position and alignment tolerances.** In general, the provision of 7.4.2.5.4 will also apply to bored cast-in-place piles.

A deviation of 1 in 25 from the required direction is permissible for piles with a rake of up to 1 horizontally in 4 vertically.

**7.4.5.4.9 Under-reaming.** When constructing enlarged bases, it is desirable for safety reasons to use mechanical tools which require the minimum amount of handwork in excavation and cleaning. All enlarged bases should be carefully inspected by a responsible supervisor and all loose soil and remoulded soil cleaned out.



Bulk excavation by hand should be kept to a minimum; the hole should be open for as short a time as possible because of the progressive deterioration of the soil. In very large bases, it may be necessary to provide shoring. For technical and safety reasons no enlarged base should normally be formed under water or in unstable ground. When the construction of enlarged bases involves men going down holes, it is imperative that all necessary precautions should be taken to ensure the safety of the operation with particular respect to falling debris and tools, partial collapse of the hole, ingress of water and noxious gases. The safety precautions in BS 5573 should be followed.

#### 7.4.6 Steel bearing piles

**7.4.6.1 General.** Piles in which the steel forms the principal load-carrying component are generally called steel bearing piles and can be formed from a wide variety of steel sections, for example, H-sections, pipes and prefabricated sections such as box piles, in general, steel bearing piles are driven as plain rolled steel sections but for hard driving conditions it may be necessary to strengthen the toes by welded-on shoes. The toes of pipe piles driven open-ended may be strengthened by welding on external or internal stiffening rings. Hollow piles may be filled wholly or partly with concrete or other material but concrete filling should not be used where resilience is required to absorb energy from lateral impact loading, for example, piles in berthing structures.

Where piles develop their bearing resistance mainly in a stratum such as sand of loose or medium density, they are sometimes enlarged by welded wings for the depth of the resistant stratum. However, in most circumstances it is more effective to keep the section plain and to develop the desired resistance by driving the piles deeper.

Safety precautions relating to equipment for driving piles and to the handling and pitching of steel piles are described in section 11.

#### 7.4.6.2 Materials

**7.4.6.2.1 Steel.** Steel sections, plain or fabricated, and pipe piles should comply with grades 43A and 50B of BS 4360 or other grades from that standard to the approval of the engineer.

**7.4.6.2.2 Concrete filling.** If box piles or pipe piles are wholly or partly filled with concrete and the concrete is required to carry the working load, the cement content of the mix should be not less than  $300 \text{ kg/m}^3$  and the slump not less than 75 mm. In general, the recommendations for precast concrete piles in 7.4.2.2.1 will also apply to steel bearing piles.

#### 7.4.6.3 Design

**7.4.6.3.1 Working stresses.** The cross-sectional area of the steel should be sufficient to withstand the blows of the hammer during driving and to carry the working load on the pile with an appropriate factor of safety. When the design safety factor on driving resistance is 2, the stress in the steel due to superimposed axial forces at working load should not normally exceed 30 % of the yield stress. However, where piles are driven through relatively soft soils to end bearing in very dense soils or gravels or sound rock, the allowable axial working stress may be increased to 50 % of the yield stress. When additional loads such as bending moments or negative skin friction will be applied subsequent to driving, the stresses due to the combined effects of these loads and the superimposed axial load should be limited to 50 % of the yield stress. When safety factors on driving resistance greater than 2 are required, it will be necessary to employ lower stresses in the steel at the pile working load, so that there is no risk of the yield stress being exceeded during driving. Further, if a pile has to be driven either through a hard stratum or to a specified level rather than to a set, so that the driving resistance may exceed the working load times the factor of safety, the pile should be designed to withstand this driving resistance.

If the finished pile projects above ground level, it may be necessary to limit the working stresses by the appropriate buckling coefficient. Similarly, for a pile that is likely to be subjected to very hard driving to penetrate through a hard layer at a stage where much of the pile is still above ground level, it may be necessary to take account of the buckling stresses in selecting the size of the pile. Where piles are jacked to the required penetration, no peak stresses due to impact are set up and higher working stresses up to 50 % of the yield stress can be used.

**7.4.6.3.2 Transfer of working load to pile.** The working load may be transmitted to the pile through a capping plate, dowel bars or cleats, unless an adequate length of pile is embedded in a concrete cap. Where a concrete cap is employed there should be sufficient thickness of reinforced concrete above the pile head to prevent punching failure. Any protective coating, to the steel should be omitted or removed from that part of the pile embedded in the cap. If the load is to be transmitted to the pile through concrete filling in tubular or box piles, the permissible stresses in the concrete should be in accordance with 7.4.2.3.1.



**7.4.6.3.3 Protection against corrosion.** The part of a pile that is driven into undisturbed ground below the permanent water table will not normally be subjected to significant corrosion. Where there is the possibility of corrosion, reference should be made to section 10 and BS 5493. If excessive corrosion seems possible, cathodic protection should be considered for that part of the pile which is below ground level or lowest tide level (see CP 1021).

The incidence of corrosion that may be expected can sometimes be forecast by ascertaining the deterioration of any similar foundation structures that have already been in existence for some years in the same locality.

**7.4.6.3.4 Lengthening.** If piles have to be lengthened, the normally recommended practice is to make a joint by butt-welding in such a manner that the full strength of the original section is developed. Details of welded joints and their execution should be in accordance with BS 5135, taken in conjunction with BS 449. The welds should be subject to normal visual inspection. The joints should be designed and constructed to maintain the true alignment and position of the pile sections. In some circumstances, it may be necessary to ensure that the joints are watertight in the case of pipe piles or box piles.

**7.4.6.3.5 Pile shoes.** Pile shoes are generally not required for H-piles and other plain rolled steel sections. However, if the piles are to be subjected to exceptionally hard driving, they may be strengthened at the bottom end by cast steel plates, or by welding plates or angles, to increase the thickness of the steel. Where penetration into rock is required a special rock point may be fitted. Tubular and box piles are also commonly driven without shoes, but if it is desired to close the bottom end this may be done by means of a welded flat plate or a shaped shoe of cast iron, cast steel or fabricated steel.

**7.4.6.4 Driving.** In general, the provisions of 7.4.2.5 for precast concrete piles will also apply to steel bearing piles, except that steel bearing piles are rarely susceptible to damage due to tensile stresses during handling or driving. A driving cap or helmet should be used if necessary to prevent excessive damage to the head of the pile during driving. Attention is drawn to the risk of deflection from the vertical of long piles and the development of excessive curvature. Where long piles are to be used the engineer should refer to the relevant literature (Francis et al. 1962; Hanna 1968).

## 7.5 Bearing capacity and test loading

### 7.5.1 Bearing capacity of a pile

The bearing capacity of a pile is dependent on the size, shape and type of pile and on the properties of the soil in which it is embedded. The ultimate bearing capacity is the load at which the resistance of the soil becomes fully mobilized. At a load greater than the ultimate bearing capacity the soil undergoes shear failure, allowing the pile to penetrate into the ground, to become, in effect, a different pile from that originally installed, with a greater embedded length and possibly with different soil conditions. The mobilization of the resisting stresses in the soil is accompanied by strain; the amount of movement of the pile which is required to strain the soil to the point of failure is dependent on the nature of the soil and the size of the pile. For practical purposes, the ultimate bearing capacity may be taken to be that load, applied to the head of the pile, which causes the head of the pile to settle 10 % of the pile diameter, unless the value of the ultimate bearing capacity is otherwise defined by some clearly recognizable feature of the load/settlement curve. Where appropriate, allowance may be made for the elastic shortening of the pile shaft under axial loading.

The ultimate bearing capacity of a pile may be calculated approximately by means of a dynamic pile formula using data obtained during driving the pile; or by a static formula on the basis of soil tests, or it may be determined by test loading.

By using a dynamic formula an estimate of the ultimate bearing capacity may be obtained from the driving characteristics of each individual pile, the accuracy being dependent on the reliability of the formula, the conditions of use and the data used.

By using a static formula an estimated value of the ultimate bearing capacity of a typical pile is obtained, the accuracy being dependent on the reliability of the formula and the soil strength data to which it is applied. The actual ultimate bearing capacity of any particular pile will differ from this value in so far as the soil surrounding it has properties different from those used in the calculation.

A loading test carried to failure gives the ultimate bearing capacity of the particular pile that has been tested. To infer the ultimate bearing capacities of other piles on the same site from loading tests alone requires either the prior testing of an adequate number of piles and the use of a statistical approach, or a site investigation that is sufficiently detailed to show the uniformity or otherwise of the soil, thus enabling the ultimate bearing capacities of other piles to be estimated.

When driving piles or boring for cast-in-place piles, valuable information about site conditions can be obtained. Thus a judicious combination of load testing with driving and soil records can ensure reasonably satisfactory results on most sites. The attention of the engineer should be drawn to any significant variation from the expected ground conditions and/or behaviour of the pile during driving, so that he may take appropriate action. The determination of the design load is discussed in 7.3.8.

### 7.5.2 Calculation by dynamic pile formulae

**7.5.2.1 General.** In non-cohesive soils an approximate value of the ultimate bearing capacity may be determined by a dynamic pile formula. All dynamic formulae are based on the following two assumptions.

- a) The resistance to driving of a pile can be determined from the kinetic energy of the driving hammer and the movement of the pile under a blow.
- b) The resistance to driving is equal to the ultimate bearing capacity for static loads.

There are many dynamic formulae, in all of which the energy available in the hammer at the moment of impact is equated to the work done in overcoming the resistance of the ground to the penetration of the pile. Allowances are made for the energy supplied by the hammer which is not usefully employed in moving the pile to a new position but is lost in causing elastic strains and vibrations in the pile, cap and soil. The various assumptions made in the calculation of these losses give rise to a variety of dynamic formulae (Whitaker 1976).

Driving formulae are not directly applicable to deposits such as saturated silt, mud, marl, clay, and chalk; they should be used with caution in any soil if on redriving after a period of rest the resistance has decreased. If the use of driving formulae is restricted to those piles whose bearing capacities are predominantly at the toe in gravels, sands and other non-cohesive soils of this type, then one of the more reliable formulae should give a calculated result within the range of 40 % to 130 % of the ultimate bearing capacity that would be determined by a test load. There is no completely reliable formula and, provided statistical appraisal shows a formula which is simple to use should be chosen in preference to a more complicated one. If as a result of test loadings on a given site a correcting coefficient can be applied to the formula, the results should then be of reasonable reliability for that particular site.

The Hiley formula is one of the more reliable and is probably the most commonly used in Britain (Cornfield 1963; Whitaker 1976).

**7.5.2.2 Wave equation analysis.** The ultimate bearing capacity of a pile may be predicted from analysis of the stress wave resulting from the hammer blow. The wave is damped by friction on the pile shaft and the form of the reflected wave depends on the friction, the resistance at the pile base and the properties of the pile. By the use of suitable equipment to measure forces and accelerations at the pile head and analysis of the wave-form, estimates of the dynamic resistance can be made which can be correlated with the static bearing capacity (Smith 1960; Forehand and Reese 1964; Poulos and Davies 1980; RSAES 1980).

### 7.5.3 Calculation from soil tests

The weight of the pile and the load it carries are supported by the frictional resistance or adhesion of the soil on the surface of the shaft and the bearing resistance at the end or base. The two resistances generally act together and it is customary to assume that the ultimate bearing capacity of a pile is reached when both the resistances are fully mobilized, so that:

$$Q + P = fA_s + A_b (q + p_o)$$

where

- $Q$  is the ultimate bearing capacity;
- $P$  is the weight of the pile;
- $A_s$  is the surface of the pile shaft;
- $A_b$  is the plan area of the base;
- $f$  is the average skin friction or adhesion per unit area of the shaft at the condition of full mobilization of frictional resistance;
- $q$  is the ultimate value of the resistance per unit area of base due to the shearing strength of the soil;
- $p_o$  is the effective pressure of the overburden at the level of the base.

In many cases it may be sufficiently accurate to assume that  $P = A_b p_o$  so that the equation becomes:

$$Q = fA_s + A_b q$$

The special case of negative skin friction or downdrag has been mentioned in 7.3.6. Soil strata imposing negative friction forces will introduce negative components into the  $fA_s$  term. If all the strata above the level of the pile base are liable to settlement, the term  $fA_s$  will be negative. It should then be treated as part of the design load and not be divided by the factor of safety.

For open ended driven tube or box piles in which a soil plug does not form during driving the full plan area  $A_b$  cannot be taken. Attention is drawn to tests on models and full size piles which have shown that very much larger settlements are required to mobilize base resistance than to mobilize shaft resistance (ICE 1966b). The values of  $f$  and  $q$  may be determined by full scale tests but are more usually assumed from tests on samples of the soil in the laboratory or by in situ test.

These tests are as follows.

a) *Laboratory tests on samples of the soil.* In non-cohesive soils, laboratory tests are of limited application to the calculation of the bearing capacity of piles because of the difficulty of obtaining samples of the soil either in the undisturbed state or in a state which is relevant to the conditions which apply when the pile is in position.

In cohesive soils, laboratory tests are more applicable. For driven piles wholly embedded in cohesive soil, the frictional resistance will depend on the material of the pile, the type of soil, the effect of remoulding and the time which has elapsed following driving. In soft clays and sensitive soils the adhesion to wood or concrete piles may increase in a few months; with steel piles the increase may be smaller and slower (Tomlinson 1980). In firm and stiff clays the adhesion may not increase pro rata as the shear strength of the clay increases, nor may there be any marked increase in the adhesion with time. Because of the wide range of adhesion values possible with driven piles in cohesive soils, loading tests should be made to check the design.

For bored piles wholly in clay soil in which the concrete is cast in place in contact with the clay, the lower limit of  $f$  is the fully softened shear strength of the clay (Ward and Green 1952; Skempton 1959). The actual value of  $f$  will depend on the type of clay, the amount of softening occurring during construction and the length of time the pile has been in place.

In the present state of knowledge,  $f$  and  $q$  are best determined empirically for the particular soil conditions and method of installation (Skempton 1959; Fleming and Sliwinski 1977; Tomlinson 1980). The value of  $f$  may be expressed as  $f = \alpha c$  in which  $c$  is the average undrained shear strength of the clay over the depth of the pile and  $\alpha$  is a coefficient which is dependent on the factors mentioned above and may in normal conditions have a value between 0.3 and 0.6. When the borehole is formed without appreciable delay and without significant water entering from fissures, the mean value of 0.5 may be adopted. Where softening of the borehole surface is evident, the value of  $\alpha$  chosen for design should be nearer the lower end of the range (Weltman and Healy 1978; Fearenside and Cooke 1978).

The ultimate base pressure  $q$  may be expressed as  $q = N_c C_b$  in which  $c_b$  is the shear strength of the clay at the level of the base of the pile and  $N_c$  is a bearing capacity factor. It is usual to take the value of  $N_c = 9$  but where the ratio of the depth to the diameter of the base is less than about 4, progressive reduction in the value of  $N_c$  to about 6 will be necessary. Care should be taken to ensure that the values of  $c$  and  $c_b$  are then appropriate to the shear conditions along the shaft and at the base of the pile respectively.

The ultimate frictional resistance on the shaft of a pile in cohesive soil is related to the mean effective horizontal stress on the shaft and to the drained angle of friction between the soil and the material of the shaft. The type and history of the soil and the method used to install the pile influence the values of these parameters. Widely applicable design methods based on the principles of effective stress are not yet available. However, as a guide to the reliability of values of the frictional resistance calculated from the undrained shear strength, empirical relationships between friction values and the effective stress parameters may be employed (Burland 1973; Fleming et al. 1985).

In general, the size of the pile has little influence on the principles of design but reference should be made to the literature for the case of large bored piles with enlarged bases in fissured clay (ICE 1966b; Marsland 1971b and c; Burland and Cooke 1974).

b) *In situ tests.* For piles depending mainly on point resistance, an assessment of bearing capacity may be obtained from deep penetration tests. There are a number of types of apparatus and methods of testing using falling weights or pressure to effect the penetration.

Of the dynamic methods the standard penetration test (Palmer and Stuart 1957; Peck et al. 1974; Nixon 1982) is widely used (see BS 1377).

The most commonly used static penetration test is that known as the Dutch or cone penetrometer method. In this apparatus a cone of 36 mm diameter is fixed on the end of a rod enclosed in a tube of the same perimeter as the base of the cone. The assembly is pushed into the soil at the required depth and the forces required to advance the cone and the tube independently are measured. Alternatively, the apparatus may permit the two forces to be measured simultaneously as the penetrometer is pushed steadily into the soil (Marsland 1974; Sanglerat 1979; de Ruiter 1982; Marsland and Quarterman 1982).

Where penetration tests are made they should be accompanied by soil sampling to enable the types of soil to be identified and related to the observed penetration resistance. The application of penetration tests to the calculation of the ultimate bearing capacity of piles is empirical and interpretation of the results is dependent on experience. It is preferable that the ultimate bearing capacities derived from penetration tests be correlated with loading tests.

A number of other in situ tests may be used according to the nature of the ground; such tests include those using vanes, which are largely applicable to soft cohesive soils (see BS 1377).

Plate-bearing tests are used on the ground at the bottom of holes (Marsland 1971b and c).

#### 7.5.4 Loading tests on piles

**7.5.4.1 General.** A loading test on a pile is made for the purpose of finding the settlement to be expected at the estimated working load, or some multiple thereof, determining the ultimate bearing capacity, or checking the structural soundness of the pile.

The loading tests should be carried out in accordance with one of the methods described in 7.5.5 and 7.5.6. It is undesirable to carry out loading tests in such close proximity to piles being installed that the test is affected thereby.

It should again be emphasized that the data determined in a loading test relate to an individual pile. The settlement and bearing capacity of a piled foundation constructed of similar piles in the same ground do not necessarily have a direct relation to the settlement and bearing of an individual pile (see 7.3.4.4 and 7.3.4.5).

Equipment and procedures for carrying out loading tests to determine the settlement characteristics and ultimate bearing capacity of preliminary piles and for checking the performance of contract piles are described in detail by Weltman (1980).

**7.5.4.2 Relation between settlement and load.** A relation between settlement and load for an individual pile may be obtained by applying a load to the pile in increments (Weltman 1980; ICE 1978). A pause of sufficient duration is made after the addition of each increment to permit settlement to take place until the rate of settlement decreases to a value which is acceptably small. When the settlement is measured, the load being maintained constant during the pause, the resulting graph of load versus settlement represents an approximation to equilibrium conditions, and from it the approximate settlement of a pile at a chosen working load, or multiple thereof, may be determined. The method is given in 7.5.5. Long-term consolidation settlements in cohesive soils cannot be assessed by this test.

**7.5.4.3 Determination of ultimate bearing capacity.** The loading of a test pile may be increased progressively by increments in the manner described in 7.5.4.2 in order to reach the ultimate bearing capacity. As the ultimate bearing capacity is approached it may be necessary to decrease the size of the increments. The point at which the soil fails to support the pile may not be clearly shown by the results (see also 7.5.1) and for this reason the constant rate of penetration test is more suitable (Whitaker 1963).

#### 7.5.5 Pile loading test using maintained loads

NOTE For further information on pile loading tests using maintained loads, see Weltman (1980) and ICE (1978).

**7.5.5.1 Preparation.** The pile head should be cut off or built up to the necessary elevation and should be capped appropriately to produce a bearing surface perpendicular to the axis of the pile. The arrangement should be such that none of the test load is carried by the ground under the cap.

**7.5.5.2 Method of loading.** The test load may be applied in one of the following ways:

- a) by means of a jack which obtains its reaction from kentledge heavier than the required test load;
- b) by means of a jack which obtains its reaction from tension piles or other suitable anchors.

The load should be measured by a calibrated load gauge and also by a calibrated pressure gauge in the hydraulic system. The jack and load gauge should be carefully aligned so that the load applied is co-axial with the pile.



When using method a) care should be taken to ensure that the centre of gravity of the kentledge is on the axis of the pile. The nearest edge of the crib supporting the kentledge stack should not be closer than 1.3 m to the surface of the test pile. Kentledge should not be used for tests of raking piles. In appropriate circumstances an existing structure of adequate mass and suitable construction may serve as kentledge.

When using method b) all anchor piles should be at a distance of at least three test pile shaft diameters from the test pile, centre to centre, and in no case less than 2 m. This spacing may need increasing in certain situations where knowledge of the absolute value of settlement of an individual pile is essential to design (Banerjee 1971).

Where a pile to be tested has an enlarged base the same criterion should apply with regard to the shaft and, in addition, the surfaces of anchor pile shafts should not be closer than one-half of the diameter of the enlarged base to the test pile base. If the anchor piles are to be permanent working piles, their level should be observed during application of the test load to ensure there is no residual uplift.

Where ground anchors are employed with method b), no part of the anchor transferring load to the ground should be closer to the test pile than 3 test pile shaft diameters. Where the pile to be tested has an enlarged base this criterion should apply with regard to the pile shaft and, in addition, no section of the anchor transferring load to the ground should be closer to the pile base than the base diameter.

**7.5.5.3 Measurement of settlement.** Settlement should be measured by one of the following four methods.

a) *Level and staff.* The level and the scale of the staff should be chosen to enable readings to be made to an accuracy of 0.5 mm. A scale attached to the pile or pile cap may be used instead of a levelling staff. A datum should be established on a permanent object or other well-founded structure or deep datum point. The datum should be situated so that only one setting up of the level is needed. It is preferable that the datum should be duplicated in case one is inadvertently demolished. The datum should not be affected by the test loading or other operations on the site.

b) *Reference frame.* A frame should be supported on two foundations or stakes placed sufficiently far from the pile and the reaction system as to be unaffected by ground movements resulting from the test; the distance should be not less than 3 test pile diameters and in no case less than 2 m. The foundations or stakes should be placed at a sufficient depth below ground to be unaffected by movements of the reaction system and movements of the ground caused by moisture changes or frost. The measurement of settlement is made by dial gauges fixed to the frame and bearing on the top of the pile, or lugs or other reference points on the pile to register the movement of the pile. If preferred the gauges may be fixed to the pile and bear on surfaces on the reference frame. Electrical displacement transducers can be used in place of dial gauges. Readings should be taken to an accuracy of 0.1 mm.

During the test, observation of any movements of the stakes with reference to a datum as in a) above should be made by means of a level and staff.

c) *Reference wire.* A strained high tensile wire may be used instead of the reference frame. The wire is positioned against a scale fixed to the pile and the movement of the scale relative to the wire is determined. Readings should be taken to an accuracy of 0.5 mm.

d) *Electro-optic position sensing.* A laser beam focussed by a lens attached to the pile may be used to produce an image on a photo-electric detector. Movement of the pile produces an electrical signal proportional to the movement. The foundations for the laser and detector units should be as described under b) above but it is convenient and likely to lead to greater accuracy if they are placed at least 10 test pile diameters from the test pile.

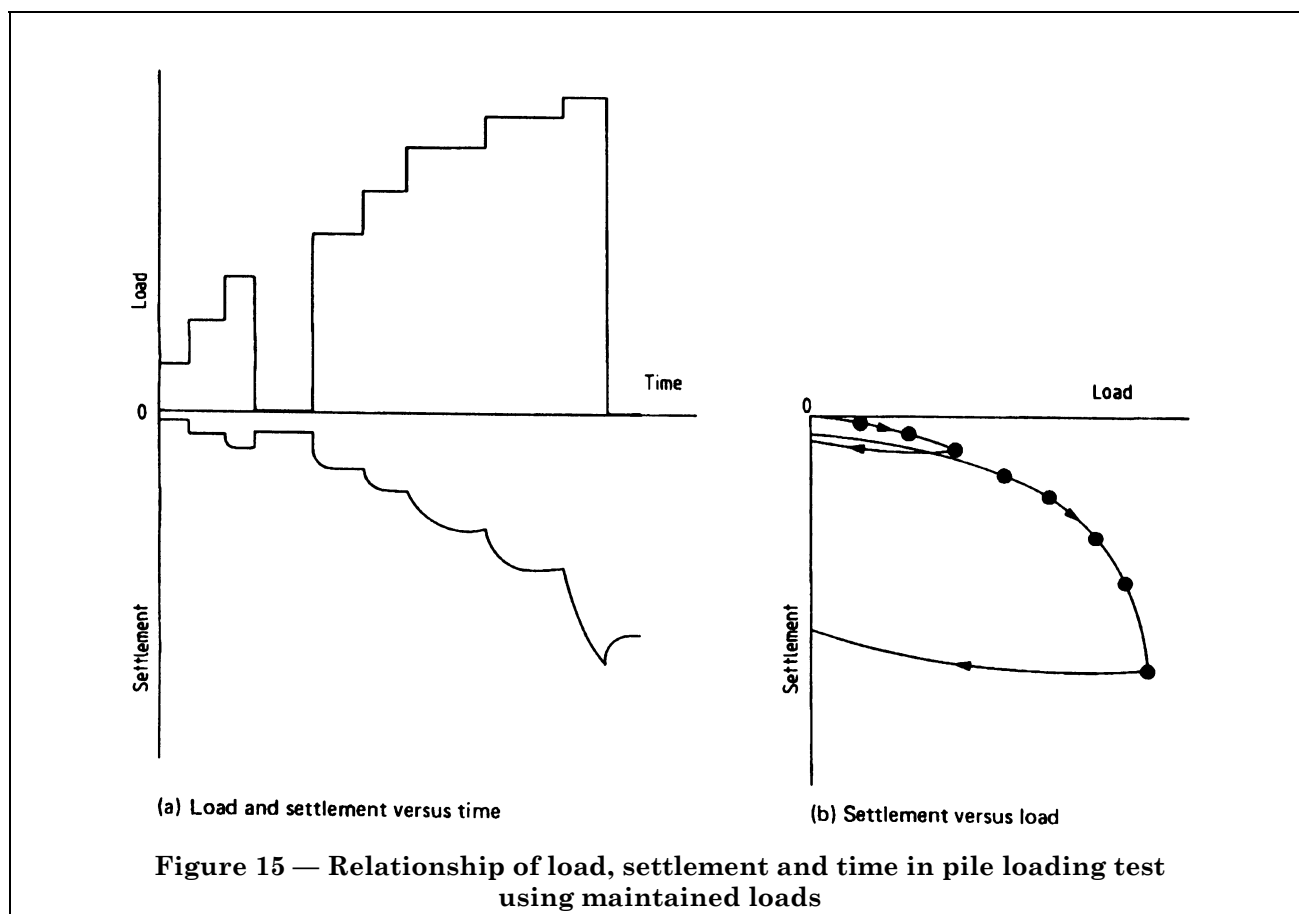
Where method b) or c) is used, protection of the frame or wire from sun and wind should be made and variations in the air temperature should be recorded.



**7.5.5.4 Procedure.** The engineer requiring the test to be performed should state the working load and prescribe the stages of loading. It is convenient to make the increments of load about 25 % of the working load, up to the working load, and appropriately smaller thereafter. The engineer should also state if the pile is to be unloaded and reloaded after the completion of any stage of loading. If it is intended to determine the ultimate bearing capacity of the pile, the engineer should make an estimate of this value to enable the necessary weight of kentledge or the appropriate reaction system to be provided.

For each stage the load or increment of load should be applied as smoothly and expeditiously as possible. A reading of the time and settlement should be taken when loading commences and readings should be made at suitable intervals as the load increases. When the load for the stage is reached, time and settlement observations should again be made. The load should then be held constant and settlement readings continued. A graph of settlement versus time should be plotted as the test proceeds, and the trend of the curve will indicate when movement has decreased to an acceptably small rate. The settlement at this point should be taken as the settlement under the applied load for the purpose of this test.

A rate of movement of 0.25 mm/h may be taken as the limiting rate for normal purposes. When the limiting rate of settlement has been reached, a further increment of load should be applied or the pile should be unloaded, whichever is required. During unloading, readings of time and settlement should be made at suitable intervals and, after removal of the load, readings should be continued until the movement effectively ceases. Where method 7.5.5.3 a) or c) is used a longer period of observation will be required appropriate to the precision of the reading. On completion of the necessary stages of loading a pair of graphs showing load and settlement versus time as abscissa should be plotted as indicated in Figure 15(a). A graph of load versus the settlement at the end of each stage of loading should be plotted as in Figure 15(b).



### 7.5.6 Pile loading test at a constant rate of penetration

**NOTE** For further information on pile loading tests at a constant rate of penetration, see Whitaker (1963) and Weltman (1980).

**7.5.6.1 Principle of test.** In the constant rate of penetration (CRP) test the pile is made to penetrate the soil at a constant speed while the force applied at the top of the pile to maintain the rate of penetration is continuously measured. As a result of the pile movement the soil is progressively stressed until it fails in shear; when that occurs the ultimate bearing capacity of the pile has been reached. It should be understood that the purpose of the test is the determination of the ultimate bearing capacity of the pile and that the force/penetration curve obtained in the test does not represent an equilibrium relationship between load and settlement. Therefore, it does not necessarily give the settlement under any maintained load.

**7.5.6.2 Preparation.** The pile head should be prepared as described in 7.5.5.1.

**7.5.6.3 Equipment.** The method requires the application and measurements of an axial force on the pile head, the measurement of pile movement and some means of controlling the speed of movement.

The reaction may be provided by kentledge supported above the pile head or a suitable system of anchor piles, as described in 7.5.5. The reaction available should be greater than the estimated ultimate bearing capacity of the pile. As the force has to be varied smoothly from zero to the ultimate bearing capacity, it is most convenient to use a hydraulic jack inserted between the pile head and the reaction system. The ram of the jack should have a travel greater than the sum of the final penetration of the pile and the upward movement of the reaction system. The pump supplying the jack may be hand or mechanically operated; if the latter, it should have infinitely variable delivery.

The jacking force should be measured by a calibrated jack and pressure gauge, and by a calibrated load gauge.

The downward movement of the pile head should be measured by means of a dial gauge carried on a beam as in 7.5.5.3. The rate of penetration may be controlled either by manually checking the time taken for small increments of penetration and adjusting the pumping rate accordingly, or by the use of a pacing device attached to the dial gauge.

**7.5.6.4 Method of testing.** The jack is operated to cause the pile to penetrate at a uniform speed. Readings of time, penetration and jacking force are made at convenient intervals of penetration which should be sufficiently closely spaced to give adequate control of the rate of penetration if the control is manual. A rate of penetration of about 0.75 mm/min is suitable for friction piles in clay. For end-bearing piles in sand or gravel, rates of penetration of 1.5 mm/min may be used. Tests have shown that the actual rate of penetration, provided it is steady, may be half or twice these values without significantly affecting the result. The test usually proceeds very rapidly and requires the services of several observers for taking simultaneous readings. The rate of penetration should be chosen to suit the available pumping equipment.

As the test proceeds a graph of force versus penetration should be made to determine when the ultimate bearing capacity has been reached.

**7.5.6.5 Ultimate bearing capacity of the pile.** The graph of force versus penetration in the case of a predominantly friction pile will be similar to one of those shown in Figure 16(a). The force may reach a maximum and decrease with larger penetrations, or the highest value may be maintained with substantially no change for penetrations between 1 % and 5 % of the shaft diameter. The value of the force reached at the points marked A in Figure 16(a) represents the ultimate bearing capacity.

The graph in the case of a predominantly end-bearing pile will be similar to that shown in Figure 16(b). In most cases the ultimate bearing capacity may be taken as the force at which the penetration is equal to 10 % of the diameter of the base of the pile. In other instances the following factors will need to be taken into account:

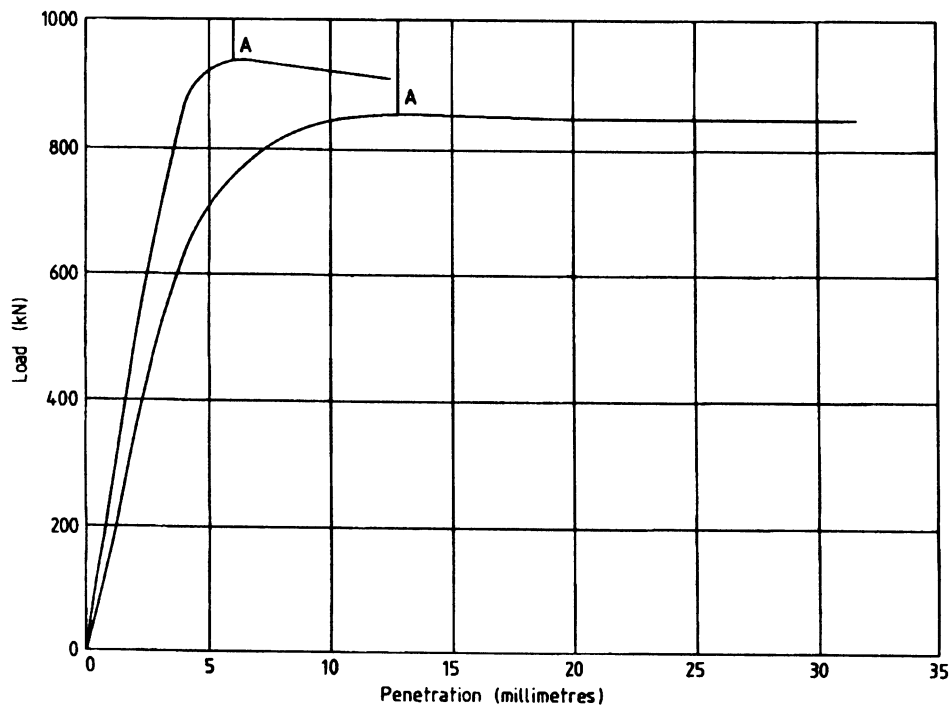
- a) the elastic shortening of the pile which for very long piles might itself reach 10 % of the base diameter;
- b) the practical difficulty of loading a large pile to a settlement as great as 10 % of the base diameter; for such piles an assessment of ultimate bearing capacity may be made from the characteristics of the load settlement curve.

## 7.6 Integrity testing of cast-in-place piles

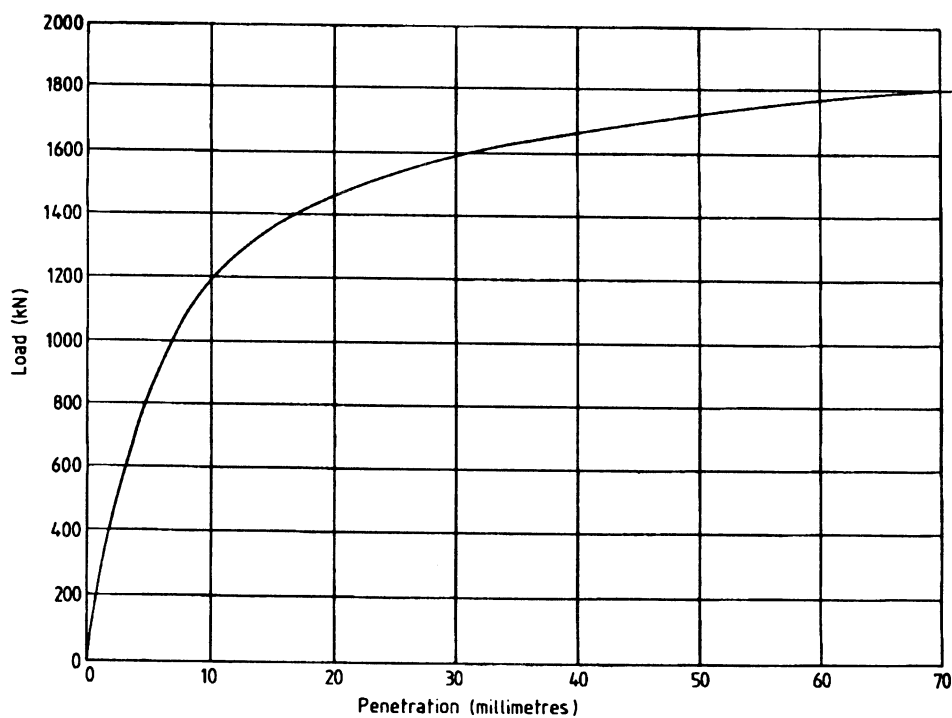
Various methods are available for checking the integrity of concrete piles after installation. In the most widely used methods impulses or vibrations are applied to the pile and measurements made of the timing and attenuation of reflected signals. Such methods may provide an indication of the soundness of the concrete but they should be undertaken by persons experienced in the method and capable of interpreting the results with specific regard to piling.

It is for the engineers to decide whether the results of the test point to the likelihood of defects being present of such significance as to affect materially the long-term load-carrying capacity of the pile. The engineer should then decide whether to carry out further examination of the pile or to take remedial action, and sound engineering judgement is called for.

NOTE For further information on integrity testing of cast-in-place piles, see Weltman (1977) and Fleming et al. (1985).



(a) Predominantly friction pile



(b) Predominantly end-bearing pile

Figure 16 — Penetration in constant rate of penetration tests

## Section 8. Tide work, underwater concreting and diving

### 8.1 Tide work

#### 8.1.1 General

**8.1.1.1 Introduction.** Tide work normally comprises intermittent construction with a large part carried out between high and low tide. According to the location of the work, the sea bed may or may not be exposed at low water.

**8.1.1.2 Applications.** Examples of structures that involve tide work are wharves, jetties, dolphins and protective works, and foreshore work such as sea walls, breakwaters, groynes, revetments and outfalls (see BS 6349).

**8.1.1.3 Conditions for tide work.** The conditions affecting tide work vary between the extremes of the sheltered conditions of an upper estuary, harbour or river and the exposed conditions of a foreshore open to ocean waves and gales. In most cases the risk of storm or flood damage has to be allowed for, and the permanent and temporary works should be designed accordingly. On exposed foreshores, all plant and materials should be readily removable to safety.

It is of particular importance, especially on navigable waterways and particularly in foggy weather, that protection from impact of passing vessels, heavy logs and other floating objects should be provided in the form of dolphins, floating fenders or other suitable protective works. Adequate warning to shipping is essential. Signs, lights and notices should be provided in the form agreed with the appropriate authority.

**8.1.1.4 Information required.** The following information may be required before a scheme involving tide work can be prepared.

- a) *Site survey.* A survey of the site and its vicinity, including in particular the nature and levels of all strata which may be affected by the works, together with the shore drift and the depth of overlying mud.
- b) *Marine data.* Records of the following, taken over as long a period as possible, including any local information applying to the site of the works:
  - 1) all available tidal information;
  - 2) direction and strength of currents during complete tidal cycles;
  - 3) extreme wind forces with directions, wave heights and flood levels, including the effect of freshwater floods on tidal currents;
  - 4) any relevant information to be derived from a study of local hydrographic charts;

- 5) particulars of marine life and pollution such as trade wastes, if any, which are likely to have deleterious effects on materials of construction;

- 6) weather records and the direction from which storms may be expected.

Although attention should be paid to extremes of tidal range, as much as possible of the work should be capable of construction within normal tidal conditions without the necessity of waiting for extreme levels to occur.

A point to be considered is whether, in winter, extreme low water on both morning and evening spring tides occurs during the hours of darkness.

c) *Local design practice.* Particulars of local practice in the design of marine structures which may have been developed to suit local conditions.

d) *Local marine regulations.* The requirements of all the authorities concerned in the particular stretch of water.

e) *Method of construction.* When carrying out the design, consideration should also be given to the proposed method of construction.

#### 8.1.2 Materials

**8.1.2.1 General.** The recommendations given in 8.1.2.2 to 8.1.2.6 apply primarily to pile structures of all kinds, as these are one of the most important applications of tidal construction. They have, however, a general application to tidal structures of other kinds.

Structures for wharves, jetties and protective works are most commonly made in timber, steel or reinforced concrete.

**8.1.2.2 Timber.** Where timber satisfies design conditions (see 7.4.1 and section 10) and is not liable to attack by marine borers, it has advantages for tidal work because it has resilience and can be cut and drilled readily in situ. Where timber is used for permanent construction reference should be made to section four of BS 5589:1979 for preservative treatment requirements.

Piles should be braced as soon as practicable after driving to avoid damage from storms or passing craft.

Bolts and fastenings should preferably be galvanized or coated with a bituminous or other suitable preservative. Contact surfaces of bolts and fastenings should be coated with a bituminous mastic before being placed together. Sinkings for bolt heads should be completely filled with similar mastic.



**8.1.2.3 Steel.** (See 7.4.6 and section 10.) Steel is at some disadvantage for marine structures in tidal waters owing to corrosion (see BS 5493) and maintenance costs. It has, however, certain advantages for jetty work in strength, resilience and speed of construction. Steel also has advantages in tropical waters, provided the corrosion element is taken into account, in locations where good concrete aggregates are unobtainable at reasonable cost and in great depths of water. Moreover, a steel structure is comparatively easy to repair when damaged. Permanent steel sheet piling is extensively used for wharves and training walls. Steel may also be employed where screw piles are necessary, although concrete screw piles can be used where conditions permit.

Bracing should be simple, robust and designed with the minimum of connections, so that water is shed quickly from the structure. In structures of moderate height it is often possible to avoid the necessity for underwater bracing by using piles of a stiff section connected by a form of portal frame above water level.

Preservative coatings are matters of special treatment (see BS 5493).

Where welded joints will be under water in marine structures, some care may be needed in selecting suitable electrodes to avoid corrosion by electrolytic action.

**8.1.2.4 Mass concrete and reinforced concrete.**

Concrete is one of the most extensively used materials for tidal work. Both design and workmanship should be suited to the conditions, and the workmanship will have to be of a sufficiently high standard (see 7.4.2 and section 10).

The work should be planned so that no more is attempted in any one tide than can be effectively completed. Concrete that is placed in situ and covered by the returning tide before it has hardened sufficiently should be of low water cement ratio and workability, dense, thoroughly mixed and compacted, and slightly richer than for similar work in the dry. Before the returning tide covers the work, the exposed surface of the concrete should be protected with polyethylene sheeting or by other means to prevent disturbance or leaching. Such sheeting should be adequately weighted or fixed. The formwork should be robust and left in contact with the concrete for a longer period than would be required above the water line. The surface of construction joints should be removed until solid concrete is reached. Reinforcement, where used, should have at least 80 mm of cover, and before concreting should be securely held against movement by wave action. In sea conditions where marine growth is rapid, placing of concrete should quickly follow the fixing of reinforcement. Attention is drawn to the increasing tendency to use precast units, where similar precautions would need to be taken at in situ junctions. The use of prepacked concrete or grouted aggregate construction may also be a useful expedient in tidal conditions, as may specially designed concrete mixes having immediate mechanical stability following compaction.

Conditions where flow of water through or over freshly placed concrete may occur should be avoided. Where such conditions exist, and for pipe setting, jointing, plugging holes and similar operations, quick-setting cement is useful, but special precautions are needed when casting in large masses.

**8.1.2.5 Bitumen and asphaltic compounds.** Infilling of foundations, mattresses or revetments in tidal zones can be effectively carried out with a wide range of bitumen mixtures. For bitumen infilling, cut-back bitumens would normally be too soft for use on slopes and penetrations between 40/50 and 80/100 would be chosen. For less permeable infilling and increased resistance to erosion, asphalt grouts and sand asphalts are used with 30/40 to 60/70 bitumens or possibly cut-back bitumen. The grading and proportions of sand and mineral filler are important in determining a mix with sufficient fluidity and heat retention for the desired penetration of the foundation.

Dense asphalts with zero voids are suitable for placing underwater. The relative density should be about 2.2 or greater.

It is essential that the bitumen or asphaltic infilling should reach a depth compatible with the size of the stones forming the face.

**8.1.2.6 Rock.** Unless the site conditions allow adequate compaction of rock fill, the grading should be sufficiently coarse and single-sized to avoid segregation on placing. Wide variations in the grading in situ caused by segregation may cause irregular settlement or, in a grouted base, render satisfactory grouting impossible.

The soundness and quality of the rock has to be equal to that required for equivalent surface work and the rock will need to be resistant to deterioration in its final environment.

Single-sized grades are most suitable for placing underwater since little more than spreading by screed is possible. Special machines for compaction underwater have been made and also bulldozers have been adapted to be operated underwater by divers but these developments have not yet become established.

### 8.1.3 Construction

**8.1.3.1 General.** Owing to the variety of conditions encountered, each job has to be considered on its merits. The method of construction adopted should be as flexible as possible so that it can be modified as the work proceeds if unforeseen difficulties are encountered.

The fundamental consideration is to make the best use of the time available. The ideal to be aimed at is for the work to follow the tide down and to be carried up in advance of the rising tide. Night work is slow and awkward, but may be economical in some circumstances. A combination of fixed and portable lighting is desirable.

Work that is only exposed on low spring tides should be planned carefully in advance, and may have to be carried out during hours of darkness in winter on certain coasts.

When work is temporarily suspended all tools, scaffolding, formwork and unfixed permanent material should be securely lashed or weighted down, especially if buoyant; otherwise they should, be removed to a place of safety.

Meteorological forecasts, when available, may be useful, particularly gale warnings, but too much reliance should not be placed upon them. A state of continual preparedness is the only safeguard.

Normal practice in concrete work is to use ordinary or rapid hardening Portland cements, but the use of special cements or admixtures may be useful in certain circumstances.

**8.1.3.2 Foreshore work.** For timber or concrete groynes the previous observations apply, and for the pitching of slopes or protection by wattles, gabions or fascines, the stability of the work and the means of prevention of damage by wave action should be studied at all times (see Civil Engineering Code of Practice No. 2).

## 8.2 Underwater concreting

### 8.2.1 Applications

Concrete can be placed successfully underwater, provided proper precautions are taken, where no other method would be possible. The major methods (see 8.2.2.3) require the concrete to have flowability, good cohesiveness and the ability to self-compact; this is achieved by an increased cement content, which permits a higher water content and thus increases the workability, and by the addition of more fine aggregate or filler to promote cohesiveness. In some circumstances the use of an admixture is beneficial. However, the design strength of concrete placed underwater should be taken as no more than two-thirds of the potential strength of the same concrete placed above water.

One of the principal applications of underwater concrete is for sealing the bottoms of cofferdams or cylinders where watertight sides can be provided but, owing to the nature of the ground, pumping out should not be employed until the upward pressure has been neutralized. In such cases the depth of concrete cast should be sufficient to give an adequate margin of safety over the upward pressure so that, after the seal has been obtained, the cofferdam or cylinder can be pumped out and the remainder of the structure built in the dry.

### 8.2.2 Construction

**8.2.2.1 Preparing foundations.** Overburden above foundation level can usually be removed by grabbing or dredging. Thin overlayers of mud or silt can be removed by divers using jets in conjunction with an airlift where necessary. All accumulations of mud or silt during the progress of the work should also be removed by grabbing, pumping or air lifting.

The importance of a clean foundation for underwater concrete is as great as for concrete in the dry. The inspection of the prepared surface, by divers if necessary, prior to concreting, is strongly recommended.

**8.2.2.2 Prevention of flow of water through concrete.** When concrete is placed in a cofferdam or cylinder, it is most important to avoid the flow of water through the concrete before it has hardened sufficiently to prevent the consequent washing out of the cement. Unbalanced water pressures may be prevented by operating sluice valves in the walls.

Water should not be allowed to flow over the fresh concrete. This is unlikely to occur in enclosed conditions, whilst in open conditions the work should be covered, as recommended in 8.1.2.4.

Screening by concrete in bags or other means causes turbulence which might increase the risk of washing-out.

### 8.2.2.3 *Placing concrete*

**8.2.2.3.1 *General.*** In general, the aim should be to place the concrete in its final position with as little disturbance or manipulation as possible. In consequence, a high workability mix e.g. slump not less than 150 mm, should be used which will flow into position without assistance. A lower slump may be more appropriate at times where a smaller size of section permits.

Laitance or slurry is likely to be increased by the use of too much mixing water, resulting in segregation. It is essential that the laitance or slurry be removed before a fresh layer of concrete is placed. Construction joints should be avoided and the concrete placed in one continuous operation wherever possible.

Underwater concrete is usually placed:

- a) in bags;
- b) by underwater bottom opening skips;
- c) by tremies or concrete pumps;
- d) by injecting grout into coarse aggregate.

**8.2.2.3.2 *Concrete placed in bags.*** Concrete in bags, which are usually made of open-weave hessian or canvas, is placed in a similar manner to that employed when building protective walls of sandbags. The bags are part-filled (about two-thirds to three-quarters) with mixed concrete and lowered to a diver, who builds them in place and flattens each bag in the process. Bags should be filled just prior to placing and their mouths should be tied up or, for the best work, sewn up. They should be built in bond with the mouth of the bag away from the outside surface. Courses of bags may be held together by driving steel spikes through them after placing. For the construction of bonded bagwork, a rich mix should be used.

Concrete for filling small pockets is sometimes sent down in bags to the diver. The diver empties the bags and puts the concrete into position. A variant of this method is to use a canvas bag about 0.5 m in diameter and 1.25 m in length, which is filled with concrete and lowered to the diver. While the bag is suspended the diver releases a slip-knot or toggle at the bottom and allows the concrete to flow out. The bags should be opened as close as possible to the final placing position of the concrete so that the flow of concrete through the water is kept to a minimum.

An “elephant’s trunk” is a similar device that is often used; this has the lower end of the bag turned up and secured by a chain.

Concrete with a high cement content is recommended for this class of work.

**8.2.2.3.3 *Concrete placed by skips.*** Concrete can be placed underwater by means of purpose-made bottom opening skips which have skirts to minimize separation of the concrete’s constituents. Bottom doors may be opened by divers or by a self-tripping device which operates as soon as the skip rests on the previously placed concrete or on the bottom. Alternatively, a hand trip wire is laid to the surface and tripped at the appropriate moment. Another method is to use a double-rope crane whereby a differential movement of the ropes opens the skip.

The aim should be to place the concrete so that it flows out from the skip on to the surface of the section previously cast causing as little disturbance to it as possible. Dropping the concrete through the water should be avoided; in this connection the self-tripping skip has an advantage. The top of the skip should be closed with overlapping canvas covers to prevent the top surface of the fresh concrete from being damaged by the swirl of water as the skip is lowered and emptied. After tripping, the skip should be raised slowly until all the concrete has flowed out.

Before commencing the day’s work, any slurry which has formed on top of previously placed concrete should be removed.

**8.2.2.3.4 *Concrete placed by tremies or by concrete pumps.*** Where large volumes of concrete have to be placed under-water, the use of tremies is the most expeditious and satisfactory method.

The tremie consists of a pipe, continuous or sectioned with a feed hopper attached, which extends well into the surface of the fresh concrete on the floor to above the water surface. As the concrete flows out more is placed in the hopper at the top, so that there is always sufficient wet concrete in the pipe, the bottom of which is below the surface of the newly placed concrete. Mobile concrete pumps with boom-controlled flexible discharge pipes may be more convenient than the conventional tremie.

The area to be concreted is covered by tremies placed at about 2 m to 3 m centres, which are replenished in sequence so that the level of the concrete on the bottom is gradually raised over the whole area. The pipes are supported by staging or by barges carrying several pipes.

Tremie pipes should not be moved laterally and effective means should be provided for raising or lowering the pipes quickly by supporting the tremie hopper from a crane. It is extremely important to ensure an adequate supply of concrete throughout the operation to the extent, in important work, of having standby plant available. Similarly, if ready-mixed concrete is used, the supply will need to be assured. The supply of concrete should be sufficient to ensure that the surface of the bay being poured rises at a minimum rate of 0.3 m/h to 0.5 m/h.

To ensure satisfactory operation, rich concrete (i.e. a mix with a cement content not less than 350 kg/m<sup>3</sup>) and having a high slump (i.e. not less than 150 mm) should be employed.

The diameters of the pipes commonly used vary from 150 mm to 230 mm according to the particle shape and size of the aggregate. For 40 mm rounded aggregate, pipes of 200 diameter are satisfactory. The lengths are usually 1 m, 2 m and 4 m and quickly detachable, watertight joints are advisable.

The receiving hopper for the concrete should have a capacity at least equal to that of the pipe it feeds.

The tremie pipes are erected over the area to be concreted with their lower ends resting on the bottom. In order to charge them, a suitable plug of buoyant material is inserted in the top of the pipe, which is then filled with concrete, the plug being pushed down by the concrete, expelling the water before it. The success of the tremie operation is very dependent on the provision of an effective plug at the commencement of the operation, ensuring that the outlet of the tremie is properly immersed at all times in the placed concrete and maintaining a continuous head of concrete in the tremie pipe (see also 7.4.5.4.2).

The rate of flow of the concrete is varied by raising or lowering the tremie pipe; raising the pipe increases the flow and lowering it decreases or stops the flow but at no time should the bottom of the pipe be raised above the surface of the concrete.

In the event of a charge of concrete in the pipe being lost, with the result that the pipe fills with water, operations should be suspended until the concrete has set. Any disturbed material should then be removed and the process restarted.

Progress is noted by taking soundings round the pipe.

**8.2.2.6 Screeding.** For roughly levelling the surface of underwater concrete, heavy screeds may be used. The screeds are worked off rails which are placed, where possible, clear of the concrete to be levelled. The rails should be supported at frequent intervals to prevent sag and wedges may be provided for

**8.2.2.3.5 Grouted concrete.** Coarse aggregate of size 25 mm or greater is first placed and compacted in position and colloidal grout is then pumped into it to fill the voids. The grout should flow upwards through the aggregate by being fed through pipes which initially reach to the bottom of the aggregate. The pipes are raised as grouting proceeds, in a similar fashion to the tremie principle. Flat or flaky aggregate is not generally suitable for grouted concrete.

Better quality concrete is produced by using colloidal grout in place of ordinary grout. The term colloidal is used to describe a special grout which possesses the property of flowing more freely than ordinary grout and which can be deposited in water without appreciable absorption of the water in which it is deposited. This grout is made by mixing cement, sand and water thoroughly in a special high shear mixer, from which it is usually pumped through a pipe to its destination. Alternatively, it can be made by the addition of certain wetting or dispersing agents to ordinary grout.

**8.2.2.3.6 Polymer modified concrete.** Concrete modified with a proprietary polymeric admixture, which almost completely eliminates separation of the constituents, can be placed using pumps or skips. Discharge is above the placement and concrete is allowed to flow through the water in a cohesive stream. Such modified concrete can result in a placement which is free from trapped water, disturbed layers and slurry.

**8.2.2.4 Formwork.** Formwork should be assembled above water in units as large as possible, weighted if made of timber, and then lowered and fixed. Where formwork has to be built up in place, a convenient method is to drive piles and attach it to them. On an uneven bottom, gaps under the formwork can be closed by concrete in bags.

**8.2.2.5 Use of reinforcement.** Reinforcement in underwater concrete should be avoided wherever possible.

If unavoidable, it should consist of heavy sections, widely spaced so as not to impede placing the concrete. It should be wired, bolted or welded together into cages above water and lowered into position in units as large as practicable.

A minimum cover of 80 mm of dense concrete is essential, but greater cover is advisable.

adjustment of level. These screeds are manipulated by a diver at each end. The calmest conditions should be selected for screeding.



## 8.3 Diving

### 8.3.1 General

Diving is an operation of such a character that it should be undertaken only by those knowledgeable and experienced in its practice. Details of diving are outside the scope of this code.

NOTE 1 For descriptions of diving apparatus and the theory and practice of diving see Davis (1951) and Bishop and Flett (1962).

NOTE 2 For regulations governing diving operations attention is drawn to Statutory Instrument No. 399 The Diving Operations at Work Regulations 1981 and CIRIA Underwater Report 23.

NOTE 3 The attention of the engineer is drawn to adverse conditions in which a diver has to work and which can apply serious restraints to his capabilities and performance.

### 8.3.2 Safety precautions

For safety precautions when diving, see section 11.



## Section 9. Site preparation for foundation work

### 9.1 Preliminary considerations

#### 9.1.1 General

Prior to any excavation being considered or undertaken there should be sufficient and adequate knowledge of the soil profile and groundwater conditions. If there is any doubt on this information, full details should be ascertained before work at site is put in hand. Difficulties can arise owing to a lack of such knowledge which can, more often than not, outweigh the apparent advantages of making assumptions to save time and money. Such assumptions often turn out in the long run to be incorrect and the actual costs then offset the initial saving. It should be emphasized that ground is often "variable" on most sites and that nothing should be assumed in so far as the soil and groundwater conditions are concerned. Detailed reference should be made to BS 5930 and more particularly to Section 12 of BS 6031:1981.

#### 9.1.2 Archaeological finds on construction sites<sup>4)</sup>

The Ancient Monuments and Archaeological Act, 1979, provides archaeologists with a statutory opportunity to investigate sites which are considered to be of historic interest before development or redevelopment takes place.

A notification period of 6 weeks is required before any excavation can be carried out in areas that have been designated of archaeological importance.

Development may be delayed for a further period of 18 weeks to allow archaeologists to complete the appropriate exploratory work.

Contact should therefore be made with the archaeological authorities at outline planning stage in order to avoid unnecessary delay to the contract. A joint approach can often be arranged between the archaeologist and the contractor in order to minimize disruption of the construction programme.

Notwithstanding the provisions of the preceding paragraphs, all fossils, antiquities and other objects of interest of value which may be found on site or in carrying out excavations in the execution of the works should (so far as may be) remain or become the property of the site owner, and upon discovery of such an object the contractor should:

- a) use his best endeavours not to disturb the object;
- b) cease work if and in so far as the continuance of work would endanger the object or prevent or impede its excavation or its removal;
- c) take all steps which may be necessary to preserve the object in the exact position and condition in which it was found; and
- d) inform the resident engineer or clerk of works of the discovery and precise location of the object.

### 9.2 Design of foundations

When preparing his designs the engineer should keep constantly in mind the site conditions and the constructional problems that will have to be overcome during the carrying out of the work. For example, where excavation involves temporary strutting, the process of design should include the problems of fixing reinforcement when the excavation is upheld by temporary strutting, the advantages of making retaining walls self-supporting and the effects on the permanent construction if strutting loads are to be carried temporarily. It is a help to those constructing the work if the designer indicates, either by drawing or by specification or both, the sequence of operations and forces involved which he has anticipated. In certain cases where the design necessitates a particular sequence of operations, these should be indicated on the drawings or in the specification.

### 9.3 Stability

The overall stability of the site includes the stability of adjoining lands and structures. It should be recognized that immediately ground is disturbed the equilibrium of everything nearby (including underground services) may be affected. One of the most important factors to be borne in mind is the water condition of the ground; the stability of the ground can be significantly changed by varying the water condition. In fine-grained soil, such as sands, silts and clays, a change in the water content may change the stability characteristics fundamentally (see 6.4).

The stability of the bottom of the excavation requires careful attention, particularly when upward water pressure exists. This aspect of excavation is dealt with in detail in BS 6031.

<sup>4)</sup> Assistance may be obtained from the County Museum or Planning Office. In case of difficulty the Inspectorate of Ancient Monuments, Department of the Environment, Fortress House, 23 Savile Row, London W1X 2BT or the Council for British Archaeology, 112 Kennington Road, London, SE11 may be contacted.

The effects of heave due to the removal of overburden on the stability of the structure close to the excavation and on the construction of foundations and basements in the excavation should be considered.

#### 9.4 Drainage

Drainage is necessary to maintain stability of slopes and precautions are necessary to prevent surface water from entering and eroding the face of the excavation. Measures will need to be taken to prevent the build-up of pressures during the construction period which may have harmful effects upon the structures, whether they are temporary or permanent.

#### 9.5 Work preliminary to demolition, shoring and underpinning of existing structures

##### 9.5.1 Notice to adjoining owners

Before site work is commenced, it is advisable, and in certain areas mandatory, for the person responsible for undertaking the work to inform the adjoining owners of his intentions.

##### 9.5.2 Site survey

Before such site work is undertaken, an adequate survey of adjacent structures which may be affected by the intended work should be undertaken.

Such a survey should include:

- a) site boundaries and building lines;
- b) conditions of adjoining and neighbouring buildings and recording of defects;
- c) extent of vaults, tunnels, and similar underground structures and a set of photographs should be taken at the time of survey;

NOTE The value of the photographs is enhanced if means of indicating the magnitude of the defects are included, e.g. an engineer's measuring scale.

- d) location and condition of utility services;
- e) the nature, quality and legal definition of the walls abutting the site (e.g. party or external);
- f) likelihood or otherwise of archaeological finds.

##### 9.5.3 Nuisance

A certain amount of noise and dust is inevitable in all these operations, but this will have to be kept to a minimum to avoid annoyance to the public and the risk of injunctions which would interfere with the progress of the work.

#### 9.6 Demolition

Demolition work should only be undertaken by persons competent and experienced in such work and should be carried out in accordance with BS 6187.

#### 9.7 Shoring

##### 9.7.1 General

All shoring should be designed or approved by a competent and experienced person. All shoring should be so positioned as to leave clear the space needed for the work of demolition and all subsequent building operations and, if necessary for subsequent operations, be so designed that part may be safely removed without affecting the whole. It should be subject to periodic inspection.

##### 9.7.2 Adjacent buildings

The removal of portions of a building during demolition usually results in making other parts unsafe, and it is therefore necessary to make a careful survey before commencing work to decide which portions will need securing before demolition commences. If the building adjoins other buildings it will have to be ascertained and agreed with the adjoining owners whether the walls are party or external walls. In either case the adjoining buildings need to be given such lateral support as is given by the building to be demolished, and it is often advisable to erect the requisite shoring before demolition proceeds too far. Care should be taken that such shoring is placed in positions that will not interfere with the erection of a new building.

##### 9.7.3 Location of shoring

At the earliest possible stage, shores or ties should be erected (in agreement with the adjoining owner when necessary) in such positions and, for raking shores, to such depths as will afford adequate support to the existing structures during building operations. Special attention should be given to all external angles of existing structures, which should be shored or tied back to a suitable anchorage. In many old buildings party walls are not bonded to external walls but are slotted into the walls to give an appearance of being bonded. Any supporting walls should be left in position for as long as possible, being raked back so as to form buttresses to existing property.

### 9.7.4 Openings

When openings in walls and piers are being cut the stability of the wall depends upon the ability of the wall material to arch over the opening to be made. There will need to be enough wall material each side of the opening to form an abutment to resist the thrust from such arching. If, therefore, the opening is near the end of the wall, it would probably be advisable to erect a raking shore against the end of the wall to resist the horizontal force induced by this thrust. Where an opening is near the base of a wall, the effect of redistribution of loading on the ground should be checked.

### 9.7.5 Ties

Where it is not possible to erect shores, it may be possible to fix ties to give equivalent restraint to the walls, but such ties need careful designing; in particular, the weight at the opposite end of the tie should be greater than, and certainly not less than, the thrust which it is desired to restrain. It is no use endeavouring to tie a wall to a floor which is resting on the wall, or indeed to any floor, unless it can be proved that the floor itself is stronger than the wall to be supported.

### 9.7.6 Maintenance

All timbering in shoring is liable to swell or shrink, and means of adjustment should be provided by hardwood folding wedges, which should be secured to maintain them in position without making future adjustment impossible.

All timber shoring should be inspected regularly by a competent and experienced person.

Particular attention should be directed to the effects on shoring supports of:

- a) freezing and thawing of the ground;
- b) softening of soils;
- c) movement of soil due to nearby excavation;
- d) effects of vibration from traffic and/or adjoining machinery;
- e) damage by impact;
- f) vandalism.

### 9.7.7 Types of shoring

**9.7.7.1 General.** Shoring systems should be structurally designed and can utilize steel sections, tubular scaffolding, timber or a combination of these. It is important that substandard materials are not used and that erection of shoring is supervised by experienced personnel.

**9.7.7.2 Raking shores.** A raking shore consists preferably of one or more whole timbers. Each shore should consist of a sufficient number of members so arranged that the centreline of each member, extended, intersects the centre of the wall opposite a floor of the existing building. The spacing of raking shores should depend on the quality and condition of the wall but should not exceed 6 m, horizontal cross-bracing may be necessary depending on the condition of the wall supporting the raking shore.

A needle of hardwood or steel should be inserted in the wall immediately below the respective floor levels to transfer the weight to the members of the shore.

Long raking shores may be more economically designed and constructed either with a combination of steel and timber or entirely of steel members.

The rake of the shores should generally be not more than 75° to the horizontal.

**9.7.7.3 Flying shores.** Where the distance between two buildings is about 9 m, timber flying shores may be used, but for greater distances it is advisable to use structural steel members or tubular scaffolding or a combination of timber and steel.

Flying shores should be needled into the walls as described in **9.7.7.2**.

Intermediate support in the form of towers may be necessary where a wall is unable to support the full weight of a flying shore or where deflection in the shore may result in unacceptable forces being imposed on the wall.

Where the floor levels of two buildings are not in the same horizontal plane, it will generally be necessary to select the floor level of one of the buildings as a datum for the horizontal member, and to provide a stiffer vertical wall piece at the other end to transmit any thrust to the floors of the other building.

Where a series of shores is erected in a continuous line, diagonal bracing may be fixed between each set of shores.

**9.7.7.4 Needles and dead shores.** Needles and dead shores are sometimes employed to support the structure when the lower sections of a wall have to be cut away. Before cutting away the wall to insert the needles, all openings and recesses in the wall should be adequately braced. The needles should be positioned as close as possible to the points of maximum load, and the holes for the needles should be made as small as possible. Provision should be made for bedding the wall upon the needles in such a way as to avoid overstressing the wall material. It is usually desirable to employ means, such as wedges or jacks, to transfer the load of the wall to the needle before cutting away the lower part of the wall.

Dead shores to support needles should be founded either directly on the ground or on another solid substructure. The loading should be distributed in such a manner as not to exceed the safe bearing capacity of the ground or substructure.

**9.7.7.5 Anchorages.** All shores should be adequately anchored and capable of resisting transverse movements.

## 9.8 Underpinning

### 9.8.1 General

**9.8.1.1 Feasibility.** Before underpinning is resorted to, the fullest possible investigation should be carried out by an experienced and competent person to determine whether an underpinning procedure will achieve the object intended.

Before considering the system of underpinning to be employed, the loads to be carried, the sensitivity of the structure and the ground and working conditions should be determined. This will facilitate the choice of available systems to meet these conditions (Prentis and White 1950; Hunter 1952).

The object of underpinning is to transfer the load carried on a foundation from its existing bearing level to a new level at a lower depth. This operation may be necessary for any or all of the following reasons:

- a) excessive movement of the foundation due to subsidence or heave;
- b) to increase the load-bearing capacity of the foundation;
- c) to allow the adjacent ground level to be lowered;
- d) to change the support system.

**9.8.1.2 Ground conditions.** Investigations of the ground and groundwater conditions below and adjacent to the structure should always be made before underpinning work is commenced. Such investigations should aim at discovering:

- a) the conditions responsible for any excessive movement so that appropriate underpinning measures will be taken;
- b) the general nature of the ground, particularly if the adjacent ground is to be lowered;
- c) the load-bearing capacity of the ground on which the underpinning is to be founded;
- d) the disposition and effect of adjoining foundations.

Until this information is available, the design of the underpinning cannot be completed.

**9.8.1.3 Structural stability.** The structure to be underpinned should be carefully examined for indications of differential foundation movement and for inherent weaknesses which may be accentuated during the process of underpinning, and the structure should be temporarily supported or strengthened as described in 9.7. Particular care should be exercised when underpinning piers, columns or walls pierced by openings.

### 9.8.2 Initial precautions

**9.8.2.1 Load reduction.** Before excavation is commenced, the live loads on the wall, pier or structure should be reduced as much as practicable where they are large in relation to the dead load, the adjoining owner's consent being obtained where necessary.

**9.8.2.2 Excavations.** All excavations necessary for underpinning should be well timbered and strutted or otherwise secured, e.g. by an appropriate geotechnical process (see section 6), to prevent the surrounding ground from moving before it is supported by the permanent work. Concrete poling boards should be used if the poling boards cannot be withdrawn. Excavations for access to construct underpinning should be similarly treated and restricted to that essential for the work. The support removed when opening up underpinning excavations will increase the load on adjoining foundations unless it is fully compensated by shoring. It is very important that these adjoining foundations are not weakened or undermined by the excavation of access trenches or approach excavations.

**9.8.2.3 Control.** Before underpinning is commenced a comprehensive inspection, carefully recorded and photographed if possible, should be undertaken to identify the condition of the structure which has to be underpinned and the condition of those structures adjacent to it.



When underpinning, frequent checks for movement or distress in the structure should be made. In most cases careful inspection is adequate but in special circumstances a detailed schedule of dilapidations should be prepared. Crack monitoring, levelling and plumbing may be necessary as the work proceeds.

### 9.8.3 Types of underpinning

Types of underpinning include:

- a) extended strip footings (continuous underpinning constructed in sections usually not exceeding 1.2 m);
- b) piles drilled through the foundations;
- c) ground beams beneath walls formed by stooling or needling and supported by piers or piles;
- d) ground beams beside walls supported by piers or piles with needles and/or load transfer by stressing or other details to transfer the load to the new support.

### 9.8.4 Design

Underpinning systems should be designed in accordance with good soil mechanics practice taking into account all vertical and lateral loads imposed upon them, including particularly transient conditions that may arise during construction. Underpinning may alter or deepen only part of a foundation if only part is to be underpinned or is shown to be unstable. The designer has to be satisfied that the whole foundation including that part modified by underpinning will continue to perform satisfactorily. If the changed support conditions will inevitably give rise to excessive differential movement, jacks may be installed temporarily or permanently to correct this. Such elaborate arrangements are rarely required except when underpinning particularly heavy walls or piers. If the whole of a structure is subsiding unevenly, for example a building on a bed of made up ground of variable thickness, partial underpinning will be unsuccessful and the whole structure has to be underpinned.

### 9.8.5 Traditional underpinning procedure

**9.8.5.1 General.** Where underpinning will not give rise to excessive ground pressure the procedure should be to execute underpinning in a series of legs, the length of each leg depending on the general character and condition of the structure to be underpinned, the intensity of the loading and the nature of the ground below. Generally, in brick and/or stone walls of normal type, each leg should be 1 m to 1.4 m in length; in walls capable of arching, the length of each leg may be increased accordingly.

Each series of legs should be planned to provide sufficient support between the legs under construction, and to ensure that the loads from the unsupported portions of the wall are distributed throughout the length of the wall. Attention should also be given to positions of openings and piers immediately above the foundations, so that sections of the structure carrying local heavy loads are not left unsupported. No fresh series of legs may be commenced until the preceding underpinning is completed and finally pinned. It is recommended that the sequence in which the underpinning is carried out should be executed in such a way as to avoid differential settlement and ensure load transfer.

Where underpinning is required on account of the settlement of the foundation, or where the safe ground pressure is likely to be exceeded during the underpinning operations, the structure to either side of an underpinning leg should be needled as described in 9.7.7 and the load transferred to temporary bearings on ground capable of carrying the additional load or other support. The siting of these bearings should be determined to avoid undermining by any operations necessary to complete the underpinning leg. When this is complete, underpinning can commence.

**9.8.5.2 Base.** The construction of the underpinning leg should be commenced immediately after the bottom of the excavation has been exposed. In all cases where the bottom is likely to be affected by exposure to the atmosphere, the last few millimetres of the excavation should not be taken out unless the underpinning can proceed forthwith. The bottom of the excavation supporting the underpinning should be sealed with concrete immediately after inspection has shown it to be satisfactory.

**9.8.5.3 Method.** Before the construction of a leg, the underside of the old wall or foundation should be cleaned and levelled ready for the new pinning. All joints against legs already constructed should be thoroughly cleaned. The underpinning leg should be constructed as quickly as possible up to within 75 mm to 150 mm of the underside of the old foundation ready for final pinning. The top of the new work should be left smooth and flat to facilitate the final pinning.



As soon as the concrete or brickwork is strong enough to support the load to be placed upon it, the final pinning should be carried out. It is recommended that the final pinning should consist of a fairly dry concrete rammed in hard to make solid contact with the soffit of the underpinned structure. "Fairly dry" means that only sufficient water has been added to moisten the mixture so that it will remain a ball when squeezed in the hand; the maximum size of aggregate should be 10 mm.

If the width of the foundation to be underpinned is greater than 1 m it is advisable to leave the outer half more than 150 mm below the underside of the old foundation to facilitate the pinning up of the furthest part.

### 9.8.6 Materials

Where underpinning involves the use of concrete, steel, brickwork, cement mortar or a combination of these materials, the quality of the materials should comply with the appropriate specifications given in other sections.

### 9.8.7 Current developments

An increasing range of alternatives to traditional underpinning methods now exists. There are developments in beam and pier construction and many new mini-piling underpinning systems, particularly for domestic structures.

**NOTE** Reference should be made to relevant literature including a Building Research Digest on mini-piling, currently in preparation. An engineer unfamiliar with this type of work is well advised to seek specialist advice when considering alternative underpinning solutions (Thorburn and Hutchison 1985).

## Section 10. Durability of timber, metal and concrete structures

### 10.1 General

This section describes how timber, steel and concrete employed in foundation structures may be expected to deteriorate in different circumstances and indicates some of the steps which may be taken to prolong their life.

### 10.2 Timber

#### 10.2.1 General

Timber is a very suitable material for both permanent foundations and temporary work. It is widely available, easily worked, strong for its mass, and capable of supporting high loads. When correctly used it can provide very long service either on land or in water, in both temperate and tropical countries. It is, however, subject to attack by certain organisms which, under the right conditions, cause rapid and extensive damage. This can be effectively resisted to a large extent either by the use of naturally durable species of timber or by preservative treatment. Apart from meeting the strength requirements, the most important step in the design of timber foundations is to ensure either that the timber has adequate natural durability in the environment in which it is to be used to meet the design life of the structure, or that an adequate level of the right kind of preservative treatment is specified, and that the timber is of a kind that is sufficiently permeable to accept it (see also 7.4.1).

#### 10.2.2 Wood-destroying organisms

Certain timbers for foundations may be subject to attack by one or other of the common wood-destroying organisms, namely, wood-rotting fungi, termites or marine borers, depending on the country and on the particular situation in which it is used.

#### 10.2.3 Fungi

Timber used in contact with the ground or any other environment where it can become permanently or intermittently damp in contact with oxygen is liable to be attacked by wood-destroying fungi. The resistance to such an attack will depend upon the natural durability of the timber species concerned and whether or not it has been treated with a wood preservative. Timbers with a high natural durability or those adequately preserved can have a useful life for many decades. In addition, the temperature of the environment will affect the rate of decay in an infected timber component. Untreated timber of low natural durability which may last for several years in temperate conditions can last for less than a year in tropical conditions.

All timber will remain free from decay indefinitely if kept below a moisture content of 22 %; because wood-rotting fungi are aerobic organisms decay cannot develop without a small supply of free oxygen. No decay will occur in timber buried in the ground below the lowest permanent water table. In these situations, therefore, piles require no protection against decay.

#### 10.2.4 Termites

In the warmer countries, foundation timber will be subject to attack by one or other of the numerous species of subterranean termites. Termites are active throughout the tropics and subtropics, in both wet and arid or desert regions; some species occur in the warmer parts of temperate countries as, for example, in southern France, but they are not found in the colder parts of these regions, apart from one or two exceptional instances. Under favourable conditions they are very destructive. In the tropics, non-durable timbers in contact with the ground would be destroyed in a few weeks. There are, however, a number of timber species which are resistant to termites and which would last many years under these conditions. Termites are not found in the British Isles.

#### 10.2.5 Marine borers

In most parts of the world, timber in the sea or in brackish water is subject to attack by a number of marine borers. In the tropics they can do severe damage in a few months, unless the timber is one of the few resistant species such as greenheart. In temperate countries, attack is generally much slower and often sporadic, but, nevertheless, except for resistant species, timber is likely to be destroyed in a few years. Marine borers have been stimulated in the UK by cooling water discharge and sometimes industrial effluent causing a rise in temperature of the water. There are three main types of marine borers, each of which attacks timber in a characteristic way:

- a) various species of the genera *Teredo* and *Banksia*, commonly known as shipworm;
- b) *Martesia*;
- c) various species of the genus *Limnoria*, commonly known as gribble.

Shipworm and gribble occur all over the world, but *Martesia* is found only in tropical waters. Further information is available in Building Research Establishment Technical Note 59.

### 10.2.6 Protection of wood

**10.2.6.1 General.** A considerable measure of protection against all these wood-destroying organisms can be obtained either by selecting species of wood that are naturally resistant to their attack or by applying preservative treatment. Information on the resistance of the common commercial timbers to wood destroyers will be found in “A handbook of softwoods”<sup>5)</sup> and “A handbook of hardwoods”<sup>5)</sup>, published by the Building Research Establishment which also contain the other main properties of these timbers.

NOTE See Oliver (1974) and BRE Digest 296.

These references also include information on the response of the various wood species to preservative treatment, i.e. their treatability. The choice in practice is between certain hardwoods possessing a high natural durability but which are impermeable to preservative fluids and a range of less durable timbers which can be treated with preservatives in order to offset their lower natural durability.

**10.2.6.2 Natural durable timber.** The information on resistance to the various wood destroyers applies to the heartwood only of the timber; sapwood, even of very durable species, is rapidly attacked by all wood destroyers. When using durable timbers it is therefore essential to specify heartwood only and to make sure that no sapwood is included. With a few timbers, for example greenheart, this can be a little difficult since the heartwood and sapwood are not easy to distinguish, but it is most important that this distinction is made, otherwise premature failures are bound to occur. If the exclusion of sapwood is not practically possible, and it is felt that the amount of sapwood present would lead to the premature loss of integrity or strength of the component, the timber should be treated as recommended in **10.2.6.3**. The actual life that can be obtained from any timber will depend also on other factors such as size and especially on the particular environment in which it will be used and on the activity of the wood destroyers in the locality. It is always advisable, therefore, to try to obtain further local advice on the performance of resistant species from either the Timber Research Laboratory or the Forest Service in the country concerned.

**10.2.6.3 Preservative treated timber.** Timber for permanent foundation work with a design life of up to several decades needs a high standard of protection, and this can be obtained only with preservatives applied under pressure. The only two preservatives suitable for this work are coal-tar creosote complying with BS 144 and copper/chromium/arsenic water-borne preservatives complying with BS 4072-1. Although the latter type is water soluble when applied, it becomes highly fixed once in the wood and can be used quite safely for timber in contact with the ground or water without fear of leaching. Both these types of preservative are effective against wood-rotting fungi, termites and marine borers.

Treatment with coal tar creosote should be carried out generally in accordance with BS 913 and treatment with copper/chrome/arsenic type preservatives in accordance with BS 4072-2. Detailed treatment schedules for timber in contact with fresh or sea water are given in Section 4 of BS 5589:1978. Other information pertaining to the treatment of piling, etc. may be found in BS 5268-5. These standards are pertinent to end-use in the UK, but could be used for other temperature regions.

A number of timbers will accept very high levels of treatment and such material can be protected for a very long time, even when exposed to the most hazardous conditions. However, many are more resistant to treatment and this can seriously limit the degree of protection that can be obtained. Information on the treatability of the various species is given in the handbooks mentioned and in BRE Digest 296.

In selecting timber for piling, it is essential to choose a species that will accept a high standard of treatment. This is particularly so where the timber is to be used in the tropics or under other conditions of high hazard. It is not always easy to do this because the large size and length of timber usually required for piling seriously limit the choice. In such cases, by far the best solution is to use timber in the round. Very much higher standards of protection can then usually be obtained, since the outer band of sapwood of round timber is much more amenable to treatment than the heartwood and, when treated, can provide a deep and uniform zone of heavily-protected wood. The use of treated round piling is standard practice in the USA and in other countries, and its use is to be strongly recommended on grounds of durability.

<sup>5)</sup> Available from the Building Research Establishment, Building Research Station, Garston, Watford, Herts WD2 7JR.

All cutting, drilling, boring, etc. operations on timber for foundations should be carried out before the timber is submitted for preservation treatment.

### 10.2.7 Temporary work

Timber for temporary work, in use for only short periods of up to about a year, is not likely to deteriorate seriously under temperate conditions, although it is advisable not to employ timbers graded as perishable for this purpose. In the tropics, however, serious damage can occur in a year, and timber with an adequate level of durability will have to be selected or it will need to be protected by some form of preservative treatment.

### 10.2.8 Advisory bodies

It is recommended that, where necessary, advice be sought from the Timber Research and Development Association, Stocking Lane, Hughenden Valley, High Wycombe, Bucks HP14 4ND and the Princes Risborough Laboratory, Building Research Establishment, Princes Risborough, Aylesbury, Bucks HP17 9PX.

## 10.3 Metals

### 10.3.1 General

Metallic corrosion, as encountered with the civil engineering use of metals, can be regarded as a form of environmental degradation in which for example, galvanic or bimetallic and bacterial corrosion as well as the rusting of structural steels are all electrochemical processes; details of these can be found in the many texts (Shreir 1976) and need not, therefore, be considered in this code.

This code is concerned mainly with the use of metals that are driven into or buried in soils or immersed in water. Some corrosion in these environments will be inevitable, although the rate of corrosion will be variable depending on the chemical stability of the metal in its service environment. The latter may itself be variable over a given structure as, for example, in the marine zoning of waterfront structures and in the variable nature of fill-soils in trenches, excavations and reclaimed or made-up grounds. Undisturbed natural soil strata, irrespective of type or chemical composition, do not, from a corrosion viewpoint, constitute a variable environment.

Guidance on the various forms of corrosion that might affect the civil engineering use of metals is given in the following sections and because structural C-Mn steels feature predominantly in this context; the guidance given highlights the most recent corrosion researches on these steels based mostly upon examinations of actual structures. It is essential that when considering the effects of corrosion on the useful life of a structure, the actual stress value at the point where corrosion occurs is related to the predicted corrosion at that point. Frequently, maximum corrosion and maximum stress do not coincide at the same point in a structure.

### 10.3.2 Corrosion between dissimilar metals

Corrosion can occur at the contact surfaces between dissimilar metals. Where, as is usual, steel is the dominant structural material, other metals are unlikely to be involved but where this is not the case, metals such as copper and nickel will aggravate the corrosion of steel while others, such as zinc, will reduce it. The danger of bimetallic corrosion is greater on seawater immersed structures and least in nominally dry atmospheric conditions.

Guidance on safe and potentially dangerous contacts between different metals is given in PD 6484.

### 10.3.3 Bacterial corrosion

Corrosion may be caused by sulphate-reducing anaerobic bacteria, which are widely distributed in soils and waters. Researches on steel piled structures show that in marine environments, bacterial corrosion almost always occurs to some degree but that it is insignificant compared with other corrosion mechanisms and causes of metal loss. Romanoff (1962); Beckwith (1979); Eadie (1979); Eadie and Kinson (1980); Morley and Bruce (1983) all report insignificant bacterial corrosion of steel piling under a range of environmental conditions even where bacterial activity was thought to be a particular problem.



Where bacterial corrosion occurs, the steel is usually deeply pitted beneath a black ferrous sulphide corrosion product. When this corrosion product is removed, the pitted metal underneath is usually bright, indicative of an active state of corrosion. Anaerobic bacterial corrosion should not, however, be confused with the visually similar corrosion of steel under reduced oxygen concentration levels as might occur beneath marine fouling, especially at mean low water level where a loose, flaky, black corrosion product also overlies bright steel surfaces. Bacterial corrosion tends to be localized and whereas pitting and perforation, even of pinhead size, of a buried pipeline is a serious matter, the same damage on a steel pile is rarely of structural significance.

Cathodic protection is effective in preventing bacterial corrosion but to be cost-effective, paints are also used to reduce impressed current demand or sacrificial anode consumption. Expert advice should always be sought on the design and installation of cathodic protection systems and also on the choice of any paints as an unsuitable choice of the latter may lead to loss of adhesion from cathodic disbondment.

#### 10.3.4 Stray electric current corrosion

Stray direct electric current installations can cause corrosion of metals and interfere with the effectiveness of any cathodic protection system. If the damage is confined to small areas, for example, to defects in protective coatings, serious localized pitting can result. Generally, stray alternating currents do not damage iron or steel seriously in this way.

Further guidance may be sought in any of the standard texts on corrosion, for example Uhlig (1948).

#### 10.3.5 Corrosion of mild steel

In most normal conditions of use, steel will be exposed to one or more of the following environments as, for example, on some waterfront structures. In such cases, the corrosion will be variable over the limits of the structure. In most natural environments, steel does not corrode uniformly and it is, therefore, important to consider this when predicting appropriate corrosion rates.

a) *Corrosion in soils*. It is most important to distinguish between corrosion in undisturbed natural soils and that in disturbed or fill-soils such as are encountered in the backfilling of trenches, excavations and land reclamation schemes.

In undisturbed natural soils, which also include beaches, river, lake, sea and harbour beds, the corrosion rate of steel driven into such soils is negligibly small, irrespective of the types of soil encountered. Such soil properties as chemical composition, resistivity, redox potential and pH value, are of no help in predicting the likely corrosion rate of steel and their use for such soils is likely to lead to the overestimate of corrosion rates. This view emerges from the many investigations of extracted steel piles that have been made over the past 20 years (Romanoff 1962; Shreir 1976; Morley 1978; Beckwith 1979; Eadie 1979; Eadie and Kinson 1980; Morley and Bruce 1983).

These investigations show that the residual thickness of steel piling driven into undisturbed soils remains within rolling thickness tolerances even after many decades of service. The best estimates that can be made of steel corrosion rates imply thickness losses of 1 mm to 2 mm over 100 years and in such circumstances, the use of protective systems cannot be justified. The isolated corrosion pitting at the groundwater table level reported in the literature (Romanoff 1962), has, in no recorded instance, been of any structural significance.

In disturbed grounds, the use of redox potential, soil resistivity and pH values, may be of some value in predicting corrosion rates providing that some experience of actual corrosion rates in the soils in question is available to "calibrate" the soil parameter measurements. Studies have been carried out, notably by Romanoff (1957); Booth et al. (1967) from which the general guidelines shown in Table 16 emerged based on the recommendations of the latter.



**Table 16 — Resistivity and redox potential values**

	Aggressive soil	Non-aggressive soil
Resistivity, $\Omega$ cm and/or Redox potential at pH = 7, V (N.H.E.) <sup>a</sup>	< 2 000  < 0.400 or < 0.430 if clay	> 2 000  > 0.400 or > 0.430 if clay
Borderline cases to be resolved by water content by the percentage by mass of water in the soil	> 20	< 20

<sup>a</sup> V (N.H.E.) is the normal (or standard) hydrogen electrode to which all potentials (V) are referred.

It has to be stressed, however, that these recommendations refer to the aggressivity of disturbed soils. Romanoff (1962), in summarizing the results of steel piling driven into undisturbed soils, encountered soil resistivities which varied from 300  $\Omega$  cm to 50 200  $\Omega$  cm and soil pH values which varied from 2.3 to 8.6, despite which, the amount of corrosion was not sufficient to affect significantly the strength or useful life of piling as load-bearing structures irrespective of the soil types encountered.

Ground anchors are often used in foundation engineering and reference should be made to DD 81.

The use of unsuitable filling materials, such as unweathered ashes or industrial wastes in contact with uncoated steel, may give rise to significant corrosion. Sand, chalk or limestone are strongly recommended as preferred backfill materials.

b) *Corrosion in water.* The corrosion rate of bare steel in water depends upon a number of factors. The marine environment is undoubtedly the most complex and variable in its effect on corrosion rate. A freshwater environment is less complicated but there are a number of important factors to consider.

1) *Corrosion in seawater.* In the marine environment, steel may be exposed to harbour or sea-bed soil strata and upwards through the complete seawater immersion zone into the intertidal and finally, the marine atmospheric zones, in each of which the corrosion rate of steel will be different (Morley and Bruce 1983). Below sea-bed level, the corrosion rate will be typically less than 0.01 mm/year to 0.02 mm/year on total thickness and in the full seawater immersion zone, the corrosion rate is unlikely to exceed 0.08 mm/year for each side exposed to seawater (0.16 mm/year on total thickness losses). Between mean low and mean high water levels, the corrosion rate will be similar to that in the fully immersed zone

at 0.08 mm/year per face exposed to seawater between tides. However, there are two marine exposure zones where the corrosion rate may exceed the values quoted, these are located just above mean high water level (the splash zone) and between low water neap and low water spring tide levels where the corrosion rate is typically 0.15 mm/year per exposed face and somewhat higher in severely exposed marine environments. In the sheltered environments of enclosed harbours, the splash and low water zone corrosion rates are generally much less than 0.15 mm/year and even slight oil pollution will inhibit very considerably the corrosion of steel piling.

With tubular or box section bearing piles and backfilled sheet-piled retaining walls which are exposed directly to the marine environment only on their external faces, corrosion occurs almost entirely on the exposed face of the pile. Corrosion on the internal surface is comparatively slight even in the presence of retained seawater either from natural seepage or when the piles were first driven. The importance of this is that the annual corrosion rates quoted in the preceding paragraph then refer, with sufficient accuracy, to the total annual loss of section thickness when hollow sections are used. This advantage is not shared by H bearing piles except below sea or harbour bed level.

2) *Corrosion in freshwater.* In freshwater, the corrosion rate for steel fully immersed will be typically 0.05 mm/year and negligible below river or canal bed level. In freshwater from carboniferous strata, calcite deposition on steel piled structures is possible, in which case steel corrosion rates may eventually fall to very low values comparable with below ground values of 0.01 mm/year.

Where steel piling is placed in canals or rivers having essentially constant water levels, a narrow band of localized corrosion, often less than 300 mm wide, may occur just below water level. The amount and distribution of this localized corrosion will be variable depending upon such factors as oxygen availability and pollution level. Normally, the rate of corrosion will be in the range 0.05 mm/year to 0.10 mm/year although in exceptional cases, corrosion rates as high as 0.25 mm/year to 0.30 mm/year have been recorded. Where past experience shows that corrosion rates may be high, some form of corrosion protection may be necessary depending upon the life required from the piling, initial section thickness and stress levels.

c) *Corrosion in the atmosphere.* Corrosion in the atmosphere can be very variable and for unprotected steel, the corrosion rate will normally be in the range 0.05 mm/year to 0.10 mm/year per exposed side, the higher value being for marine or severely polluted industrial atmospheres. Protective coatings can, however, be used and maintenance painting carried out as necessary or at planned maintenance intervals. The use of slowrusting “weathering” steels containing copper and other alloying additions may be beneficial. These require a period of up to about 4 years to develop a protective rust patina.

d) *Corrosion in unusual conditions.* Unusual soil, water or atmospheric conditions may give rise to significant corrosion. Thus, for example, special consideration should be given in the following circumstances, some of which have been discussed in the preceding sections:

- 1) in tropical environments;
- 2) where aggressive chemicals are present as a result of ground contamination or atmospheric pollution;
- 3) where sulphate reducing bacteria occur in disturbed soil conditions;
- 4) where steel is in contact with other metals and materials;
- 5) in the splash and low water zones of marine structures;
- 6) where abrasion damage from shingle, sand and silt laden waters is possible or where floating fendering systems may abrade piling and the action of ships propellers in shallow waters may disturb harbour bed material causing abrasion damage and loss of steel thickness.

Contact of steel with wood used, for example, as rubbing strips on jetty structures, can cause higher than normal corrosion rates because of crevice corrosion effects between the wood and steel interfaces. Stand-off wooden rubbing strips are to be preferred. The risks of using unsuitable backfill materials have already been mentioned and similar concern should be given to landfill and ground contamination by previous industrial or other use of the site.

### 10.3.6 Steel with copper or other low alloy content

The inclusion of a copper content in steel has not been found to have any measurable effect in reducing corrosion in soils or water (Committee for Waterfront Structures 1975; Morley and Bruce 1983). A copper content in the range 0.25 % to 0.35 % may effect some useful improvement in atmospheric zone corrosion resistance but in splash zone environments, the benefits, if any, are marginal.

There are specifications available for a wide range of low alloy steels with claimed improvements in marine corrosion performance. Alloy combinations appropriate to the fully immersed zone do not, however, necessarily work effectively in the marine splash zone and, therefore, the choice of these alloy steels tends to be a compromise between conflicting requirements. For a variety of technical reasons, these low alloy steels are more expensive to make and depending upon the alloy additions, there may be welding difficulties and some unsatisfactory aspects of mechanical properties.

## 10.4 Concrete

### 10.4.1 General

Good quality concrete has satisfactory durability for many purposes, but for some applications consideration should be given to the effect of certain disruptive agencies on concrete below ground or in sea or freshwater. The extent to which precautions are required depends considerably on the particular site conditions, so that detailed recommendations cannot be given. Much will depend on first hand knowledge of the ground conditions surrounding the concrete; where there is any doubt a ground investigation should be undertaken together with an analysis of the ground water. Particular care is needed with old industrial sites or tips.

Durable concrete requires an appropriate specification such as that given in BS 8110, together with good workmanship. Where the quality of workmanship is in doubt, precast concrete can have advantages as it is produced in controlled conditions.

#### 10.4.2 Sulphate attack

Sulphate salts may occur in solid form in the natural soil, contaminated ground, made ground or in dissolved form in groundwaters or sea water. The rate of attack for a particular type of cement depends on the concentration of the solution, the groundwater conditions and the permeability of the concrete. The method of construction has a significant effect on the rate of attack as it determines the age at which the surface is exposed to a sulphate environment, the workability requirements of the concrete, the curing arrangements and the degree of inspection which is possible. These factors are discussed in greater detail in BRE Digest 250. The importance of superficial attack depends on the type and mass of the structure. The recommendations for the type and quality of concrete to be used for natural soils when the cement is of ordinary Portland or sulphate-resisting Portland types are given in Table 17 which is taken from BRE Digest 250. Although this table should be used in consideration with the foregoing factors determining rate and importance of attack and method of construction. It is important to note that the cement contents given in Table 17 are the minimum recommended and that some types of construction, e.g. cast-in-situ piles, may require higher cement contents to achieve sufficient workability at the maximum free water/cement ratio recommended. High slag content Portland blastfurnace cement concretes and ordinary Portland cements with additions of pulverized-fuel ash (p.f.a.) are given in Table 17.

To resist sulphate attack it is essential that concrete is dense and well compacted. This is a more important quality than high cube strength. Unshuttered concrete in aprons and foundations to seal walls should not be placed at a steeper slope than 1 in 3. For steeper slopes than 1 in 3, an extremely dry mix is necessary and decay by sulphate attack follows unless top shuttering is used. Soil permeability is an important factor in corrosive attack on foundation concrete. The ease with which the contaminated groundwater can move around and be replaced is all important. Where foundations are built in an impermeable clay soil, acid or sulphate attack only penetrates the concrete to such a small extent that the incorporation of a few centimetres of dense "sacrificial concrete" will obviate the need for special cements.

#### 10.4.3 Acid attack

Well compacted, impermeable concrete, particularly if made with limestone aggregates, is resistant to low concentrations of acid, but strong solutions will attack concrete made with all types of cement. Heath and moorland water usually contains organic acids from plant decay and free carbon dioxide which may slowly dissolve cement from any concrete surface against which it flows. Porous concrete may be significantly affected and therefore benefit from a protective membrane but dense uncracked concrete will have less need for protection (Gutt and Harrison 1977, Bartholomew 1979).

The use of supersulphated cement is applicable here also but again the advice of the manufacturer should be sought.

Table 17 — Concrete exposed to sulphate attack

NOTE The recommendations are for concrete in a near-neutral groundwater; for acid conditions see Gutt and Harrison (1977).							
Class	Concentration of sulphates expressed as SO <sub>3</sub>			Type of cement	Dense fully compacted concrete made with aggregates complying with BS 882 or BS 1047		
	In soil		In ground-water		Minimum cement content <sup>a</sup>	Maximum free water/cement <sup>a</sup> ratio	
	Total SO <sub>3</sub>	SO <sub>3</sub> in 2 : 1 water : soil extract					
1	%	Less than 0.2	g/L	Less than 0.3	Ordinary Portland cement (OPC) Plain concrete <sup>b</sup> Rapid hardening Portland cement (RHPC), or combinations of either cement with slag <sup>c</sup> or p.f.a. <sup>d</sup> Reinforced concrete Portland blastfurnace cement (PBFC)	kg/m <sup>3</sup>	
						250	0.70
2	0.2 to 0.5	1.0 to 1.9	g/L	0.3 to 1.2	OPC or RHPC combined with minimum 70 % or maximum 90 % slag <sup>e</sup> OPC or RHPC combined with minimum 25 % or maximum 40 % p.f.a. <sup>f</sup> Sulphate resisting Portland cement (SRPC)	330	0.50
						310	0.55
						290	0.55
3	0.5 to 1.0	1.9 to 3.1	g/L	1.2 to 2.5	OPC or RHPC combined with minimum 70 % or maximum 90 % slag OPC or RHPC combined with minimum 25 % or maximum 40 % p.f.a. SRPC	380	0.45
						330	0.50
4	1.0 to 2.0	3.1 to 5.6	2.5 to 5.0	SRPC		370	0.45
5	Over 2	Over 5.6	Over 5.0	SRPC plus protective coating <sup>g</sup>		370	0.45

NOTE 1 Different aggregates require different water contents to produce concrete of the same workability and therefore a range of free water/cement ratios is applicable to each cement content. In order to achieve satisfactory workability at the specified maximum free water/cement ratio it may be necessary to increase the cement content above the minimum specified.

NOTE 2 Within the limits specified in this table, the use of p.f.a. or slag in combination with sulphate resisting Portland cement (SRPC) will not give lower sulphate resistance than combination with cements complying with BS 12.

NOTE 3 If much of the sulphate is present as low solubility calcium sulphate, analysis on the basis of a 2 : 1 water extract may permit a lower site classification than that obtained from the extraction of total SO<sub>3</sub>. Reference should be made to BRE Current Paper 2/79 for methods of analysis and to BRE Digests 250,275 and 276 for interpretation in relation to natural soils, fill and hardcore.

<sup>a</sup> Inclusive of content of p.f.a. or slag. These cement contents relate to 20 mm nominal maximum size aggregate. In order to maintain the cement content of the mortar fraction at similar values, the minimum cement contents given should be increased by 50 kg/m<sup>3</sup> for 10 mm nominal maximum size aggregate and may be decreased by 40 kg/m<sup>3</sup> for 40 mm nominal maximum size aggregate.

<sup>b</sup> When using strip foundations and trench fill for low rise buildings in class 1 sulphate conditions, further relaxation in the cement content and water/cement ratio is permissible.

<sup>c</sup> Ground granulated blastfurnace slag (see BS 6699).

<sup>d</sup> Selected or classified pulverized-fuel ash complying with BS 3892.

<sup>e</sup> Per cent by mass of slag/cement mixture.

<sup>f</sup> Per cent by mass of p.f.a./cement mixture.

<sup>g</sup> See CP 102.

#### 10.4.4 Chloride content

Whenever there are chlorides in concrete containing embedded metal there is an increased risk of corrosion of the metal. The presence of high concentrations of chlorides may also adversely affect the sulphate resistance of concrete.

It is recommended that the total chloride content of the concrete mix arising from the aggregate together with that from any admixtures and any other external source should not exceed the limits, expressed as a percentage relationship between chloride ion and weight of cement in the mix, as given in BS 8110.

#### 10.4.5 Industrial tips

Conditions found in industrial tips are generally the most difficult to deal with as far as the protection of concrete is concerned. The groundwater is sometimes a dilute acid and sulphates may be present in high concentrations, particularly in ground near colliery waste tips. Unless effective measures can be taken to remove the chemical impurities in the soil or permanently lower the water table to below the level at which the concrete will be placed, then only complete isolation of the concrete by an adequate thickness of suitable inert material will give the protection likely to be required (see BS 6543 and Barry (1983)).

#### 10.4.6 Frost attack

While buried concrete is unlikely to be subject to frost attack, consideration should be given to the effect of freezing and thawing on any concrete partially exposed to the atmosphere. Where necessary, the entrainment of approximately 5 % of air in the concrete is recommended to improve the frost resistance of concrete made with Portland cement having a maximum aggregate size of 20 mm (see BS 8110 for other maximum aggregate sizes).

#### 10.4.7 Corrosion of reinforcement

To ensure durability and stop corrosion, reinforcing steel should be surrounded with an adequate thickness of good quality, well compacted, homogeneous concrete, free from honeycombing or other defects. The protection of reinforcement depends upon the quality of the concrete, its compaction and impermeability as well as the amount of cover. The parts of a structure most susceptible to the corrosion of embedded steel are those exposed to intermittent wetting and drying, especially by sea and moorland water. Nominal cover to reinforcement for various conditions of exposure and grades of concrete are given in BS 8110.

#### 10.4.8 Unsuitable aggregates

The majority of aggregates are likely to be suitable for concrete subject to chemical attack. However, it is desirable for concrete subject to abrasion, such as by beach gravel carried in sea water, to be made with aggregates of adequate mechanical strength. Certain aggregates in the presence of moisture are known to react with the soluble alkali content of the concrete, causing expansion and disruption of the concrete. However, the number of incidents of this nature in the UK is very small. There is at present no British Standard test for the reactivity of aggregates with alkalis but some guidance is given in BRE Digests 237 and 258. The general solution to the problem is to exclude moisture and limit the total water soluble alkali content of the concrete to less than 3.0 kg/m<sup>3</sup>.

Some aggregates, especially in Scotland, are susceptible to excessive drying shrinkage and reference should be made to BRE Digest 35.



## Section 11. Safety precautions

### 11.1 General

#### 11.1.1 Statutory requirements

Attention is drawn to the responsibilities of all persons engaged in construction works in respect of health and safety. The Health and Safety at Work etc. Act 1974 extends the obligations and protection covered by previous legislation to include all persons at work. These obligations are in addition to the duties of employers and others under existing health and safety legislation. The Health and Safety at Work etc. Act 1974 requires that employers ensure the health, safety and welfare at work of their employees and that their undertakings are conducted in such a way that persons not in their employment, who may be affected, are not exposed to risks to their health or safety. The self-employed person also has responsibilities in this respect in that he has to conduct his undertaking so as not to expose himself and others to risks to their health and safety. In addition, the persons conducting such operations have a duty to advise others who may be affected by the way in which the works are carried out, giving information relating to any aspects of the undertaking that might affect their health and safety.

Attention is drawn particularly to the Factories Act 1961 and to the Regulations made under the Act which apply to construction work and may affect any of the operations involved in preparing foundations.

Foundation works associated with the construction of a building or a defined engineering structure constitute a "Building Operation" (BO) or a "Work of Engineering Construction" (WEC) by virtue of the definitions in Section 176 of the Factories Act 1961 and the associated Extension of Definitions Regulations. Section 127 of the Factories Act 1961 applies a number of its provisions to BOs and WECs and therefore to associated foundation works.

The main Regulations made under the Factories Act 1961 that are directly relevant to foundation works are the four Construction Regulations Statutory Instruments which apply to all BOs and WECs and are listed in appendix D.

#### 11.1.2 Precautions

Safety is an attitude of mind and those responsible for the safe operation of any construction works should not only be fully conversant with the legislation referred to in 11.1.1 but should actively encourage a safe approach to any operation. The principal causes of accidents can be categorized as follows:

- a) falls of persons (from temporary works, into excavations, etc.);
- b) falls of materials (collapse of the sides of excavations, tunnels, material falling into excavation from spoil tips, etc.);
- c) unintentional collapse of the whole or any part of a building or structure and of structures erected as temporary works (this includes the effect of excavations alongside existing buildings or structures and inadequate timbering of excavations);
- d) lifting operations and unfenced machinery;
- e) fires and explosions;
- f) electrical;
- g) conditions leading to asphyxiation resulting in accidents;
- h) accidents due to lifting and carrying, poor means of access, bad housekeeping and striking against objects.

Unsafe places of work contribute to falls of persons and materials. There is also a risk to persons arising from the unintentional collapse of structures occurring due to poor engineering execution and insufficient attention being paid to temporary supports or shores.

The design and construction of temporary works associated with foundations should reflect particular care in the selection of working stresses and load assessment, for example allowance for back-fill and surcharge loads. Reused materials may bear the effects of fatigue, heat, chemical deterioration, animal or bacteriological action, organic or ultraviolet deterioration, deformation or previous adaptations where loss of cross-sectional area has occurred.

Temporary structures should be designed to accommodate all calculable loads with an adequate safety factor that is not less than that for permanent works and the calculations should take full account of the tolerances adopted. Consideration should be given to increasing the factor of safety if the design or the equipment used has novel features or if there are special uncertainties about the loads encountered in practice or if novel methods of erection are used.

It is essential, that a thorough check is carried out on the design of temporary works by a competent designer, particularly where a major works may involve the assembly of several parts designed independently. It is recommended that a temporary works co-ordinator is appointed to ensure that all the procedures and checks have been carried out and that the works are constructed in accordance with the design.

NOTE Attention is drawn to the need to examine access scaffolds, excavations, shafts, earthworks, tunnels, cofferdams and caissons in accordance with the Construction Regulations at the required time intervals and record such examinations on the prescribed forms (see appendix D).

The wearing of suitable clothing for the job is another major aid to safety. All persons who work on construction sites should be encouraged to wear suitable safety clothing which includes helmets, boots or shoes and gloves. Injuries to the head, hands and feet are frequent but equally serious accidents cause painful damage to eyes.

NOTE Attention is drawn to The Protection of Eyes Regulations 1974 and the Abrasive Wheels Regulations 1970.

Ear protection should be worn where appropriate, particularly when working in confined spaces with pneumatic or electrical tools.

Engineers, foremen, chargehands and work people should attend recognized safety courses and should be encouraged to become proficient in elementary first aid. A qualified supervisor needs to be appointed in accordance with the requirements of the Construction Regulations (see appendix D). The Regulations require that first aid boxes, maintained fully equipped and in the charge of a responsible person, are kept on all sites. A trained first aid man is required on sites employing more than 50 persons.

Public safety should also be considered by those persons undertaking construction work. Fencing or other effective means should be provided to prevent unauthorized access or trespass on to the site. All open pits and boreholes should be securely covered or otherwise protected. Particular care should be taken to avoid accidents to children who may trespass on the site. If the entrance to a site crosses a public road or footpath this has to be kept clear of obstructions, mud and spoil. Care should also be taken to ensure, as far as possible, that adjacent property and services are not damaged by the operations. The site should be kept clear of unnecessary obstructions such as spoil heaps, discarded wire ropes and reinforcement rods, timber with protruding nails, etc. Concrete mixers should be kept clean and should have guards on the dangerous parts of the power unit and gearing. Transport equipment on the site, such as dumpers and excavators, deteriorates unless regularly maintained and the condition of tyres should be watched as skidding on wet sites can become a danger. The drivers of all equipment have to be trained and skilled in its operation. A proper maintenance service should be provided, with inspection at regular intervals, to minimize the number of breakdowns, which are expensive and may be dangerous and to give the operators a sense of pride in the good functioning of their equipment.

Good lighting is essential to the satisfactory working and safety of any construction operation. Where artificial lighting powered by an electricity supply is provided, the voltage to earth for trailing cables should not exceed 55 V. All electrical installations should be properly earthed.

Cables should be protected from accidental damage and kept clear of movable equipment. Plugs and sockets should be of the waterproof type.

There are regulations appertaining to the transport of persons by water and the safety requirements where persons work on or adjacent to water. These requirements include the provision of suitable rescue equipment.

### 11.1.3 Environmental hazards

Consideration should be given to any special environmental hazard that may be encountered when works are carried out in and around existing factory or chemical premises which may house or discharge harmful products or where harmful processes are being carried out. Liaison should be carried out with the owners or occupiers of such premises to determine what precautions should be taken, including personal protection if necessary.

Similar considerations may apply to old industrial sites and landfills where buried structures in the ground may be contaminated with dangerous solids, liquids or gases (RECLAN 1979).

Where excavation takes place in ground that has been contaminated by leakage from petroleum storage tanks, the atmosphere in and adjacent to the excavation should be tested in order to detect the presence of a flammable atmosphere.

### 11.1.4 Fume from welding and flame cutting

The fume from welding and flame cutting arises from a number of sources and produces toxic asphyxiating gases that are dangerous, particularly in a work area that is confined or badly ventilated. These sources are as follows:

- a) from the metal being welded or the metal in the filler rod, e.g. iron, aluminium, or from the constituents of various types of steel such as nickel or chromium;
- b) from any metallic coating on the article being welded, e.g. zinc and cadmium from plating, zinc from galvanizing and copper as a thin coating on continuous mild steel filler rod;
- c) from any paint, grease, etc., on the article being welded, carbon monoxide, carbon dioxide, smoke and other irritant breakdown products; also lead fumes from cutting old steel or iron work painted with lead-based paints;
- d) from the flux coating on the filler rod, e.g. inorganic fluoride;

- e) by the action of heat or ultraviolet light on the surrounding air, e.g. nitrogen dioxide and ozone;
- f) from the inert gas used as a shield, e.g. carbon dioxide, helium and argon.

Depending on the type of welding process and the metals being welded, it is likely that where reasonably practical local exhaust ventilation should be used. Where welding is being carried out in a confined space, fume control by local exhaust ventilation or the use of suitable respiratory protective equipment will be essential. More detailed information on welding fume and methods of control, including local exhaust ventilation may be found in "The Welding Engineers" Handbook' published by The Institute of Welding. BS 4275 gives advice on the selection, use and maintenance of respiratory protective equipment.

#### **11.1.5 Excavations associated with foundations**

It is imperative that a competent and thorough investigation be undertaken of the ground conditions before any excavation is started. The person responsible for the work on site should not proceed until sufficient information is available, including the water pressures in the ground to be excavated and also the ground conditions below the level of the excavation. Interconnection of the strata and presence of man-made and natural weaknesses will have to be ascertained. The positions of existing services, particularly gas, water and electricity, should be established and the possible presence of toxic or flammable liquids and gases should be considered (see BS 6031).

Excavations should be carried out on the assumption that soil conditions are not uniform and site conditions are unfavourable. Timbering or other support should be considered necessary in excavations of 1.2 m depth and over where persons are required to enter except where the sides are sloped back to a safe angle as approved by a person competent and experienced in such matters. It should not be assumed that unsupported vertical sides may be safely excavated in rock strata since the orientation of the geological planes of separations, and any soft materials contained in them, has to be taken into consideration. Excavations should be inspected by a competent person at least once a day when persons are required to work in them.

Because of the greater depths, accidents during the construction of deep foundations are more likely to cause serious injuries and fatalities, e.g. from the collapse of excavations or by persons falling into them. Therefore particular attention should be paid to safety precautions, both in the design and in the construction and maintenance of the works.

## **11.2 Cofferdams and caissons**

### **11.2.1 General**

In addition to the regulations and recommendations mentioned in 11.1, the following regulations and recommendations for the safety of personnel are particularly applicable to construction work in cofferdams and caissons.

Statutory Instrument 961 No. 1580  
Construction (General Provisions) Regulations 1961  
Part V. Cofferdams and caissons, and  
Part VIII. Work on or adjacent to water

The need for protective helmets has been mentioned in 11.1 and their use is particularly important for those in piling gangs and persons engaged in working in excavations or in areas with confined headroom.

Ladders should be of substantial construction and secured to prevent slipping. They should not rise more than 9 m without intermediate platforms.

### **11.2.2 Land cofferdams**

If the sheeting to the cofferdams extends less than 1 m above ground level, guard-rails should be provided to form a barrier at the edge; toe-boards should also be provided. When a land cofferdam is located near to a watercourse, historical flood levels should be checked to ensure that the cofferdam is adequate to withstand exceptional flooding.

### **11.2.3 Cofferdams in water**

Lifebelts with lines attached should be provided and be readily available at positions where they are likely to be needed. Adequate guard-rails should be provided to jetties or floating stages. A safety boat, which will have to be continuously manned, should be available at or close to the work. All men working over water should be required to wear a buoyant life-jacket.

### **11.2.4 Caissons in compressed air**

It is essential that conditions of work in compressed-air chambers are very carefully controlled to avoid health hazards (see 11.3).

The regulations and precautions applicable to work in compressed air are given in 11.3.

### **11.2.5 Electricity**

Supply of electric power for lighting and hand tools should be at a voltage not exceeding 65 V to earth.

Where higher voltages are required for supply to machinery, the supply cable should either be armoured or enclosed in continuous metal conduit, see CP 1017 and the IEE Regulations for electrical installations (the Wiring Regulations) 1981.

### 11.2.6 Off-shore work

An adequate signalling or telephone or radio system should be provided for off-shore working for communication at all times.

### 11.2.7 Fire

Particular attention should be paid to fire hazard on timber jetties or working platforms. Drip trays should be provided for oil drums and under machinery; fire extinguishers have to be provided and kept in working order. Special attention should be given to fire risk in compressed air working chambers and air-locks (see 11.3).

### 11.2.8 Air testing

In deep and confined excavations a continuous routine should be established for testing for noxious gases and deficiency of oxygen.

## 11.3 Excluding water from excavations

### 11.3.1 Use of compressed air

**11.3.1.1 General.** There are a number of regulations applicable to work in compressed air which need to be adhered to rigorously. It is essential that those responsible have full knowledge of current regulations and procedures relevant to such work. Persons supervising work in compressed air should be experienced and competent.

The use of compressed air has its limitations. In particular, as the air pressure is raised to balance increasing water head, the health risk to men working in the compressed-air chamber increases. Special precautions always need to be exercised by rigid control of decompression times and medical supervision of the workers to prevent damage to health (Catton 1968).

Reference should be made to published works on the health hazards of work in compressed air (Haxton and Whyte 1965; Kell and Ridley 1967; McCallum 1967; Haswell 1969) which draw attention to the incidence of "bends" and other more severe forms of decompression sickness such as paralysis or loss of consciousness, as well as to the delayed effect of aseptic bone necrosis. Particular attention is drawn to CIRIA Report 44. These severe health risks become significant in pressures greater than  $100 \text{ kN/m}^2$  ( $1 \text{ kgf/cm}^2$ ) above atmospheric pressure.

### 11.3.1.2 Regulations applicable to work in

*compressed air.* The regulations applicable to work in compressed air under the Factories Act 1961<sup>6)</sup> are listed in appendix D. In addition there is the associated prescribed leaflet Form 754<sup>6)</sup>. The advisory leaflet gives detailed guidance to workers in respect of the personal precautions they should take while engaged in compressed air working.

Whilst no restriction is put on the length of the working period by the regulations, experience shows that there is a large measure of acclimatization to compressed air working. Men with no experience and, in addition, those experienced men who have not recently been in compressed air, should for the first few shifts be allowed to work for no more than 4 h under pressure. New men, following on their medical examination, should first undergo a test compression and decompression accompanied by a competent person (see Regulation 13 of Statutory Instrument 1958 No. 61 (see appendix D)) in order to inspire confidence. Any work in compressed air by new men will need to be supervised by a competent person experienced in such work.

The Medical Research Council's Decompression Sickness Panel introduced decompression tables in 1966 known as the MRC or "Blackpool Tables", the use of which is recommended in place of the decompression tables in the 1958 Regulations. Permission to use these tables is necessary by application to HM Chief Inspector of Factories.

It is recommended that whenever work involving compressed air at pressures greater than  $100 \text{ kN/m}^2$  ( $1 \text{ kgf/cm}^2$ ) above atmospheric pressure is undertaken the Medical Research Council's Decompression Sickness Panel should be consulted. The Regulations and decompression tables applicable to the work should be specified in the relevant contract documents.

### 11.3.1.3 Design and working of compressed air

*installations.* The design and operation of compressed air installations should be undertaken under the supervision of an engineer qualified and experienced in the use of compressed air for civil engineering work. There are a number of essential features which should be incorporated.

<sup>6)</sup> Published by HMSO



a) There will need to be adequate standby arrangements available for immediate use to ensure maintenance of the air pressure and essential services in the event of failure of the normal power supply or plant. An adequate supply of air at the required pressures will need to be available at all times when the plant is working, and a reserve of air supply capacity from an alternative power source should be available in case there is an unexpected loss of air at the face or elsewhere. The capacity of the standby plant available for emergency use will depend amongst other things on the nature of the ground to be encountered and the reliability of the sources of power employed.

b) In those conditions where the ground is self-supporting, and where compressed air is being used to exclude groundwater, standby arrangements to provide against breakdown of the power supply will generally not be necessary where it is possible, with assurance, to have ample time for the men to withdraw to positions of safety before the workings can be flooded.

Dependent upon the quantity of incoming water and the storage capacity of the sumps and shafts, consideration should be given to the use of standby pumps to secure the stability of the foundation works where, in the opinion of the engineer, there would be a hazard due to the failure of the main pumps for water clearance.

c) A good standard of lighting should be observed. Emergency low-illumination battery-operated lamps should be installed at each working face and operated at all times when work is being carried out so that injury is avoided during sudden electrical failure.

d) Smoking in compressed air should be forbidden and all workers in compressed air should be warned of the added danger of fire in pressurized workings. Flammable materials in the working chambers will be rendered much more so, owing to the concentration of oxygen. Air flow through the workings will cause ignited materials to burn fiercely, and if fire occurs anywhere in the chamber, smoke will be carried up to the working face, endangering the men.

Beds of peat have been known to catch fire below ground and burn in the stream of air escaping through them.

Timber in confined spaces such as airlocks should be treated so as to be non-flammable. A water pipe with fire hose couplings should be provided through bulkheads for fire fighting purposes and should meet the requirements of the local Fire Brigade where appropriate.

e) It should be pointed out that the minimum fresh air requirement under Regulation 7 of Statutory Instrument 1958 No. 61 (see appendix D) is  $0.3 \text{ m}^3$  of fresh air per minute per person at the pressure in the chamber, and not at atmospheric pressure.

In cases where there is only a small loss of air at the working face, a fresh air supply should be piped to the face so as to deliver the stipulated volume to the men working there. In such a case the circulation will be maintained by exhausting the surplus volume through the bulkhead or air-lock.

Care should be taken to check that the soils through which excavation is to proceed are not contaminated with deleterious matter which, when coming into contact with the air, gives off a gas which could be dangerous to men working in the contaminated air.

Special watch should be kept on pits or workings in the vicinity of the compressed air work for the presence of de-oxygenated air. Accidents have been caused by compressed air passing through several hundred feet of porous ground and entering excavations in which men were working; in these instances the air lost a large proportion of its oxygen content in oxidizing organic material in its passage and so was unable to support life when issuing into the adjacent working.

f) There needs to be ample provision of access through bulkheads and sufficient ladders for men to leave working faces to go to the surface without hindrance. Wherever practicable, escape routes through airtight bulkheads in tunnels should be provided in the crown. There should be a satisfactory alarm system operated from the face which will advise supervisory staff behind and on the surface that an emergency has arisen. It is very desirable that some form of telecommunication should be available to enable contact to be made between underground workings and the surface.

g) Good working conditions for the men have to be provided and maintained. In addition to the provision of a medical lock in accordance with Statutory Instrument 1958 No 61, when working at pressures in excess of  $100 \text{ kN/m}^2$  ( $1 \text{ kgf/cm}^2$ ) it is essential that medical facilities are on site at all times. Washrooms and clothes-drying facilities should be provided whenever work in compressed air is being carried out.

h) Man-lock doors should be designed to sustain small pressure (at least  $70 \text{ kN/m}^2$  ( $0.7 \text{ kgf/cm}^2$ )) above the maximum pressure used in the chamber.



Decompression locks should be heated under thermostatic control during decompressions to counteract chilling and means of ventilation should be provided.

### 11.3.2 Precautions when excluding water from excavations by freezing the surrounding ground

Men working in excavations in frozen ground should be provided with adequate warm clothing, gloves and leather boots. After use, boots need to be stored overnight in a warm place so that they are sufficiently pliable for use the next day.

Provision should be made for heating hand-operated tools.

### 11.3.3 Chemical grouts

Some chemicals used as additives in grouts are highly toxic although in practice they are used in such small quantities that there is no risk from the grout itself. However, the additives themselves are highly toxic and the manufacturer's safe-handling instructions should be followed. In particular, water soluble acrylamide requires very stringent safety precautions to avoid skin contact, breathing dust, mist or vapour. It requires clean work clothes every day which should consist of long-sleeved overalls, head covering, rubber or plastics gloves and rubber footwear. A respirator and goggles are also required when changing the grout or cleaning up chemical spills. The manufacturer's instructions should be observed for the full safety requirements.

## 11.4 Pile foundations

### 11.4.1 General

**11.4.1.1 Introduction.** Before commencing piling operations reference should be made to the statutory safety requirements discussed in 11.1 and listed in appendix D.

Provision has to be made for an exploratory investigation before work is commenced, to locate underground services which may be damaged during the operations and cause injuries to workers (see "Recommendations on the avoidance of danger from underground electricity cables"<sup>7)</sup> published by the National Joint Utilities Group). The authorities concerned should be consulted about possible dangerous conditions and recommended safe working distances from their equipment.

In keeping with the general advice given in 11.1, safety helmets and safety footwear should always be worn. Piling sites are often very wet and the use of rubber boots is essential; these should be reinforced by steel not only in the toes but also in the soles, to prevent injury by sharp objects concealed under the spoil and mud. Safety harnesses should be worn by men required to climb up pile frames or other equipment and these should be used with fixed anchorages or mobile automatic locking slides on vertical runners or ropes.

Noise levels during certain operations can be injurious to hearing and so require the wearing of ear protectors.

**11.4.1.2 General public.** The contractor will need at all times to ensure that members of the public are protected from any accident or injury arising from the piling operations.

**11.4.1.3 Site supervision.** A competent person, properly qualified and experienced, should be appointed to supervise the piling operations. This person should be capable of recognizing and assessing any potential dangers as they arise, e.g. unexpected ground conditions that may require a change in construction technique, or unusual smells which may indicate the presence of noxious or dangerous gases.

**11.4.1.4 Equipment.** Most of the hazards on a piling site arise in operating the equipment and plant. A piling frame is a "lifting appliance" within the meaning of the Factories Act 1961 and the Construction (Lifting Operations) Regulations 1961, as are also mobile cranes, excavators and winches. Refer to these Regulations for inspection, testing and examination requirements and the time periods in which these should be carried out.

**11.4.1.5 Special site conditions.** Particular site conditions should have corresponding safety provisions. High tension overhead power lines should not be approached nearer than 5 m because of the danger of arcing. If the power lines are owned by the Central Electricity Generating Board and not by a local authority, permission has to be obtained from the CEGB to work within 12 m of the pylons or cables.

If a diver is employed for the purpose of assisting pitching or examining the piles, his work should be carried out in accordance with 11.6.

Piling inside excavations or on hillsides should be carried out with particular care. Excavations need to be well shored or braced and banks cut back in case the vibration of pile driving causes falls or instability.

<sup>7)</sup> Available from the Water Authorities Association, 1 Queen Anne's Gate, London SW1H 9BP.

### 11.4.2 Driven piles

**11.4.2.1 Use of equipment.** The following points should be carefully observed.

- a) Operators should never stand under or near a pile hammer. (While temporary compression values are being taken and on certain types of hammer the operator, inspector or engineer has to be close to the hammer; it follows that all care should be exercised during these times.) If any adjustment has to be made at or below a hammer, the hammer should be rested on a pile or secured by the hammer retaining pin through the pile frame leader. The hammer should be properly secured and not held on the pawl of the winch lifting-rope drum.
- b) All equipment, particularly pile frames and leaders, should be examined at predetermined intervals for any loose bolts.
- c) Ladders should be provided on frames and leaders to give access to the frame head and hammer; where the ladder rises by more than 9 m intermediate landings should be provided.
- d) Where pile frames are mounted on rails, the rails should be of adequate size, laid on and secured to properly ballasted sleepers, and joined together with fishplates and bolts. The frame should be secured before any piling commences and proper stops maintained at the ends of the track.
- e) Frame should be secured by guy ropes attached to safe anchorages and checked before leaving the frames unattended, especially overnight and at weekends. Rope ends should not be left trailing on the ground and never coiled loosely on winch warping drums.
- f) Diesel pile hammers should be cleaned regularly to avoid an accumulation of diesel oil which may become a fire hazard. They should be fitted with a trip wire or rope so that the hammer can be stopped from ground level and workmen do not have to climb ladders to operate the fuel cut-off.
- g) The exhaust of steam hammers should be arranged so that workmen are not endangered by the discharge of steam or scalding water.
- h) All hoses and hose connections should be in good condition and properly secured to the hammer inlet. The end of the hose should be tied to the hammer to prevent a flying end if a connection should break loose.

i) Helmets, diving caps, anvil blocks and other parts receiving impact should be inspected regularly for damage or fracture. Worn parts should be replaced before wear becomes excessive and particular care should be taken to avoid wear that will develop a stress raiser on a moving part. Precautions should be taken to ensure that the helmet cannot fall.

j) Before making any adjustments on the cylinders of steam winches, the main stop valve should be closed and the drain cocks opened.

k) Steam boilers should comply with all regulations concerning pressure vessels and steam raising equipment. The attendant should have a thorough knowledge of running and maintaining steam boilers.

l) Frayed wire ropes and worn shackles should be replaced and loose bolts and connections should be made safe. (See Regulation 34 of Construction (Lifting Operations) Regulations 1961.)

m) All moving parts shall be fenced unless safe by position or construction (see Section 14 of The Factories Act 1961).

n) If hollow piles are cut off at near ground level and are not concreted immediately, or cast-in-situ piles finished below ground level, proper secure covers or fencing should be provided to prevent the risk of injury to persons from falling into the holes.

o) Raking frames with power-operated raking gear need to have a stop fitted to prevent the screw running out and allowing the frame to collapse.

p) Pile driving equipment designed for the installation of proprietary types of cast-in-situ piles should not be used for other work without reference to the manufacturers, or unless all the consequences of such use have been fully investigated by a competent engineer, especially from the point of view of overloading and stability.

Where the machine has hydraulically operated leaders, these should be racked back at the end of a period of use to ensure the rams are closed to avoid instability or the collapse of a frame due to hydraulic leakage. Hammers should be left at the bottom of their travel when not in use.

**11.4.2.2 Precast concrete piles.** The greatest need for safety precautions with precast concrete piles arises during handling and pitching. Site conditions, e.g. marine works, will also indicate any special precautions that may be necessary.

It is essential that the requirements of the Construction Regulations (see appendix D) relating to precautions on raising, lowering and securing loads when using heavy lifting equipment be strictly observed. The correct type of equipment, and hammers of at least an adequate size, should be selected to minimize wear and tear and consequently reduce breakdowns and accidents.

**11.4.2.3 Steel piles and steel sheet piling.** The safety precautions necessary in handling precast concrete piles are also applicable to steel piles and interlocking steel sheet piling. A U-shackle or other suitable lifting shackle should always be used. A short steel bar through the lifting hole with a wire rope stop is not permitted. With very long piles it is sometimes necessary to sling the pile some distance from the top to gain sufficient headroom for pitching. A wire rope or chain is then slung round the pile. This is dangerous because the pile may slip through the string. Where this is unavoidable, a safety wire rope with a hook at the bottom should be attached to the sling, so that the hook may be hooked under the bottom end of the pile. All operators on the ground should stand well clear of the pile during pitching.

On no account should the pile topman who guides the pile into the interlock of a pile already pitched stand at the top of a ladder leaning against the pitched pile. He should be provided with a fenced platform which can often be built onto the temporary timber trestle used for pitching the piles in panels or provided with a safety harness attached to permanent static line. If a man-riding skip used for access is suspended from a crane, then the hoisting mechanism of the crane has to be provided with automatic braking facilities.

Only skilled and experienced men with adequate and safe equipment should be required to carry out the potentially hazardous operation of interlocking steel sheet piles.

The extraction of steel sheet piles should be carried out with an extractor of adequate size, so that the performance is not mainly dependent on the pull of the crane and there is no danger of overloading or overturning the crane.

#### 11.4.3 Bored piles

**11.4.3.1 Piles bored with tripod.** Constant supervision is necessary to ensure that the wire to the winch, and also the brakes and clutches, are in good condition.

NOTE Attention is drawn to the need to examine wires, sheerlegs and lifting gear to the requirements and at the time periods prescribed in the Construction Regulations (see appendix D).

Hooks should be of the safety type such as the “Liverpool” or have a safety catch or clip to prevent accidental displacement of the load.

Knots in ropes and chains in direct tension are expressly forbidden by the Regulations. The brakes have to be of a type that can be locked in the ON position to prevent accidental displacement.

Whenever possible the winchman should be able to see the top through which he is boring but if this is not possible then a banksman will need to be employed.

Naked flames and smoking should not be permitted in boreholes or anywhere near potentially dangerous areas.

Sheer legs always have to rest on a secure flat surface, timbers being used if necessary. If a tube jams in the ground, it is important not to overload the winch, legs, etc., and heavy jacks, suitably supported, should be used to lift the tube. When acquiring gear for boring piles, it should be borne in mind that the greatest load is applied when tubes are being withdrawn from the ground.

**11.4.3.2 Large diameter bored piles.** In addition to the general precautions given in 11.4.3.1, safety precautions should be observed in accordance with BS 5573.

## 11.5 Tide work

### 11.5.1 Statutory requirements

The statutory requirements for the safety and welfare of persons engaged in tide works are governed by the Factories Act 1961 as amended by the Health and Safety at Work etc. Act 1974; these are listed in appendix D.

### 11.5.2 Work over or adjacent to water

NOTE Attention is drawn particularly to Regulations 23 and 24 of the Construction (General Provisions) Regulations 1961.

Taking into account the circumstances, in some cases, especially in flowing water, buoyed lines will be desirable at a suitable distance from the work to give anyone who falls in an adequate chance of securing a handhold while awaiting rescue. Particular attention is drawn to the need for adequate floodlighting at night at least within the limits of the buoyed lines. In fast flowing waters or tidal waters a safety boat, boatmen and lifebelts at conspicuous positions should be provided. All men should wear a life-jacket when working over water.

## 11.6 Diving

### 11.6.1 General

This clause governs the use of divers on work of engineering construction and on maintenance operations. It is not intended to be a detailed description of diving procedure, but to give guidance on the employment of divers.

Planning the safe and successful execution of any diving operation depends upon the establishment of a clearly defined chain of responsibility involving the client, the diving contractor and the diving supervisor.

NOTE Attention is drawn to the relevant statutory regulations and codes of practice.

### 11.6.2 Selection of diving services

A client requiring diving services for any underwater operation should carry out the following.

- a) Select a diving contractor or other employer of divers having a diving team adequately equipped and competent to carry out the work.
- b) Clearly define the scope and nature of the underwater work involved.
- c) Supply to the diving contractor or other employer of divers precise and adequate information regarding conditions both on the surface and underwater at the work site or, alternatively, provide the opportunity and facilities for the diving contractor to obtain this knowledge prior to the commencement of the diving operations.
- d) Provide meteorological and hydrological forecasts prior to and throughout the diving operations.
- e) Ensure that any related or adjacent engineering work or marine operations are properly co-ordinated with the diving operations. Interaction of operations may require that a permit to work system be initiated. When divers are operating underwater, warning signals should be displayed by means of flags, lights or any other equally effective means.
- f) Arrange for the health, safety and welfare of all diving and support personnel, except when diving is in progress, or when on board vessels provided by the diving contractor.

## 11.7 Safety precautions connected with demolition, shoring and underpinning

NOTE Before commencing operations connected with demolition, shoring and underpinning, reference should be made to BS 6187. Attention is drawn to the statutory provisions listed in appendix D.

### 11.7.1 Demolition

Where demolition is to take place using explosives or by mechanical means such as the use of a balling machine, special care should be taken to protect adjacent areas, adjacent properties, public services and members of the public. Such operations should be carried out under the supervision of a competent person experienced in the methods to be used and the precautions to be observed.

### 11.7.2 Use of explosives

The use of explosives in works connected with foundations should be carried out in accordance with BS 5607.

This method has to be supervised by fully trained and competent persons who are experienced in the work and know the dangers likely to arise and the precautions to be observed.

Before any explosives are used in demolition operations the police should be informed and their assistance sought where it is necessary to keep the area free from members of the public.

There is always a possibility that mains and other services may be damaged by shock or vibration; consequently the appropriate authorities should be notified.

Proper and secure storage is necessary at all times for explosives, detonators and fuses. Detonators should be kept either in suitable locked cases or boxes separate from any other explosives. They should be stored only in a store licensed or registered under the Explosives Acts. Detonators should only be removed when about to be used for preparing a charge.

Electrical methods of firing the detonator are commonly used and only apparatus properly designed for this purpose should be used.

Fires, open lights and smoking should be prohibited within 6 m of any explosives. There is a risk with all explosives, but blackpowder and detonators are especially susceptible to premature ignition by spark or flame. Any person proposing to detonate an explosive charge should determine the danger zone likely to be created before firing the explosive. There should be an adequate audible system in use to give warning before firing and when danger has passed, i.e. all is clear. Notices explaining warning systems, flags and look-outs should be posted at all approaches.

Where necessary, precautions should be taken to protect other properties from shock and vibration. Flying debris should be controlled by means of blast mats, baffles or other blast-protection measures.



In the event of a misfire the area should be cleared and only the competent person should return to the site of operations until the misfire has been cleared. Most explosives manufacturers give advice on the extent of time which should elapse before the misfire is approached and no attempt should be made to reduce this period.

Electrical storms, transmitting apparatus and certain electrical installations can affect detonators, without actual contact, effecting detonation by extraneous electrical energy entering the shot-firing circuit.

In the case of electric storms, the area should be cleared of persons when the storm approaches within 8 km.

An essential rule in the safe use of explosives is to keep an accurate record to show the disposition of explosives that have been issued from the store.

### **11.7.3 Underpinning**

Particular attention is drawn to Regulation 12 of the Construction (General Provisions) Regulations 1961 and the need to support the sides of excavations associated with underpinning.

Safety will be greater on sites if it is constantly borne in mind that these operations are of a specialist nature and require the employment of competent persons. The detailed sequence of operations should be planned with safety in mind.



## Appendix A Derivation of charts for the determination of allowable bearing pressures on weak and broken rocks (see 2.2.2.3.1.12)

### A.1 Geotechnical logging and assessment

The data from core logging, in situ field tests in boreholes and laboratory tests on representative specimens of the core from the boreholes are gathered together in a series of geotechnical logs. The full suite of logs displays the following basic information against depth or level:

- a) Rock type and description
  - b) Weathering state
  - c) Fracture spacing, fracture intensity, RQD
  - d) Assessed strength
  - e) Special remarks, e.g. sheared zones
  - f) Permeability or Lugeon value
  - g) Point load strength
  - h) In situ modulus
  - i) Seismic velocities, crosshole and/or uphole
  - j) Water content and bulk density
  - k) Atterberg limits
  - l) Carbonate content
- } if applicable
- m) Compressive strength
  - n) Young's modulus

From consideration of items a), b), c) and f), the appropriate mass factor is assessed for each lithological or structural unit using experience and judgment. This is combined with the corresponding Young's modulus to produce a rock mass modulus/depth profile which is adjusted in the light of any in situ modulus or seismic results. If elastic parameters have not been determined the intact modulus is estimated on the basis of the rock type and its assessed or measured compressive strength (Hobbs 1974).

The settlement of the foundation is then calculated in the usual way using the appropriate elastic theory. This procedure is essential for foundations of important structures. Certain rocks undergo consolidation and/or creep under load; consideration should be given to an allowance made for the effects of such processes in estimating the settlement and allowable bearing pressure of foundations on such rocks.

### A.2 Calculation of allowable bearing pressure

The settlement of a rigid foundation or average settlement of a flexible foundation at the surface of a homogeneous elastic half-space is given by the equation

$$s = \frac{q_a B(1 - \nu^2) l}{E_m} \quad (1)$$

where

- $q_a$  is the average pressure on the rock;
- $B$  is the width or diameter of the footing;
- $\nu$  is the Poisson's ratio of the rock mass; since  $\nu$  is generally about 0.2 the term  $(1 - \nu^2)$  may be taken as one;
- $E_m$  is the modulus of the rock mass;
- $l$  is the influence value dependent upon the shape of the footing and the rigidity of the footing relative to the rock mass; typical values are (Lysmer and Duncan 1969):

Shape	Circular	Square	Rectangular $L/B$		
			2	5	10
Rigid	0.79	0.82	1.1	1.6	2.0
Flexible (mean value)	0.85	0.95	1.3	1.8	2.2

The equation may be rewritten as follows:

$$q_a = \frac{E_m}{l} \left( \frac{S}{B} \right) \quad (2)$$

where

- $S/B$  is the settlement ratio, i.e. the permitted settlement in relation to the width of the footing.

If the mass factor of rock mass is  $j$  then:

$$E_m = jE \quad (3)$$

where

- $E$  is the Young's modulus of the intact rock, which is related to the uniaxial compressive strength  $q_c$  of the intact rock as follows:

$$E = M_r q_c \quad (4)$$

where

- $M_r$  is the modulus ratio.

From this equation  $E_m$  is calculated as follows:

$$E_m = jM_r q_c \quad (5)$$

which inserted in equation 2 for the average bearing pressure produces the following general equation for the allowable bearing pressure

$$q) \text{ allowable} = q_c \left( \frac{S}{B} \right) \frac{jM}{l} \quad (6)$$

In the preparation of the curves (see Figure 1) the influence value  $l$  has been taken as one, although strictly for a rigid square footing the value is 0.82; but in view of the fact that  $q_c$  and particularly  $j$  cannot be known with comparable accuracy the approximation is justifiable.

The  $j$  value has been taken as numerically equal to the average discontinuity spacing, expressed in metres, on the assumption that the discontinuities are reasonably tight.

The four rock groups have been given the following modulus ratios.

Group 1: 600

Group 2: 300

Group 3: 150

Group 4: 75

These ratios are considered to be conservative.

NOTE The calculation of allowable bearing pressure given in Figure 1 assumes that  $E_m$  is constant with depth and that the rock layer is infinitely thick. This can lead to large errors in the calculated settlement if  $E_m$  varies or the rock layer is thin. A method of calculation which takes account of the variation of  $E_m$  and finite layer thickness is given in Meigh (1976).

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## Appendix C Further reading

### C.1 Books

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### C.2 Papers

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## Appendix D Recommendations and statutory requirements affecting the safety, welfare and health of persons at work

Particular attention is drawn to the following statutory requirements affecting the safety, welfare and health of persons at work.

- Health and Safety at Work etc. Act 1974
  - The Factories Act 1961 as amended by the Health and Safety at Work etc. Act 1974
  - Statutory Instrument 1961 No. 1580  
The Construction (General Provisions) Regulations 1961
  - Statutory Instrument 1961 No. 1581  
The Construction (Lifting Operations) Regulations 1961
  - Statutory Instrument 1966 No. 94  
The Construction (Working Places) Regulations 1966
  - Statutory Instrument 1966 No. 95  
The Construction (Health and Welfare) Regulations 1966
  - Statutory Instrument 1974 No. 903  
Woodworking Machines Regulations 1974
  - Statutory Instrument 1958 No. 61 as amended by SI 1960 No. 1307  
The Work in Compressed Air (Amendment) Regulations 1960
  - Statutory Instrument 1960 No. 688  
The Diving Operations Special Regulations 1960
  - Statutory instrument 1969 No. 690  
The Asbestos Regulations 1969
  - SR & O 1908 No. 1312  
The Electricity Regulations 1908 as amended by SR & O 1944 No. 739
  - The Electricity (Factories Act) Special Regulations 1944
  - The Protection of Eyes Regulations 1974
  - The Abrasive Wheels Regulation 1970
- Reference should also be made to the IEE Regulations for electrical installations (the Wiring Regulations) 1981; to “Guide to the Construction Regulations”, published by the Federation of Civil Engineering Contractors and the National Federation of Building Trade Employers; to “Construction Safety”, published by the National Federation of Building Trade Employers; to the “Building and Construction Regulations Handbook”, published by the Royal Society for the Prevention of Accidents and to a Health and Safety at Work series of booklets issued by the Department of Employment entitled “Safety in Construction Work”.

The following Statutory Regulations also apply to diving operations:

Statutory Instrument 1974 No. 1227  
The Offshore Installations (Diving Operations) Regulations 1974

Merchant Shipping Act 1974, Chapter 43,  
Part IV

Statutory Instrument 1975 No. 116  
The Merchant Shipping (Diving Operations) Regulations 1975

Mineral Workings (Offshore Installations) Regulations Act 1971, Chapter 61

Statutory Instrument 1973 No. 1842  
Offshore Installations (Inspectors and Casualties) Regulations 1973

Submarine Pipelines (Diving Operations) Regulations 1976



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