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UDC 69.021.15 : 624.15

Code of Practice for

Foundations

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This Code of Practice represents a standard of good practice; compliance with it does not confer immunity from relevant legal requirements, including byelaws.

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Foreword

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 has now passed to the Council for Codes of Practice of the British Standards Institution for whom the present revised edition has been prepared.

Recent research following structural collapse in 1973 and 1974 has considerably altered the understanding of the behaviour and sensitivity of high-alumina cement concrete. As a result of both the research and the events themselves 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 must 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.

NOTE. The superscript numbers used throughout the text of this Code relate to the references listed in Appendix A.

Metrication

SI units are used throughout this Code with imperial units in parentheses.

However, having regard to the many countries in Europe and the rest of the world where the Code may be read, and where technical metric units are in use, the committee considered it essential to include these units alongside SI units wherever necessary to facilitate widest understanding of the Code.

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1. General

1.1 Scope

This Code of Practice deals with the design and construction of foundations for the normal range of buildings and engineering structures and is not intended to cover foundations for special structures.

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

Detailed information for the design and construction of foundations for reciprocating machinery is given in CP 2012 : Part 1.

The Code is divided into the following nine sections in addition to this introductory general section:

2. Design of foundations
3. Shallow foundations
4. Deep and subaqueous foundations
5. Cofferdams and caissons
6. Geotechnical processes for dewatering and treatment of substrata
7. Pile foundations
8. Tide work, underwater concreting, and diving
9. Excavation, demolition, shoring and underpinning
10. Durability of timber, metal and concrete structures.

Section 2 of this Code deals with the general principles of design; Sections 3, 4, 5 and 7 are concerned with more detailed considerations of the design and installation of the main types of foundations. Sections 6, 8 and 9 relate to site operations and construction processes involved in foundation engineering and Section 10 describes the factors affecting the durability of the various materials used in foundation structures.

This Code does not cover foundations made with high-alumina cement.

NOTE. The titles of the British Standard publications referred to in this Code of Practice are listed on the inside back cover.

1.2 Definitions

For the purposes of this Code of Practice the following definitions apply:

1.2.1 General terms

Foundation. That part of the substructure in direct contact with and transmitting loads to the ground.

Substructure. That part of any structure (including building, road, runway or earth work) which is below natural or artificial ground level, in a bridge including piers and abutments (and wing walls) which support the superstructure.

Gross loading intensity. The intensity of vertical loading at the base of a foundation due to all loads above that level (for non-vertical loads see 2.3.2.3.7).

Net loading intensity. The additional intensity of vertical loading at the base of a foundation due to 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.

Ultimate bearing capacity. For a particular foundation, that value of the net loading intensity at which the ground fails in shear. It is determined by calculation from either field loading tests on small areas or strength tests on undistributed samples. (See also 1.2.4.)

Presumed bearing value. The net loading intensity considered appropriate to the particular type of ground for preliminary design purposes. Values for various types of ground are usually given in the form of a table. The particular value is based either on local experience or on calculation from strength tests or field loading tests using a factor of safety against shear failure (see 2.2.2.2.2).

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Allowable 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 given structure to take up this settlement. It is, therefore, a combined function of both the site conditions (including all construction in the vicinity) and the characteristics of the particular structure it is proposed to erect (see 2.2.2).

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.

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

Tie. A member in tension.

Waling. A horizontal beam supporting a cofferdam wall or poling boards or runners against external pressure.

1.2.2 Terms relating to shallow foundations

Pad foundation. An isolated foundation to spread a concentrated load.

Strip foundation. A foundation providing a continuous longitudinal bearing.

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

Ground beam. A beam in a substructure transmitting a load to a pile, pad or other foundation.

1.2.3 Terms relating to cofferdams and caissons

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.

Blow or boil. The phenomenon of the soil being forced into a cofferdam from below by the upward flow of water or air.

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

Caisson. A structure which 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.

Caisson box. A caisson which is closed at the bottom but open to the atmosphere at the top.

Caisson, open. A caisson open both at the top and at the bottom.

Caisson, compressed-air or pneumatic. 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.

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

Cofferdam, cellular. A cofferdam enclosed by a wall consisting of a series of filled cells of circular or other shape in plan.

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

Cylinder. An alternative name for an open caisson or monolith of cylindrical form.

Depth of cut-off. The depth reached by the sheet piling or cofferdam walls below excavation level.

Kentledge. See 1.2.1.

Lacing. A system of ties connecting one setting of walings or struts to another.

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

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

Puncheons. Vertical struts transmitting the weight of the bracing to the excavated ground surface inside a cofferdam.

Skin friction. The frictional resistance of the surrounding soil on the surface of cofferdam or  Documents

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Strut. See 1.2.1.

Waling. See 1.2.1.

1.2.4 Terms relating to pile foundations

Allowable load. The load which 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.

Anchor pile. See 'Tension pile'.

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

Batter pile. See 'Raking pile'.

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. If it 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.

Bearing capacity. See 'Ultimate bearing capacity of a pile'.

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, or precast concrete sections which are grouted in place.

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

Cased pile. A pile with a permanent shell or casing subsequently filled with concrete.

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

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 head during driving.

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

Driven cast-in-place pile. A pile formed in the ground by driving a permanent or temporary casing, and filling it with plain or reinforced concrete.

Drop or stroke. The distance which the hammer or ram is allowed to fall.

Drop hammer. A weight, usually of metal, raised by a winch and allowed to fall under gravity.

Enlarged base. An enlargement of the base area of a pile, formed:

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

Factor of safety. The ratio of the ultimate bearing capacity to the working load.

Follower (or long dolly). A removable extension which 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.

Hammer, single-acting. A hammer raised by steam, compressed air, or internal combustion and allowed to fall under gravity.

Hammer, double-acting. 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.

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

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

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.

Kentledge. See 1.2.1.

Large diameter bored pile. A pile, generally 600 mm (24 in) or more in diameter, formed by boring with power-driven mechanical drilling or grabbing equipment.

Leaders. Generally of 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.

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

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.

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

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

Precast pile. A reinforced or prestressed concrete pile cast before driving.

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.

Proof load. A load applied to a selected working pile to confirm that it is suitable for the load at the settlement specified. A proof load should not normally exceed 150% of the working load on the pile.

Raking pile. A pile installed at an inclination to the vertical.

Ram. The rising and falling part of the hammer which delivers the blow.

Resistance. See 'Ultimate bearing capacity of a pile'.

Screw pile. A pile screwed into the ground consisting essentially of a column provided with a helix or helices at its base.

Set. The net distance by which the pile penetrates into the ground at each blow of the hammer.

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. It is designed to resist mainly lateral forces and to reduce seepage; it may be vertical or at an inclination.

Tension pile. A pile which is designed to resist an upward force.

Test pile. A pile to which a load is applied to determine the load/settlement characteristics of the pile and the surrounding ground.

Ultimate bearing capacity of a pile. The load at which the resistance of the soil becomes fully mobilized, but see 7.5.6.5.

Under-reamed pile. See 'Enlarged base'.

Working load. The load which the pile is designed to carry (see also 'Allowable load').

Working pile. One of the piles forming the foundation of the structure.

1.2.5 Terms relating to demolition, shoring and underpinning

Adjoining owner. A person or persons owning lands, a building or portion of a building which abuts upon or is within 3 m (10 ft) of the proposed building operation. Where the excavations for a new building are below the foundations of an adjoining building and, particularly where underpinning has to be done, buildings up to a distance of 6 m (20 ft) should be included.

Building owner. A person or persons intending to demolish, erect, extend or alter a building or other structure.

Cleat. A stop fixed to prevent movement of a strut or waling.

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

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

Headtree. A horizontal member placed immediately on the heads of dead shores.

Leg. A support in an underpinning forming part of the permanent work.

Needle. A member inserted into or through a wall to give it temporary or permanent support. 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 the abutment to the shore.

Poling board. A flat member in contact with the ground and supporting the face or sides of an excavation, and usually 1 m to 1.5 m (3 ft to 4 ft 6 in) long.

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Runner. A vertical member 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.

Dead shore. A vertical strut supporting a horizontal member which is carrying the wall or other load from above.

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.

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, the whole of the series of struts being braced or laced together.

Sill or soleplate. A member placed under the foot of a shore for the purpose of distributing the load.

Soldier. A vertical member supporting horizontal poling boards or walings.

Strut. See 1.2.1.

Tie. See 1.2.1.

Waling. See 1.2.1.

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

2. Design of foundations

2.1 General

Section 2 of the Code deals with the principles of, and makes recommendations applying generally to, the design of foundations of all types.

2.1.1 Classification of soils and rocks. The classification of soils and rocks used in this Code is set out in Table 1 and is discussed in 2.2.2.3. Further definition and description can be obtained from CP 2001.

2.1.2 Principles

2.1.2.1 General. Foundation design is concerned primarily with ensuring that the movements of a foundation are within the limits that can be tolerated by the proposed structure without adversely affecting its functional requirements. Structures vary widely in their capacity to accommodate movements of their foundations, and it is thus desirable that the design of the structure and the foundation be considered as interrelated problems.

Foundation design requires an understanding of the local geology and an appreciation of the various types of ground movement which can occur. A knowledge of the behaviour of soil and rock and the way in which the structure will respond to movements of its foundations is also necessary. Attention is drawn to the fundamental importance of knowing the groundwater conditions.

Foundation movements arise either from the effects of applying loads to, or removing loads from, the ground, or from movements of the ground independent of the load.

The absolute movement of foundations may affect the purpose of the structure, external connections to the structure such as pipes or drains and the relation of the structure to adjacent structures. Relative movements between different parts of the foundation lead to distortion of the structure and, if not limited, may cause damage to finishes, cladding or structural members¹.

2.1.2.2 Ground movements due to application of load. Movement of foundations under the influence of loading may result from overstressing the ground, which leads to plastic deformation in the ground below the foundation. In extreme cases shear failure may occur.

Even when the factor of safety against shear failure is adequate, settlements of foundations take place. These settlements may be what are known as *immediate settlements*, which take place as the load is applied, or *consolidation settlements* due to the dissipation of excess pore pressures as water, or water and air, are expelled from the voids in the soil, with a consequent decrease in volume. This consolidation will be slow in soils of low permeability. In some cases further settlement in the nature of a long term creep may occur.

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2.1.2.2.1 Shear deformation and failure

2.1.2.2.1.1 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 soil 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 soil and the increased overburden pressure at foundation level, but also with shear forces that can be mobilised 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 bearing capacity for foundations are outlined in 2.2.2. In Table 1 presumed bearing values for foundations on various classes of soils and rock are indicated.

It must however be realized that the existence of an adequate factor of safety against shear failure will not necessarily ensure that the foundation settlements will be sufficiently small. In particular, the allowable bearing pressure for a large foundation 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 structural element at the particular locality tested, but the results may not be representative of the bearing characteristics of the site as a whole. Deep-seated failures due to the combined loads of a number of foundations may occur, and this possibility should be checked. It should be noted that the settlements recorded in connection with loading tests will not necessarily 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 2.3.2.3.6 and 2.3.2.3.7.

2.1.2.2.1.2 SOFT LAYERS BELOW FOUNDATION. When soft layers of soil 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 consolidation settlements in the soft layer.

2.1.2.2.1.3 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).

2.1.2.2.1.4 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.

2.1.2.2.2 Settlement. The magnitude of the settlement that will occur when foundation loads are applied to the ground depends on the rigidity and compressibility of the underlying strata. In silts and clays the 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; due 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 secondary consolidation, and will need special consideration. In sands and gravels, 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:

- (1) 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;
- (2) an analysis of the settlement based on measured soil properties and the stresses applied to the ground by the structure (see particularly 2.2.2.3.2 and 2.2.2.3.3).

It must 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 2.3.2.6).

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2.1.2.3 Movements independent of applied load. Movements may result from causes not connected with the loads applied by the foundations². Some of the causes of ground and foundation movement are listed below:

- (1) Seasonal changes or the effects of vegetation leading to shrinking and swelling of clay soils.
- (2) Frost heave.
- (3) The application of artificial heat or cold to the supporting ground.
- (4) Changes in groundwater level resulting from designed or unforeseeable drainage, or modification of the groundwater regime by construction, or by natural causes.
- (5) Loss of ground due to internal erosion or solution by percolating water, and loss of fines by pumping operations.
- (6) Loss of support due to adjacent excavations, dredging, scour or erosion by streams or floods.
- (7) Continuing settlement of natural deposits or fill.
- (8) Landslides or soil creep on slopes.
- (9) Movement of ground resulting from sink or swallow holes or underground workings (including tunnelling).
- (10) Vibrations, including seismic disturbances.
- (11) Deterioration of made ground or fill.
- (12) Deterioration of the substructure itself.
- (13) Alteration of the properties of the ground due to natural or artificial causes.

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

2.1.2.4 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 discussed in 2.3.3.

2.1.2.5 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. Details of construction procedures involved in foundation engineering are given in Sections 5 to 9.

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

2.1.3 Groundwater. The presence of groundwater frequently causes difficulty during construction or in completed works. Section 6 deals with the removal of water from works during construction. Exclusion of water from completed structures is dealt with in 2.3.4 and in CP 102, to which reference should be made.

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, for example, the clearance of vegetation, the modification of existing drainage facilities and basement construction. The latter may interfere with the natural groundwater flow causing a rise in the water table. Substructures should be designed to be stable with the highest groundwater level that is likely to occur. Where it is necessary for the structure to be watertight, it should be made so to a sufficient height to guard against capillary action and to allow for a subsequent rise in the water table. These effects are dealt with in 2.3.2.6, 2.4.3, 3.2.1, 3.2.7, 4.3 and 4.4 and some have already been mentioned in 2.1.2.3.

2.2 Design: soil considerations

2.2.1 Site exploration and soil tests

2.2.1.1 General. Before foundations are designed and a method of construction determined, it is desirable to carry out a site exploration to ascertain the character and variability of the soil 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.

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If possible, exploration should be carried out before purchase of a site, since it may indicate it would be more economical to acquire a better site at a greater initial cost.

Site investigations should be carried out generally in accordance with the principles set out in CP 2001. In the present Code, brief reference is made only to points of particular importance in foundation design.

Comments on exploration in relation to dynamic loading are given in 2.3.2.4.2.

2.2.1.2 Collection of available information. As a preliminary, it is usually advantageous to collect information relating to the site prior to commencing its exploration. Details which 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 may prove valuable. Enquiries relating to 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 liable to landslips or to local subsidence arising from mining operations or swallow holes. The inspection of buildings in the neighbourhood, particularly with regard to settlement and the incidence of cracking, may yield valuable information when structures of a similar nature are proposed.

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 may be plentiful, exploration may be limited to checking that the soil 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 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 its probable bearing capacity to be assessed. Where the ground is unsuitable for trial pits and for deeper exploration, boreholes are required.

Samples should be procured for the purposes of identification from each stratum. Where appropriate, undisturbed 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 6.2.2.1). 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 undisturbed 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 and borings. The disposition and spacing of the trial pits or borings should be such as to reveal any major changes in thickness, depth or properties of the strata. The number of trial pits or borings required will vary with the size and type of structure, the general nature of the site and the completeness or otherwise of local geological 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, 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.

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. 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 when there exist 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 should be explored.

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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.

The seasonal variation in the level of the water table may also be important. Where deep excavation is required, the location of water-bearing strata should be determined with particular care, and the water pressure in each 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 or soil may contain constituents in amounts sufficient to cause damage to Portland cement concrete or buried metals (see Section 10). In such cases it will be desirable to obtain a chemical analysis of samples of the groundwater or soil (or sometimes both) to assess the necessity of special precautions.

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 a proof that a much larger area or a group of piles will carry load pro rata, particularly where a stronger layer of soil overlies a weaker one (see CP 2001 : 1957, Appendix J110 to J113).

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 desirable that the exploration should be adequately supervised and that the interpretation of the results should be closely related to the design as a whole.

2.2.2 Allowable bearing pressures on various types of ground

2.2.2.1 General. This section deals with design recommendations for all types of foundations. Recommendations covering particular types of foundations are given in subsequent clauses of the Code.

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; the 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 obtain during all stages of the construction of the entire work. The construction procedure should be arranged so as 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 analysis³.

2.2.2.2 Allowable bearing pressure and presumed 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 value has been introduced into 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.

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

- Group I. Rocks.
- Group II. Non-cohesive soils.
- Group III. Cohesive soils.
- Group IV. Peat and organic soils.
- Group V. Made ground or fill.

The presumed bearing values for each of these groups 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 where they are weathered, decomposed, heavily shattered or steeply dipping. Where a foundation is carried some distance into unweathered rock, the allowable bearing pressure may often be increased over the presumed bearing values given in Table 1. Each case will need to be considered individually. The effect of joints and fissures 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.

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

2.2.2.3.1.2 FIELD AND LABORATORY MEASUREMENTS. The choice of method of field and laboratory measurement in rocks and the detailed location of tests is dependent on the condition of the rock, including the degree and variability of weathering and the nature and spacing of joints.

The condition of the rock is best assessed by visual inspection in trial pits and examination of drill cores taken in a suitable manner. It is usually necessary to concentrate field and laboratory testing on the weaker zones or layers since these may control the design. These weaker zones or layers are those which may cause poor core recovery. In any event, care should be exercised where core recovery is less than 100%. Choice of core size should also be related to the rock condition, larger diameters being required where the rock condition is poor.

In the field, loading tests in trial pits are usually the best available method of assessing the strength and stress/strain characteristics. A vertical loading test on a plate is the commonest form of test, but side-jacking tests on plates in pits and expansions of cylinders in drill holes are also employed. Seismic methods may be helpful in conjunction with loading tests to assess the deformation characteristics.

In the laboratory, strength and deformation characteristics can be obtained from triaxial compression tests, but care should be exercised in the choice of specimen size and orientation in relation to the joint pattern, and in ensuring that representative specimens are tested. Furthermore, the interpretation of the results will depend on the relationship of the specimens tested to the overall rock condition.

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.

2.2.2.3.1.3 HARD IGNEOUS AND GNEISSIC ROCKS IN SOUND CONDITION (CLASS 1). The allowable bearing pressure on these rocks is so high that only in very rare cases is the full capacity likely to be called on. 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.

2.2.2.3.1.4 HARD LIMESTONES AND HARD SANDSTONES (CLASS 2). 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.

Limestones, including 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 centimetres wide to large crevasses and caves. In places these are connected to the surface by swallow holes (sinkholes) which may be masked by surface deposits. Solution may also produce an extremely irregular surface to the rock which may be buried under superficial deposits.

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Table 1. Presumed bearing values under vertical static loading
(see definition in 1.2.1)

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 2.1.2.2.1). Reference should be made to other parts of the Code when using this Table.

Group	Class	Types of rocks and soils	Presumed bearing value		Remarks
			kN/m ² *	kgf/cm ² or tonf/ft ² *	
I Rocks	1	Hard igneous and gneissic rocks in sound condition	10 000	100	These values are based on the assumption that the foundations are carried down to unweathered rock
	2	Hard limestones and hard sandstones	4 000	40	
	3	Schists and slates	3 000	30	
	4	Hard shales, hard mudstones and soft sandstones	2 000†	20†	
	5	Soft shales and soft mudstones	600 to 1 000	6 to 10	
	6‡	Hard sound chalk, soft limestone	600	6	
	7	Thinly bedded limestones, sandstones, shales	To be assessed after inspection		
	8	Heavily shattered rocks			
II Non-cohesive soils	9	Compact gravel, or compact sand and gravel	>600	>6	Width of foundation (<i>B</i>) not less than 1 m (3 ft). Ground-water level assumed to be a depth not less than <i>B</i> below the base of the foundation. For relative density and groundwater level see 2.2.2.3.2
	10	Medium dense gravel, or medium dense sand and gravel	200 to 600	2 to 6	
	11	Loose gravel, or loose sand and gravel	<200	<2	
	12	Compact sand	>300	>3	
	13	Medium dense sand	100 to 300	1 to 3	
	14	Loose sand	<100	<1	
III Cohesive soils	15	Very stiff boulder clays and hard clays	300 to 600	3 to 6	Group III is susceptible to long-term consolidation settlement (see 2.1.2.2.2) For consistencies of clays see Table 2
	16	Stiff clays	150 to 300	1.5 to 3	
	17	Firm clays	75 to 150	0.75 to 1.5	
	18	Soft clays and silts	<75	<0.75	
	19	Very soft clays and silts	Not applicable		
IV	20	Peat and organic soils			See 2.2.2.3.4
V	21	Made ground or fill			See 2.2.2.3.5

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

† For weakly-cemented sandstones see 2.2.2.3.1.6.

‡ For disturbed chalk see 2.2.2.3.1.8(1).

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

2.2.2.3.1.5 SCHISTS AND SLATES (CLASS 3). 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.

2.2.2.3.1.6 HARD SHALES, HARD MUDSTONES AND SOFT SANDSTONES (CLASS 4). Hard shales and hard mudstones 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.

The allowable bearing pressure on soft sandstones is very variable, being dependent on the cementing material, and may, for important structures, be assessed taking into account the results of loading tests and crushing tests on representative samples of the rock.

Very weakly cemented sandstones which are so friable that they can easily be disintegrated between the fingers may be found. In spite of this they may stand in cliffs of appreciable height. Frequently beds of stronger sandstone are weathered to this condition near the surface. A presumed bearing value of 1000 kN/m² (10 kgf/cm²) (10 tonf/ft²) would be appropriate. For important structures, a reliable assessment of the allowable bearing pressure may require loading tests.

2.2.2.3.1.7 SOFT SHALES AND SOFT MUDSTONES (CLASS 5). In this class are materials intermediate between hard cohesive soils and rock. These have fairly high bearing capacity but are apt to swell on release of stress, and on exposure to water they are apt to soften and swell appreciably. Particular care is therefore necessary in protecting the surface exposed during excavation to prevent deterioration. Where fissured and permeable, water lowering may be necessary.

2.2.2.3.1.8 CHALK AND SOFT LIMESTONES (CLASS 6)⁴. Chalk and soft limestone include a variety of materials composed mainly of calcium carbonate, and the allowable bearing pressure may vary widely. To justify the presumed bearing value of 600 kN/m² (6 kgf/cm²) (6 tonf/ft²) the chalk or soft limestone will have to be sound and should require a pick for removal.

Chalk and soft limestone will deteriorate when exposed to water or frost. A protective layer of concrete should be laid as soon as the final excavation level is reached.

The possibility should not be overlooked of irregular buried topography, caves and swallow holes in chalk and soft limestone (see 2.2.2.3.1.4). Old underground workings also exist in some chalk and soft limestone areas. Other points to note on these rocks are:

- (1) *Disturbed chalk.* Disturbed chalk includes transported chalk (chalk which has been removed from its original place of deposition) and frost-shattered chalk (chalk which has been subject to frost action in glacial times). Terms such as 'putty chalk', 'solifluxion products' and 'Coombe rock' are sometimes used to describe disturbed chalk. These materials are more akin to cohesive soils, although they may contain rock fragments. The allowable bearing pressure of such chalk may be assessed by comparison with cohesive soils of similar consistency. In some cases it may be necessary to carry out loading tests.
- (2) *Hard sound chalk.* Conventional procedures for taking undisturbed samples in hard sound chalk are likely to destroy the rock structure with the formation of putty chalk, so that the resulting sample is much softer than the in-situ material. The use of these procedures will therefore lead to an incorrect assessment of allowable bearing pressure. A more useful approach is the use of in-situ penetration and loading tests. The results of these in-situ tests may justify a higher value than given in Table 1.
- (3) *Soft limestone.* Limestones may contain various impurities in considerable quantity. *Argillaceous limestone* contains a considerable proportion of clay; with increasing clay content the material is progressively softer and grades into *calcareous clay*. To qualify for inclusion under Class 6 in Table 1 the quantity of clay or silt impurity must be small.

2.2.2.3.1.9 THINLY BEDDED LIMESTONES AND SANDSTONES (CLASS 7). These are stratified rocks occurring in beds several centimetres thick, often separated by clays or soft shales. With such materials the allowable bearing pressure depends very largely on the nature of the layers of clay or soft shale between the rock beds.

2.2.2.3.1.10 HEAVILY SHATTERED ROCKS (CLASS 8). These rocks have been badly cracked and broken up by natural processes including weathering, frost action and faulting. Their allowable bearing pressure needs to be assessed by examination in-situ, including loading tests if necessary.

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2.2.2.3.2 Allowable bearing pressure on non-cohesive soils

2.2.2.3.2.1 GENERAL. This group 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 CP 2001 : 1957, C213 and H160). Their behaviour under load depends on the following factors:

(1) *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, mica. Unless the deposit consists of hard mineral particles, the presumed bearing values in Table 1 may have to be reduced.

(2) *Density of packing.* This is a very important factor; on loose deposits the allowable bearing pressures are generally low, and large settlements may occur.

(3) *Grading.* Deposits composed of a mixture of different sized particles are in general stronger than those where the particles are nearly all one size.

(4) *Size of particles.* In general, the larger the particles, the higher is the allowable bearing pressure.

(5) *Groundwater level.* This affects the stability of foundations, since the allowable bearing pressure is reduced when the groundwater level is near the foundation level.

(6) *Flow of water.* This may have an adverse effect by washing out the finer particles.

(7) *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.

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 form of these tests are the standard penetration test and the Dutch cone test, and they are referred to in CP 2001 : 1957, Appendix J. These tests, if carefully executed in soil undisturbed by boring operations, enable an estimate of the relative density of packing to be made⁶⁻⁸. 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 following broad groups:

Relative density of packing	Number of blows for 0.3 m (1 ft) penetration
Loose	Less than 10 blows
Medium dense	10 to 30 blows
Dense (or compact)	More than 30 blows

A rough correction for the effect of depth may be made¹²⁶.

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 when 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 not only depends 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 of packing, 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 text books and papers⁹.

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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 relative density of packing 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 of packing⁹.

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 ground water 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 (see 2.3.2.4).

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 'runs' or 'blows' 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 pre-determined 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 CP 2001 : 1957, C230). 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 2.1.2.3). In most cases the volume

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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 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:

(1) *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 CP 2001 : 1957, Appendix F.

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.

(2) *Field tests.* Simple methods of estimating the consistency of clays in the field are given in Table I and Appendix C of CP 2001 : 1957. The shear strength of soft sensitive soils may be measured by in-situ vane tests (see CP 2001 : 1957, Appendix J).

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 CP 2001 : 1957, Appendix J. Penetration tests are sometimes used in these soils to compare consistencies. The limitations of such tests should be fully appreciated and they are discussed in CP 2001 : 1957.

(3) *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 CP 2001 : 1957, Appendix H, and BS 1377.

In Table 1, clays and silts are classified on the basis of consistency, which is related to the undrained shear strength as in Table 2 (see CP 2001 : 1957, C230).

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 foundation^{9, 10, 11}. 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 2.2.2.3.2.2. 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.

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.

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Table 2. Undrained (immediate) shear strength of cohesive soils

Consistency			Undrained (immediate) shear strength		
in accordance with CP 2001 : 1957	widely used	Field indications	kN/m ²	kgf/cm ² or tonf/ft ²	lbf/ft ²
Very stiff	Very stiff or hard	Brittle or very tough	greater than 150	greater than 1.5	greater than 3000
Stiff	Stiff	Cannot be moulded in the fingers	100 to 150	1.0 to 1.5	2000 to 3000
	Firm to stiff		75 to 100	0.75 to 1.0	1500 to 2000
Firm	Firm	Can be moulded in the fingers by strong pressure	50 to 75	0.5 to 0.75	1000 to 1500
	Soft to firm		40 to 50	0.4 to 0.5	750 to 1000
Soft	Soft	Easily moulded in the fingers	20 to 40	0.2 to 0.4	375 to 750
Very soft	Very soft	Exudes between the fingers when squeezed in the fist	less than 20	less than 0.2	less than 375

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 2.1.2.2.2 gives rise to considerable and large settlements which may be prolonged indefinitely, and specialist advice should be sought.

The nature of settlement in cohesive soils is discussed in 2.1.2.2.2. 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^{9, 12, 13}. 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^{9, 12, 13}. This method frequently underestimates 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^{14, 15}.

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

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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. 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^{16, 17}.

2.2.2.3.6 Non-uniform site conditions. 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 (see also 2.1.2.3, 2.2.2.3.1).

2.2.2.3.6.1 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 slipped¹⁸.

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 2.3.2.6). 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.

2.2.2.3.6.2 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 of 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.

2.2.2.3.6.3 LAYERS OF SOFTER MATERIAL. The need to avoid overstressing and excessive settlement in soft layers has been referred to 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 conditions¹¹.

2.3 Design: structural considerations

2.3.1 General. The design of the substructure and superstructure and the characteristics of the ground are all interrelated. All foundations move as a result of applied load; there may also be ground movements independent of load (see 2.1.2.3). 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. 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.

2.3.2.1 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 not feasible nor economic to remove.

2.3.2.2 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 structure¹.

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 which 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. The systematic measurement of actual settlements at permanent reference points will therefore have to be regarded as a very desirable adjunct to the design of the structure, and should be practised more widely.

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2.3.2.3 Foundation loads

2.3.2.3.1 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.

2.3.2.3.2 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.

2.3.2.3.3 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%.

2.3.2.3.4 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 2.3.2.3.3 above, 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.

2.3.2.3.5 Settlement. In considering settlement, a close estimate should be made of the average dead and live load likely to be imposed, since loads which 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 2.3.2.4).

2.3.2.3.6 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 loading¹⁹.

2.3.2.3.7 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 = the vertical component of the inclined load,
 H = the horizontal component of the inclined load,
 P_v = the allowable vertical load,
 and P_h = 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 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 means¹⁹.

2.3.2.4 *Dynamic loads*

2.3.2.4.1 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.

The supporting ground under dynamic loading can suffer progressive non-reversible strains. With granular materials in a loose to medium dense state, there can be settlement due to compaction and, in the presence of water, there may be instability due to the build-up of excess pore pressure. Weakly cemented materials may undergo progressive deterioration. Over-consolidated clays may soften under pulsating loading. In the latter case, this 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.

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

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

2.3.2.4.3 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. This condition may lead to excessive amplitudes of vibration of the system.

Even when resonance is avoided it is still necessary to limit amplitudes to levels which can be tolerated by the machine and by the foundation. While the level of vibration may be acceptable by the machine and by 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 (1939)²⁰, and Steffens 1952²¹.

Detailed information concerning methods of design of foundations to sustain dynamic loads can be found in a number of publications, including Barkan (1962)²², Whitman (1966)²³, Whitman and Richart (1967)²⁴. In addition there is an annotated bibliography for CP, 'Foundations for machinery' (in course of preparation).

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 will be found in CP, 'Foundations for machinery'.

A number of factors to be considered in the design are outlined below:

(1) 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.

a. 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.

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b. 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.

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

(2) 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.

(3) In general, the foundations of machinery should not be connected with the foundations of the structure enclosing it: it may occasionally 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.

(4) Typical 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 (for example oil dampers). The spring and dampers are generally placed in an inertia block between the anvil and the foundation element, which is usually a concrete-lined pit.

2.3.2.5 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.

2.3.2.6 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 2.1.2.3.

Erosion of soft or weakly cemented material along faults, dykes or in beds can lead to general or local loss of support. In chalk and limestone country, solution caverns (e.g. swallow holes) (see 2.2.2.3.1.4) 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 excavation, whether for new foundations, for cuttings or for 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 piled 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, and 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 and hence lead 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 3.2.7.1).

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. (Deterioration of the substructure is dealt with 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.

2.3.2.7 Mining subsidence^{27, 28}. In mining areas, subsidence of the ground beneath a building 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 foundation 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 should also be consulted for subsidence history, for previous structural failures, and for the intentions as to future mining.

2.3.3 Types of foundation

2.3.3.1 General. Shallow foundations, including strip, pad and raft foundations, are dealt with in Section 3. Deep foundations include caissons, cylinders, piles and deep basements. The design of deep foundations generally and the design of piles are discussed in Sections 4 and 7 respectively, and the construction aspects of cofferdams and caissons are discussed in Section 5.

2.3.3.2 Choice of foundation. The choice of foundation will be discussed in relation to the soil types given in Table 19, 11, 25.

2.3.3.2.1 Shallow foundations. Shallow foundations are generally suitable for most structures where rock of Classes 1 to 4 occurs near ground level. The allowable bearing pressure will be relatively high, and the proportions of the foundations will normally be economic. Special anchorages, for example drilled cable holes, rock bolts, etc., will be required for tension loading.

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The same remarks apply for shallow foundations on soft shales, mudstone and sound chalk (Classes 5 and 6), but here, for very heavy loadings, consideration of the settlement will be necessary.

Where thinly bedded rocks or heavily shattered rocks occur (Classes 7 and 8), consideration will need to be given to the weakest zones in the significant depth of stressing. In such cases, 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 compact gravel or sand deposits (Classes 9, 10, 12 and 13) above the water table, and also on firm or harder clays (Classes 15, 16 and 17). 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.

Shallow foundations are not generally suitable for soft or very soft clays and silts (Classes 18 and 19), peat and organic soils (Class 20). Where such soils occur in comparatively thin layers, it may be expedient to excavate the soft material and replace it with better material.

Spread foundations should only be used with caution on made ground or uncontrolled fill (Class 21), and then only for lightly loaded structures which 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.

2.3.3.2.2 Deep foundations. When no adequate bearing stratum exists at depths shallow enough to permit economic construction of strip, pad or raft foundations, deep foundations 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 foundation.

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 (Classes 7 and 8), or weathered rocks (Classes 5 and 6) 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 and silts (Classes 18 and 19), peat and organic soils (Class 20) and made ground or fill (Class 21) are met near ground level, in most cases (see 2.3.3.2.1) it will be necessary to employ deep foundations penetrating to an adequate bearing stratum. 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.

In some instances it will be economic to construct a deep basement founded on an adequate bearing stratum. Where the bearing stratum is too deep for a conventional basement, it may be advantageous to use a semi-buoyant foundation (see 4.2.2.2). In areas where an unsaturated upper layer of clay exists, which is likely to 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 a properly designed and ventilated air space beneath the occupied parts. When the occupied parts are in direct contact with the ground, damp proof membranes may be required. Reference should be made to CP 102 and 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 2.3.3.2), 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 pile-driving 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.

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 2.4.3).

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 obtain 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 construction 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 foundation 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 (flood water or the pumping of groundwater) are potential causes of deterioration.

A bearing stratum of clay or certain weak rocks is particularly liable 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, blinding concrete may be insufficient and special precautions will 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.

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 arise 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 CP 2003).

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, for 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 permeability³.

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.

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Pile caps should be made large enough and pile groups should be designed to cover unavoidable variations in the positions of individual piles up to 75 mm (3 in) each way. When the variation is in excess of 75 mm (3 in) it may be necessary to amend the design of the pile cap.

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

2.4.5 Minimum cover in reinforced concrete. The cover to rods in reinforced concrete cast against the ground should be not less than 75 mm (3 in). When the concrete is to be cast against an irregular surface, an increase in cover is necessary to make sure that this minimum is maintained. When the concrete is to be cast upon adequate blinding concrete, the cover excluding the thickness of the blinding may be reduced to 40 mm (1½ in) (see CP 114).

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 (10 ft), and include many strip, pad and raft foundations. The choice of 3 m (10 ft) is arbitrary; shallow foundations where the depth/breadth ratio is high may need to be designed as deep foundations (see 2.1.2.2.1.1). Similarly, some of the considerations in this part apply to deeper foundations where the depth/breadth ratio is low.

For structures 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 is given in 2.3.3.2.1, and the principles of design have been covered in Section 2. This Section 3 of the Code will amplify certain aspects which require special consideration 9, 11, 25, 26.

3.2 Design considerations

3.2.1 General. The depth to which foundations should be carried depends on three principal factors: first, reaching an adequate bearing stratum; second, in the case of clay soils, penetration below the zone where shrinkage and swelling due to seasonal weather changes and due to trees and shrubs are likely to cause appreciable movement; and third, penetration below the zone in which trouble may be expected from frost. Other factors such as ground movements and heat transmitted from the building to the supporting ground may be important.

3.2.2 Allowable bearing pressure. The general principles relating to bearing pressure are covered in Section 2. Where practicable the centre of area of foundation or group of foundations should be arranged vertically under the centre of gravity of the imposed loading (see 2.3.2.3.6).

Where foundation support is provided by a number of separate bases these should, as far as practicable, be proportioned so that differential settlement is a minimum (see 2.3.2.2).

3.2.3 Selection of types of shallow foundation. The selection of the appropriate type of shallow foundation will normally depend on the magnitude and disposition of the structural loads and the bearing capacity of the ground.

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. Walls between columns could be carried on ground beams spanning between the pad foundations. However, strip foundations are appropriate where column loads are comparatively small and closely spaced or where walls are heavy or heavily loaded. Strip foundations are also used with heavy moving loads such as cranes. The incorporation of adjacent pad foundations into a combined foundation can also be used to support eccentric loads to resist overturning or oppose horizontal forces.

Where the allowable bearing pressure would result in large isolated foundations occupying the majority of the available area it is logical to join them to form a raft and spread the loads over the entire area.

3.2.4 Pad foundations. For minor structures, such as two-storey dwelling houses and lightly framed structures, pad foundations are frequently of unreinforced concrete; the angle of spread of load from the pier or base-plate to the outer edge of the ground bearing should not exceed 1 vertical in 1 horizontal. Where brick or masonry foundations have been used, the same rule should apply.

The thickness of the foundation should be not less than 150 mm (6 in).

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For other than minor structures it is customary to use reinforced concrete foundations. This is particularly important where there is insufficient depth to transfer the load to the ground bearing within the permissible angle of spread.

3.2.5 Strip foundations. Similar considerations to those in 3.2.4 apply to reinforced strip 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 may occur. On sloping sites strip foundations should be on a horizontal bearing, stepped where necessary to maintain adequate depth.

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 twice the height of the step, whichever is the greater. The steps should not be of greater height than the thickness of the foundation unless special precautions are taken.

The thickness of unreinforced foundations should be not less than 150 mm (6 in), and care should be taken with the excavation levels to ensure that this minimum thickness is maintained.

3.2.6 Raft foundations

3.2.6.1 General. Raft foundations may be used in the following circumstances:

- (1) 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, swallow holes, etc., where the function of the raft is to bridge weaker zones.
- (2) For heavier structures where the ground conditions are such that there are unlikely to be significant differential settlements. In many such cases the raft can be considered as a nominal one replacing isolated foundations occupying the majority of the available foundation area.
- (3) Where differential settlements are likely to be significant. In such cases, the raft will require special design, involving an assessment of the disposition and distribution of loads and contact pressures (see 2.3.2.5).
- (4) Where mining subsidence is likely to occur. Design of the raft and structure to accommodate mining subsidence requires special consideration, often involving provision of a flexible structure ^{27, 28} (see 2.3.2.7).
- (5) Semi-buoyant 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 (see also 3.2.7.1).

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 Effect of seasonal weather changes

3.2.7.1 Shrinkage and swelling of clay soils. 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 or ground shrinkage, and the development of cracks. The extent and depth of the drying is a function of the lack of precipitation, the evaporative potential of the weather, the supply of water to the area from adjacent bodies of free water, such as rivers and aquifers, and particularly of the extent and concentration of growing vegetation^{25, 29}.

In periods of wet weather clay soils swell, and the cracks tend to close; 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 replenish the existing deficiency of water, a low infiltration rate may prevent replenishment and produce surface run-off, and a subsurface zone, or zones, deficient in water may persist for many years.

Changes in the volume of clay ground are of world-wide importance in the design of foundations, particularly in areas with long, dry seasons; they may well need consideration in Great Britain for foundations located within about 5 m (15 ft) of the ground surface.

In Great Britain, in areas of heavy 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 (5 ft to 6 ft) under crops and grass pastures where irrigation is not practised. Experience shows that under such conditions the strip foundations of traditional brick and masonry buildings should be founded at a depth of at least 1 m (3 ft) in order to reduce the relative movement to acceptable limits.

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The shrinkage of clay may be increased by the drying effect produced by shrubs or trees. The range of influence depends on the species, size and number of the shrubs or trees and increases during dry periods. 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 some 5 m (15 ft). 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.

Buildings with foundations not more than 1 m (3 ft) deep on shrinkable clays should be kept at a distance from all trees of at least the mature height of the tree; if the trees are in rows, the distance should be increased. This rule should also be observed for trees planted after the building is completed. Shrubs may also cause damage to shallow foundations whereas pile foundations will protect a building from shrubs placed near it³⁰. In cases of doubt specialist advice should be sought. Similarly where trees have been removed and swelling is expected, the foundation should generally be carried below the affected zone so that the superstructure is not uplifted irregularly or stretched horizontally by any mass of swelling clay left between the base of the foundation and the lower parts of the superstructure. Alternatively, the structure may be designed to accommodate the expected movements. The degree of protection offered by piled foundations depends on the depth of the piles, and in the worst cases they may need to be founded not less than 5 m to 7 m (15 ft to 20 ft) below ground level³⁰.

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.

3.2.7.2 Effect of frost. Certain soils, particularly silts, chalk, fine sands and some clays, are subject to exceptional expansion when frozen, owing to the formation of layers of ice. Under certain conditions 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 exist, the foundations should be carried below the frost line. For most places in the British Isles a depth of 0.5 m (1 ft 6 in) below ground level will be sufficient for protection in this respect, but in mountainous districts and in areas subject to prolonged frost periods a greater depth will be required. (In very severe winters, high ground near London has been known to be frozen to a depth of 1 m (3 ft).) Care should be taken not to leave, during or after construction, thin concrete rafts or floor slabs exposed to long periods of frosty weather where soils subject to frost heaving are within 0.5 m (1 ft 6 in) of the concrete surface. In colder countries frost may penetrate much more deeply.

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

3.2.8 Effect of heating. Boiler installations, furnaces, kilns, underground cables and services and other artificial sources of heat may cause shrinkage of clay by drying out the ground beneath them; the drying out can be to a substantial depth. They may also cause damage to the foundation material, particularly concrete.

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 in many cases some form of forced ventilation or cooling by circulated water may be required^{31, 32}.

3.2.9 Ground movements. Movements of the ground which may result from changes not connected with the loads applied by the foundation (including those mentioned in 3.2.7 and 3.2.8 above) are listed in 2.1.2.3 and further discussed in 2.2.2.3.5, 2.2.2.3.6 and 2.3.2.6, to which reference should be made.

3.3 Safety precautions

3.3.1 General

Particular attention is drawn to the following statutory requirements affecting the safety, health and welfare of workmen on construction sites:

The Factories Act, 1961³³.

Statutory Instrument 1961 No. 1580,

The Construction (General Provisions) Regulations, 1961.

Statutory Instrument 1961 No. 1581,

The Construction (Lifting Operations) Regulations, 1961.

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Statutory Instrument 1966 No. 94,
 The Construction (Working Places) Regulations, 1966.
 Statutory Instrument 1966 No. 95,
 The Construction (Health and Welfare) Regulations, 1966.
 SR & O 1908 No. 1312,
 The Electricity Regulation 1908 as amended by SR & O 1944 No. 739,
 The Electricity (Factories Act) Special Regulations, 1944.

Further legislation is listed under other sections of this Code and reference should also be made to the 'Guide to the Construction Regulations'³⁴, published by the Federation of Civil Engineering Contractors and the National Federation of Building Trades Employers, to 'Construction Safety'¹²⁷, published by the National Federation of Building Trades Employers, to the 'Building and Construction Regulations Handbook'¹²⁸, 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'¹²⁹.

Before any excavation is started, the person responsible for the work on site should have full knowledge of the ground conditions, including the water pressures in the ground to be excavated and also the ground conditions below the level of the excavation. 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 also CP 2003 and reference ¹¹⁰.

3.3.2 Support of excavations. An excavation should be properly supported, or the sides sloped back to a safe angle (see 2.2.2.3.6.1 and CP 2003), before the excavation reaches a depth where men working in it would be buried or trapped if there were a collapse. A store of suitable supports should be kept on the site to provide immediate strengthening, if found necessary. No timbering or other support for any part of an excavation should be erected or substantially added to, altered or dismantled, except under the direction of a competent person possessing adequate experience of such work. All material for such work should be inspected by a competent person on each occasion before being taken into use, and material found defective in any respect should not be used. The condition of the ground being excavated may necessitate the sides of the excavation being closely supported. This is particularly important when material stacks, excavating plant, cranes or pile-driving apparatus are positioned adjacent to an excavation. The strutting of excavations in proximity to a highway, whatever its use, will need to take into account the support of services such as gas and water mains, sewers and underground tunnels, in addition to the loading on the highway from foot and vehicular traffic. The stability of the excavation should be investigated in relation to the safety of the services, their structural condition and likely movement. Likewise, structures on adjacent lands may necessitate an appraisal of the live and dead loads, and whether the resultant of the loading will cause a surcharge on the sides of the excavation, or whether the excavation will affect the stability of the neighbouring structure. It may be necessary to provide temporary shoring, strutting, ground treatment or other support to the structure to safeguard its stability.

3.3.3 Access. Suitable and sufficient safe access should be provided to all excavations. Attention is drawn to the Statutory Instrument 1961 No. 1580, the Construction (General Provisions) Regulations 1961³³. Reference should be made to these regulations with regard to fencing of excavations.

4. Deep and subaqueous foundations

4.1 General

This Section of the Code deals with the design of deep foundations, i.e. foundations generally deeper than 3 m (10 ft) (see 3.1) but excluding piled foundations, which are covered in Section 7. Construction aspects of cofferdams and caissons are dealt with in Section 5.

The circumstances which require deep foundations have been discussed in 2.3.3.2.2. Careful consideration should be given to possible alternative forms of shallow foundation 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 and may allow the use of additional facilities such as underground car parks, thus increasing the value of the development.

4.2 Types of deep foundation

4.2.1 General. Deep foundations may be classified under the following headings:

(1) *Basements or hollow boxes.* These may be constructed in open unsupported excavation or in supported excavation.

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(2) *Caissons*. These may be open well caissons (monoliths) or pneumatic caissons.

(3) *Cylinders and shafts*. These may be sunk in the dry by hand or mechanical excavation or, in the wet, by mechanical excavation including boring techniques.

(4) *Piles*.

(5) *Diaphragm walls*. Concrete walls constructed in a mud filled trench, as well as being used as basement or retaining walls, may also carry vertical loads either in conjunction with their retaining function or as separate foundation elements.

(6) *Combined foundations*. These may be a combination of any of the above.

4.2.2 Choice of type of deep foundations

4.2.2.1 General. Given satisfactory performance of the type of deep foundation selected, economy is then the only criterion 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 nearness of other structures or services. A basement foundation may be selected because ground of better bearing characteristics can be reached by it or because of the advantage of the extra space gained.

In general, it may be said that, where a deep foundation is unavoidable, a large number of fairly small but dispersed loads can most economically be carried by piles, either pre-fabricated or in situ. Heavy concentrated loads are better carried on piers, cylinders, large bored piles or caissons. Obviously the larger the load, the more substantial the foundation element must be.

4.2.2.2 Basements or hollow boxes. 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 waterproofing when a watertight structure is required (see CP 102), and allowance will need to be made for the lateral earth pressure or 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 cofferdam will be necessary. Cofferdams are discussed in detail in Section 5 but they should be designed to work in economically with construction. In addition to the extra space gained by basement construction, a further advantage is the reduction in net loading obtained. The maximum exploitation is obtained where soft soils exist to a very great depth by using a buoyant foundation. The weight of the structure and substructure, including any imposed loading, then balances the weight of excavated material, and the increase in stress at foundation level is zero. Such a foundation can involve problems of heave and the effect of fluctuating groundwater; large variations in live loading can give rise to additional movement. In practice, therefore, such foundations are usually designed to be semi-buoyant and to rely for part of their support on the strength of the ground or on piles. Buoyant foundations are sometimes installed as cellular units, the soil being removed from within the cells until the foundation sinks to the required level, due care being taken to avoid inflow of ground and instability.

Where the excavation is relatively straightforward and the ground at formation level provides adequate bearing for constructional operations, and where use can be made of the extra space, the semi-buoyant foundation may show some economy over other forms of deep foundation. However, where elaborate methods of construction, such as the sinking of cellular units, are necessary, such foundations are rarely economic unless the depth to an adequate bearing stratum is great^{11, 25}.

4.2.2.3 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 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 sunk by excavation by hand 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 30 m (100 ft) below water level. Open well caissons have no such restrictions and have been used to some 80 m (250 ft) below water level.

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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* to simplify the task of constructing the lowest part of the caisson. They have also been used to form the foundations for buildings when the space within the caisson can be used for basements.

The cost of labour working in compressed air is so high that other possible forms of foundation should be investigated carefully before a pneumatic caisson is adopted. In addition, recent investigation has indicated certain hazards to health not previously recognized ³⁵(see 6.5.1).

4.2.2.4. Cylinders, shafts and large bored piles. Cylinders are essentially small open well caissons comprising a single cell. 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 as the cylinder is being sunk. Cylinders are sometimes sunk by the method of *underpinning*. Part of the ground beneath the forward edge of the cylinder is removed and a ring segment is installed, and so on. This method is likely to be employed only with special ground conditions and in the dry.

Below the water table, cylinder sinking methods 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 7.4.5).

Cylinders are often filled with concrete and are sometimes reinforced.

4.2.2.5 Piles. Piled foundations are described in detail in Section 7.

4.2.2.6 Diaphragm walls ³⁶. Where diaphragm walls, formed by excavating and concreting in special mud filled trenches, are intended to carry vertical loads, special care needs to be taken to ensure that the concrete is placed on a clean bottom to the trench. Furthermore, the possibility of disturbance and deterioration of the natural ground should be borne in mind since this will also lead to additional settlement. Under such conditions a considerable settlement should be allowed for during the early stages of loading (see 6.5.3.)

4.2.2.7 Combined foundations. Where ground conditions are not uniform, it is advisable to consider whether the use of more than one type of foundation would result in greater economy. For example, if an old buried river channel crosses a site where a satisfactory bearing stratum otherwise exists near ground level, then it might be economical to use shallow foundations combined with piles in the deeper part. When using combined foundations, special attention will need to be paid to the effect on the structure, particularly where differential settlements can occur.

4.3 Alteration of the ground as a result of excavation

4.3.1 General. The process of excavation for a deep foundation and the methods of temporary support are likely to bring about important changes in the ground conditions, which need to be considered in the foundation design. In general, no matter what method of excavation is adopted, a downward and inward displacement of the ground immediately surrounding the excavation will occur, while the central area below the excavation will uplift as the excavation is carried out. Such movements in the most favourable conditions will be principally of an elastic nature but, where the ground is chiefly clay, swelling and softening or shrinkage and hardening will occur in the course of time.

In the British Isles climate 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.

In dry climates, on the contrary, 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. Complete restriction of vertical movements alongside and beneath the excavation may not be practicable in some circumstances, and allowance for this and for its effects on adjoining property should be made in the design of the temporary and permanent works.

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4.3.2 Effect of excavation on skin friction. Except where the surrounding ground is settling, deep foundations may obtain some vertical support from the ground in contact with the sides of the structure, but the amount of support in the form of skin friction will depend to a large extent on the method of excavation.

The placing of backfill into an open excavation with sloping sides may cause a settlement of the ground from its heaved position and settlement of the completed foundation, and for the purposes of foundation design it should be assumed that no skin friction develops around the vertical sides of the foundations to assist support. Indeed, if highly compressible filling is used it may tend to drag the foundation down.

Where the excavation is made with vertical sides supported temporarily with walls, such as sheet-pile cofferdams, cylindrical shafts, caissons, concrete walls cast in mud filled 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 will depend on the construction method. 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, or in the construction of diaphragm walls, the skin friction may be quite small. In some cases, for example in sinking caissons, it is possible to measure the average skin friction present during construction.

4.3.3 Effect of excavation on lateral pressure. The horizontal forces acting on the foundation will depend very greatly on the influence of the method of excavation on the properties of the surrounding ground. The influence of several different methods of excavation may be mentioned.

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 which is heavily compacted or likely to swell may exert a considerable lateral pressure on the foundation. Consideration should be given to the possibility of using granular material lightly compacted to exert a small lateral pressure, and to its complete drainage, bearing in mind that a reduction in the allowance for overburden pressure may need to be made in considering the bearing capacity.

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 by 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 overburden pressure. 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.

4.3.4 Reducing deterioration of the ground below excavations. Where sands and gravels or laminated deposits with a high horizontal permeability exist at or below foundation level, it is essential to reduce the water pressure during construction so that there is no tendency for the bottom to lift or 'boil' on account of excess water pressure. Effective methods of reducing the water pressure are discussed in reference²⁶.

The liability of the bottom to deform plastically as a mass in open excavations in cohesive soils may be estimated by the usual methods of slope stability analysis. Where the excavation is made with laterally supported walls, the liability for the bottom to heave plastically may be estimated by methods such as those given by Bjerrum and Eide³⁸. A factor of safety of at least 1.5 should be used against plastic deformation, otherwise considerable weakening of the foundation soil and additional settlement can be expected during reloading of the ground.

Where the foundation soil consists of over-consolidated clays and 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 (5 ft to 10 ft) of ground in small areas and replacing it with a substantial thickness of concrete³⁹ (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.

Swelling and softening at foundation level are augmented by water, by freezing and thawing and by traffic. Any foundation material which 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 prevent swelling.

4.4 Heave, bearing capacity and settlement

Estimates of the bearing capacity and settlement of structures should take into account the condition of the ground after it has been changed by the construction. In the British Isles climate the bearing capacity will generally be smaller and the settlement larger than for the virgin ground. Thus it follows that the amount and rates of settlement of structures in these conditions may be larger and more rapid than expected. Settlement calculations should therefore assume a deterioration or heaved condition. The reliability of such calculations is not well established, since there is little experience or comparison with field performance

4.4.1 Methods of estimating heaving and softening rates

4.4.1.1 General. The amount of vertical heaving and the rate at which it occurs depends on the reduction in vertical stress created by the construction, on the type and succession of underlying strata and on the groundwater and surface-water conditions.

In moderately homogeneous deposits the heave may be separated into two components. First, there is an elastic type of heave which occurs simultaneously with the progress of excavation, which does not involve migration of pore water within the deposit and may pass unnoticed unless it is measured^{9, 40, 41}. Second, there is a slow, long-term swelling involving absorption of water from the upper surface or migration from below, which may continue for many years or until adequate reloading by the foundation construction restores the equilibrium^{37, 42-44}. In deposits where there is high horizontal permeability heave can be rapid.

Foundations, or parts of foundations, which do not restore the original effective vertical stress to the ground may continue to lift many years after construction with tilting if the ground is loaded unsymmetrically.

In soils such as sand, gravel and rocks, which do not contain active clay minerals, the amount of elastic heave and swelling is small and the swelling rapid. In normally consolidated and over-consolidated clays the amount of elastic heave and the swelling increase with the clay content and the activity of the clay. The rate of swelling is slow, but the amount can be large (many centimetres).

4.4.1.2 Amount of elastic heave. The amount of elastic heave will, in general, be greater in the centre of a level excavation in level homogeneous ground and less at its boundaries⁴⁵. The amount of heave in the central part of an excavation that is large in plan dimensions compared with its depth could be estimated from elastic theory, given a knowledge of the appropriate elastic parameters (Young's modulus and Poisson ratio). Unfortunately, at the present time this determination is not within the scope of usual practice. The attempt may be made to measure these parameters in the laboratory on high quality samples of the ground or they may be determined more accurately from field measurements^{9, 37, 40, 45}. (It should be added that there is a lack of full-scale field observations, so that there are considerable uncertainties in estimates of heave.)

It is not possible at present to estimate the amount of elastic heave of the ground in the vicinity of the excavation boundaries; the movement will be less in such positions, and it may even be a settlement⁴⁵.

4.4.1.3 Amount and rates of swelling and softening. There is not much experience of the reliability of estimates of the amount, and still less of the rates, of swelling and softening in the ground beneath excavations on the basis of laboratory tests on soil samples. There are very few field measurements. One of the major problems in the estimates arises from the difficulty of establishing the sources of free water which feed the expanding soil. Better estimates can probably be made in thick homogeneous beds of normally consolidated or over-consolidated clays than in more complex ground conditions. Laboratory measurements may be made of the expansibility and the coefficient of swelling of undisturbed samples in the oedometer under the appropriate reductions in the effective vertical stress. Calculations of the amount and rate of swelling can then be made in a similar way to that used for consolidation settlement calculations⁴³.

The extent and degree of softening of clays can also be estimated if a relationship is established in the laboratory between the degree of swelling and the shearing resistance.

4.4.2 Allowable bearing pressure and settlement calculations

4.4.2.1 General. Calculation of allowable bearing pressure and settlement calculations will in general follow the form already indicated in 2.2 and 2.3.

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Calculations of the load applied to the ground by deep foundations should follow the recommendations given in 2.3.2.3, due allowance being made for the upward water pressure, which can be large.

In general the total load (superstructure plus substructure) applied to a deep foundation should exceed the total weight of excavated material, otherwise heaving may occur after construction even in saturated ground. In any case the total load at all stages during construction and after should exceed the maximum possible ground water upthrust, otherwise the structure may suddenly rise if the groundwater level rises.

Replacement of the total excavated weight will often involve some settlement of the structure and the underlying ground on account of deterioration of the ground during excavation.

4.4.2.2 Sharing of vertical load between the sides and base of a deep foundation. Where the depth/width ratio of a foundation is large and the ground is homogeneous, the full shearing resistance between the vertical sides of a foundation and the ground is likely to be mobilized at a relatively small settlement compared with the settlement associated with allowable loading of the base of the foundation. This condition is similar to that when a single pile is loaded⁴⁶.

Where the ground is homogeneous and the width of the foundation is much greater than the depth, it is unlikely that any appreciable shearing resistance will be mobilized at the sides of the foundation; in most other ground conditions the resistance will be small.

Where the foundation is deep compared with its width and its base bears on a more resistant stratum, such as rock, it is unlikely that the full shearing resistance between the ground and the sides of the foundation will be developed.

4.4.2.3 Effect of settling ground and negative skin friction. On sites containing recent clay, silt, peat or fill, especially where filling has recently been placed on the earlier surface, the ground will settle around a deep foundation including piles taken to a firmer stratum. 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.

4.4.2.4 Buoyant and partially buoyant foundations. It is important with buoyant and partially buoyant foundations to consider the tilting effects which can occur if the substructure applies an asymmetrical load to the ground; for example, when a heavy, tall tower is located to one side of a hollow box foundation. This requirement is of particular importance on clays, saturated or unsaturated, where swelling is likely to occur.

4.4.2.5 Distribution of load in combined foundations. Deep basement foundations are often used with piled foundations beneath them in order to reduce settlements. In design it is normally assumed that the load is transferred to the ground by means of the piles only. This is probably a conservative assumption where the basement is built in contact with a saturated clay on which the overburden has been reduced by excavation, and it is likely to be an erroneous assumption where the clay is unsaturated and water is admitted. In both cases the clay will swell; in the first case a vertical pressure on the basement rather less than the original overburden pressure is likely to be mobilized, and in the second case a considerably greater pressure may develop depending on the degree of unsaturation. An adequate space, or a sufficient layer of compressible material such as foamed plastics, is necessary between the basement and the underlying ground to ensure that the piles carry the base load of the foundation, otherwise the piles may act as vertical anchors rather than supports.

4.5 Safety precautions ³³

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 men falling into them. Therefore particular attention should be paid to safety precautions, both in the design and in the construction of the works.

The provisions in 3.3 apply equally to deep foundations and should be studied in detail. Reference should also be made to other relevant clauses of this Code as under:

Cofferdams and caissons	5.4
Special methods for excluding water from excavations	6.5
Pile foundations	7.6
Tide work	8.4
Excavation	9.1.9
Demolition, shoring and underpinning	9.6

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5. Cofferdams and caissons**5.1 General**

5.1.1 Introduction. The provisions of this Section of the Code 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.

5.1.2 Preliminary investigations. An investigation of the ground and other local conditions should be made in accordance with the recommendations of CP 2001 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 'blow'. 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 requirements of the appropriate Codes of Practice and British Standards.

5.2.1.1 Timber. 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 CP 112 and in the Bulletins of the Forest Products Research Laboratory.

For cofferdams the quality used should be not less than 50 Grade as defined in CP 112. The stresses applicable to green timber ('green stresses') should be used. Where available, timber of species Group S1 should be preferred. These include imported Douglas fir and pitch pine and home grown Douglas fir and larch. Home grown Scots pine may also be used. This is in species Group S2.

5.2.1.1.1 Stresses. As a guide the 'green stresses' for 65 Grade and 50 Grade timber of the species Group S1 are as in Table 3, the figures in imperial units being extracted from CP 112, Table 10. That table also gives the corresponding figures for other species and grades.

Table 3. Species group S1 green stresses

	65 Grade			50 Grade		
	SI	Metric	Imperial	SI	Metric	Imperial
	kN/m ²	kgf/cm ²	lbf/in ²	kN/m ²	kgf/cm ²	lbf/in ²
Bending and tension parallel to grain	9 000	90	1 300	7 000	70	1 000
Compression parallel to grain	6 000	60	900	5 000	50	700
Compression perpendicular to grain	1 500	15	220	1 300	13	190
Shear parallel to grain	900	9	130	700	7	100
Modulus of elasticity						
Mean	9 000 000	90 000	1 300 000	9 000 000	90 000	1 300 000
Minimum	4 500 000	45 000	650 000	4 500 000	45 000	650 000

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5.2.1.1.2 Permissible stresses. The permissible (or working) stresses should be arrived at by applying the relevant modification factors set out in CP 112. The design of the members and fastenings should follow the relevant clauses in that Code.

5.2.1.1.2.1 SLENDERNESS RATIO. The basic stress should be modified in accordance with CP 112. The effective length of struts should be taken as recommended in 5.3.3.8.5.

5.2.1.1.2.2 COMPRESSION MEMBERS SUBJECT TO BENDING. The stresses in such members should not exceed those calculated using Table 25 of CP 112.

5.2.1.1.2.3 FACTOR FOR DETERIORATION. The basic stresses should be reduced by 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.

5.2.1.1.2.4 FACTOR FOR TEMPORARY NATURE OF BRACING. The bracing should be designed for the most severe conditions which it may be called upon to withstand. If these can be assessed with confidence and if the probability of their occurrence during the life of the cofferdam is small, then the basic stress may be multiplied by a factor not exceeding 1.5.

5.2.1.2 Reinforced concrete. The quality of concrete should be in accordance with clauses 201 to 209 of CP 114, and the stresses should be in accordance with Clause 303. The stresses for temporary work or for precast components that are only temporarily subjected to loading, which can be confidently assessed, may however be increased at discretion above the normal stresses, but in no circumstances to more than $33\frac{1}{3}\%$ above those specified in the Code. If the work is carried out under adverse conditions, lower stresses should be used.

5.2.1.3 Steel. The steel used in the construction of cofferdams and caissons should comply with the requirements of BS 4360; workmanship and stresses should be in accordance with BS 449, with a permissible increase at discretion up to 25% for temporary work or temporarily loaded members where the loading can be assessed with confidence.

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 is 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 choice of type of construction 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.

Generally, cofferdams are used for depths up to 18 m (60 ft) below water level, depending on the nature of the soil. For deep cofferdams it may be preferable not to lower the water below a certain level, the last stages of excavation and concreting 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 the recommendations of Civil Engineering Code of Practice No. 2 'Earth retaining structures' and the recommendations of this Code, CP 2004. During design a review should be made with the object of identifying any adverse forces or conditions not specifically mentioned in the Codes and provision should be made for these.

5.3.3 Cofferdams

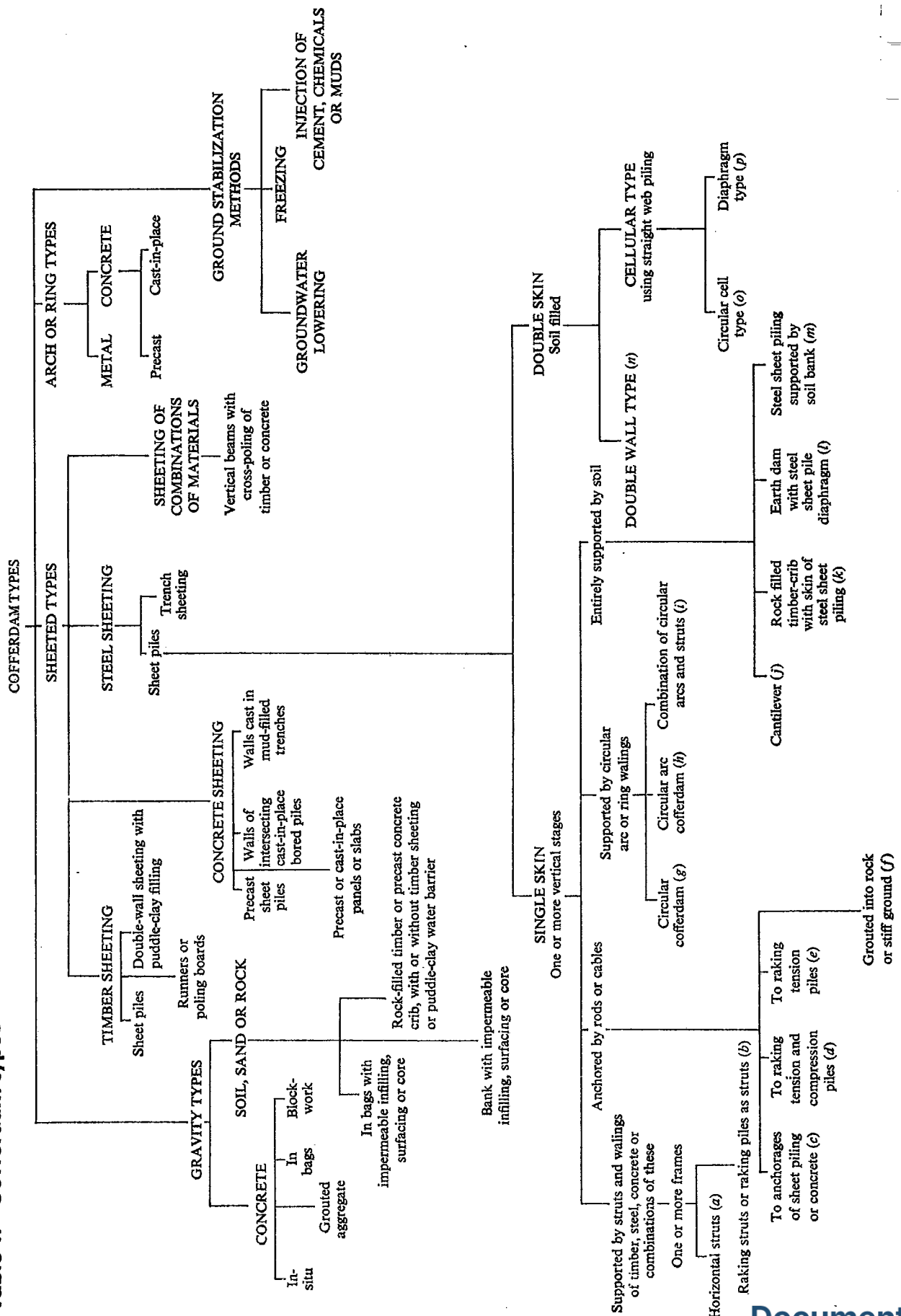
5.3.3.1 General classification. The main types of cofferdam are indicated in Table 4, the upper part showing all types, the lower those with steel sheet piling.

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:

- (1) Whether a land cofferdam or water cofferdam is required.

Table 4. Cofferdam types



The above types are illustrated in Fig. 1. The appropriate part of Fig. 1 being the letter given in parentheses.

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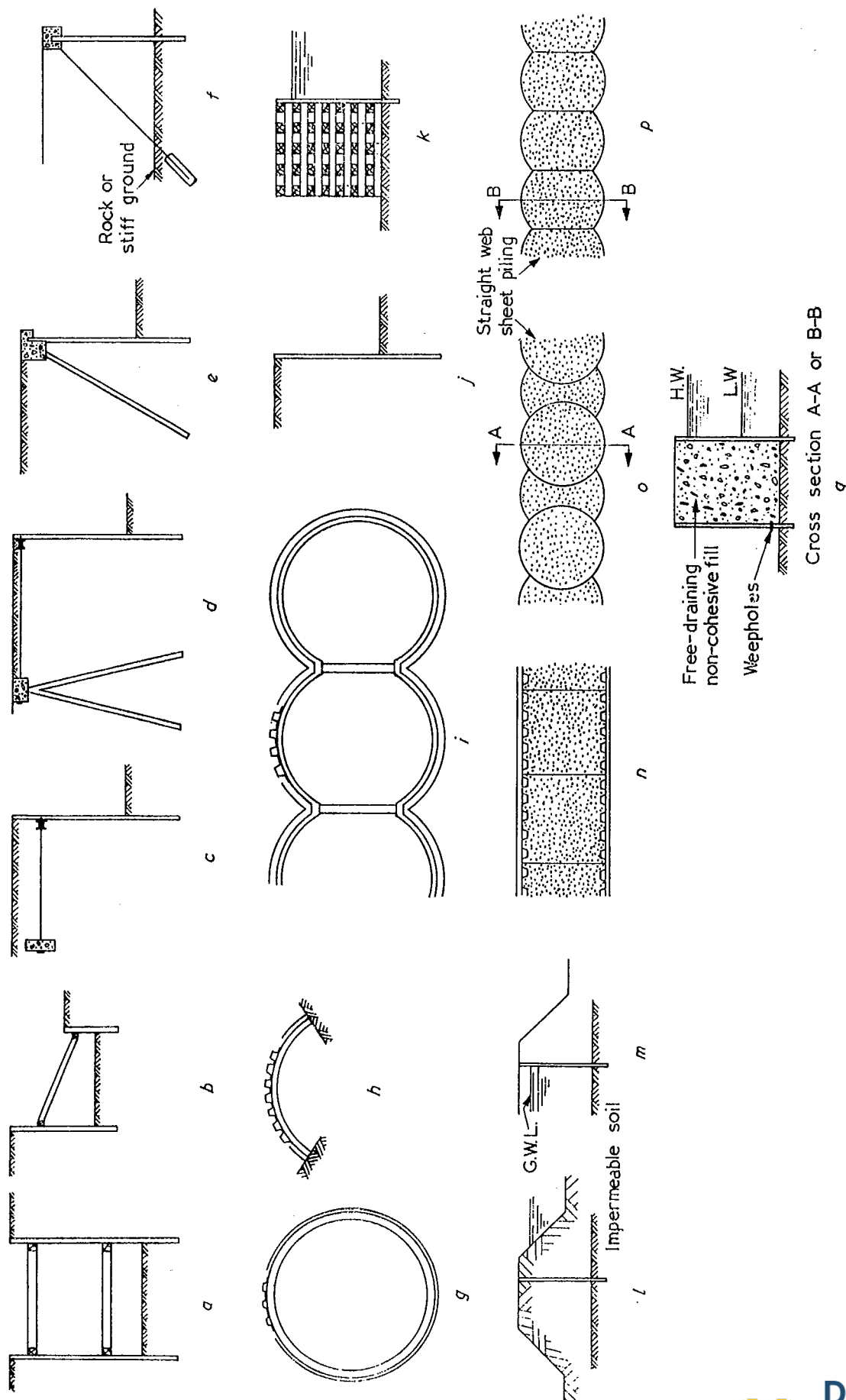


Fig. 1. Cofferdams using sheet piling (see Table 4)

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- (2) The nature of the permanent structure to be built within the cofferdam.
- (3) The plan dimensions of the working area required inside the cofferdam.
- (4) The total depth of soil and/or water to be retained.
- (5) Soil conditions below and above foundation level.
- (6) Groundwater levels and their fluctuation, and for water cofferdams the tidal, seasonal and flood levels.
- (7) For water cofferdams, the strength of the current, wave action and scour before, during and after construction.
- (8) Possible effects of the cofferdam construction on existing buildings or other structures close to it.
- (9) Availability of materials.
- (10) Methods of constructing and dismantling the cofferdam.
- (11) Time available for construction of the cofferdam.
- (12) Noise, vibration, fumes and fire risk.
- (13) Relative total costs of the cofferdams where two or more types are otherwise suitable.

5.3.3.2 Certain of the types shown in Table 4 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 which can be considered are:

- (1) combinations of cofferdam types;
- (2) final excavation carried out under water;
- (3) 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. These 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 CP 2003.)

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.

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 (12 ft) square.

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

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The timber cribs may be floated into position, suitably weighted with rock for sinking and then completely filled with rock, including finer material to check infiltration. 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. Double-skin cofferdams are self-supporting gravity structures, either of 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.

5.3.3.5.1 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 'Earth retaining structures'), while the outer line of piling acts as the anchorage. Ordinary trough types of steel sheet piling are, therefore, appropriate. The width of the dam should be not less than about 0.8 times the retained height of water and/or soil.

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 be 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.

5.3.3.5.2 Cellular cofferdams. Cellular cofferdams are constructed in the manner shown in Fig. 1, drawings *a*, *p* and *q*, 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 only be satisfactory on a clay foundation 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.

Sheeting without any bracing should be designed as cantilevers fixed in the ground. 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 follow 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 which 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 centre line 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 indicated in the following paragraph.

The shear forces in the interlocks may then 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:

- (1) the piling passes through very soft clay or other weak material;
- (2) it is prevented by rock from penetrating to the normal depth of cut-off;
- (3) where the piling is used as a cantilever or if it is cantilevered to a substantial height above the level of the highest waling;
- (4) if there is back-filling 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 clause is concerned only with the case where the soil at and below final excavation level 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. Quite apart from the question of stability, a reasonably large cut-off will also have the effect of reducing the amount of pumping capacity required to keep the cofferdam dry.

The following three cases differ in the manner of pumping:

- (1) *Pumping from sumps at excavation level only.* Upward seepage of water
 - a. causes a reduction in the passive resistance of the soil below excavation level^{13, 47}, and
 - b. may be sufficient to cause 'boiling' of the soil at excavation level.

To counteract the effect of a 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-off⁴⁸ as indicated in Table 5 and Fig. 2. Table 5 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. Very fine uniform sands may require a greater cut-off than shown ⁴⁹.

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The tendency to boiling can be reduced, in the case of a water cofferdam, 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 which results if the cut-off is increased. Boiling may also be averted by sinking an array of relief wells inside the cofferdam which extend well below the final excavation level. Relief wells are not pumped and may consist of porous or perforated pipes, or boreholes filled with a 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.

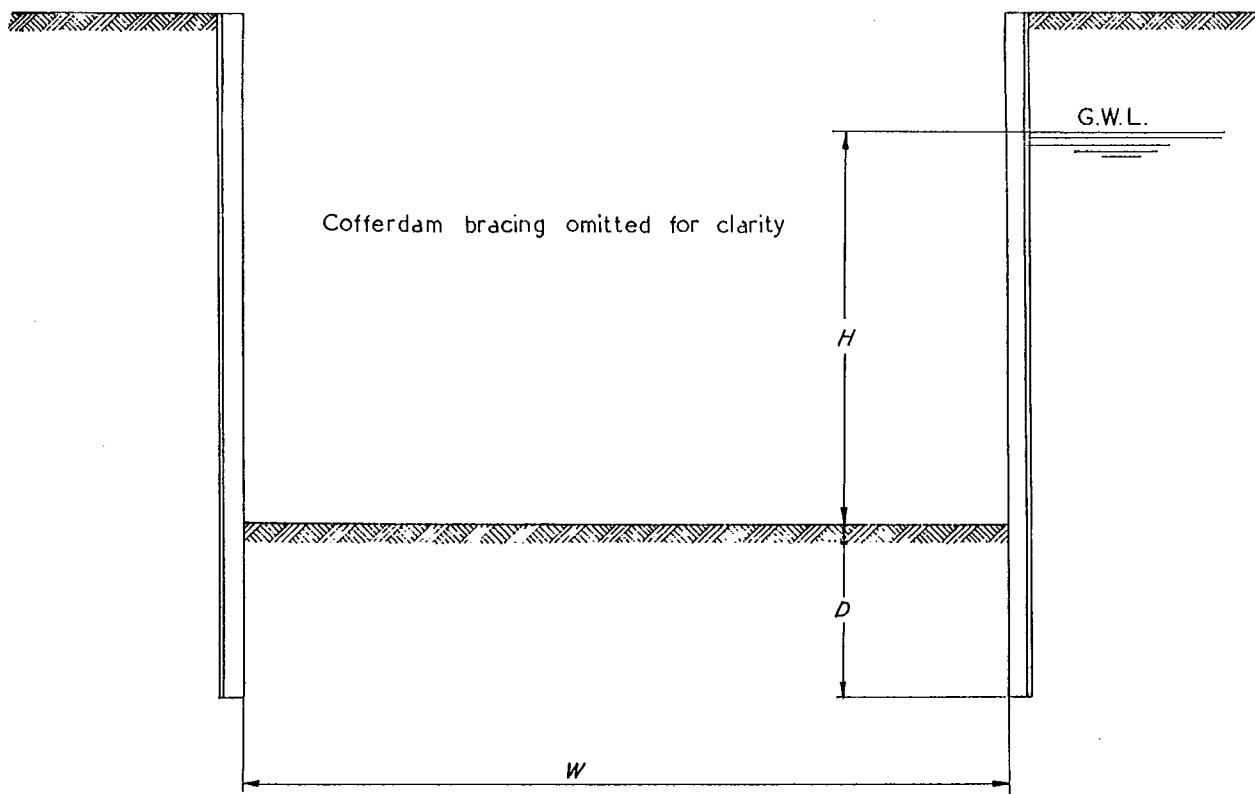


Fig. 2. Cofferdam showing depth of cut-off (cohesionless soils)

Table 5. Minimum values for depth of cut-off for cohesionless soils where there is no significant lowering of the external water level (see Fig. 2)

Width of cofferdam, W	Depth of cut off, D
$2H$ or more	$0.4H$
H	$0.5H$
$0.5H$	$0.7H$

(2) *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.

(3) *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* and *b* in 5.3.3.7.3(1) are eliminated, and cut-off should be determined as in 5.3.3.7.3(2). If the excavation level is below the lowered groundwater level, then when applying Table 5, *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 5.3.3.7.3(2) or 5.3.3.7.3(3) should be adopted, or all the piles should be driven before excavation.

The effects *a* and *b* in 5.3.3.7.3(1) can also be eliminated by carrying out final excavation under water and depositing a sufficiently thick concrete base plug under water 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:

- (1) to take out the final excavation in narrow widths transverse to the cofferdam and replace by concrete, or
- (2) to employ relief wells within the cofferdam extending into the lower permeable stratum⁵⁰, or
- (3) to employ borehole pumps in deep wells in the same manner, or
- (4) to carry out final excavation under water and then deposit an adequate base plug of concrete 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^{38, 9}. 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 it will cause loss of strength owing to remoulding, and this effect should be allowed for in the design⁵¹.

5.3.3.8 *Design of bracing*

5.3.3.8.1 General. Cofferdams may be braced by one or more settings of walings or 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 the 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