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Code of practice for

Earthworks

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Contents

Foreword

This code of practice is a revision of CP 2003, published in 1959, and has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee. It deals with earthworks forming part of general civil engineering construction such as highways, railways and airfields; bulk excavations for major structures and excavations in pits, shafts and trenches for foundations, pipelines and drainage works. The code excludes tunnel excavations; earthworks for dams and reservoirs; dredging and reclamation; and earthworks for canals, river training, sea defences, irrigation and land drainage. CP 2003 is now withdrawn.

The code is written in general terms and its application to any particular branch of civil engineering or to any particular construction project may be subject to the special requirements of the work under consideration.

This code is divided into three sections. Section 1 is a general section and includes information on safety procedures.

Section 2 deals with cuttings and embankments, grading and levelling, and describes methods of designing and constructing these earthworks for highways, railways and airfields. Methods of bulk excavation in open ground conditions are also described.

Section 3 deals with trenches, pits and shafts, describing methods of excavation and of providing temporary support for the sides.

Generally the code has been prepared in relation to conditions existing in the British Isles. Some of the recommendations may not be appropriate for work in countries overseas where climatic and other factors require different design and construction techniques.

The construction methods described in the code are necessarily quite basic in conception. Some of them, especially those used for supporting the sides of excavations, are traditional in character, and show few changes from the methods described in the 1959 edition of this code. The newer techniques usually involve utilization of the permanent substructure as a means of ground support and they are dealt with in Civil Engineering Code of practice 2 (under revision) and in CP 2004.

In this revision changes have been made in the methods described for assessing the stability of cuttings and embankments. Advances in the science of soil and rock mechanics since the preparation of the 1959 code have added considerably to the knowledge and understanding of the behaviour of earth and rock slopes.

At the time of revising this code, methods of limit state design as adopted in CP 110 have not been applied to earthworks. This is because of the different concepts concerning the engineering properties of soils and rocks and the dominant time-dependent character of stability and deformation problems. In the CP 110 design method, characteristic values can be adopted for the dead load of the structure, the imposed and wind loadings, and the strength and deformation properties of the constituent materials. These characteristic values can be multiplied by partial safety factors which take into account statistical variations in the applied loadings and measured properties of materials. If the factored dead, imposed and wind loadings do not exceed the factored calculated strength of the structure, the ultimate limit state has not been reached. No similar approach has yet been established for earthworks as a practical method of design. In earthworks very large potential shear surfaces are involved compared with those in failure zones of structural members. The soils and rocks traversed by these large shear surfaces can vary widely in composition, fabric and shear strength depending on how the materials were originally formed, the effects of time on their fabric and strength, their stratigraphic level and inclination in relation to the earthworks profile, the changes in strength of the soil or rock mass consequent on constructing the earthworks, and variations in the ground water regime.

Earthworks can undergo large deformations before the peak shear strength is mobilized and failure takes place. The serviceability limit is therefore likely to be that of the structures sited within or adjacent to the earthworks rather than a factor of the earthworks alone.

Earthworks may be undertaken in site conditions influencing the stability of existing structures and thus affect the lives and property of the general public. The safety of construction operatives has also been an important consideration in preparing this revision of the code. It is therefore strongly recommended that when the consequences of failure of earthworks could be serious, adequate measures be taken to ensure safe working conditions and permanent stability of the completed works. Expert advice should be sought on the problems and risks involved.

Attention is drawn to need to obtain local authority planning approval for earthworks such as cuttings, embankments, borrow pits and spoil heaps. Measures may be required to deal with wide-ranging effects on the environment. Early consultation with the appropriate national and local government authorities and conservation societies is therefore recommended.

NOTE The numbers is square brackets used throughout the text of this code relate to the bibliographic references given in Appendix B.

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

This code of practice represents a standard of good practice and therefore takes the form of recommendations. Compliance with it does not confer immunity from relevant statutory and legal requirements.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to x, pages 1 to 110, an inside back cover and a back cover.

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

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Section 1. General

1 Scope

This British Standard recommends a code of practice for earthworks forming part of general civil engineering construction.

Section 2 of the code describes methods of designing and constructing cuttings and bulk excavation. It also covers embankments and areas of general regrading in cut and fill for highways, railways and airfields. Certain works such as docks and power stations include both bulk excavation in open ground and deep excavations with supported sides, the latter being dealt with in section 3 of this code. Where such supported excavations do not represent a large proportion of the total volume of earth-moving they will be dealt with mainly in section 2.

Although site investigations for general purposes are described in BS 5930[1)], the methods of investigation for stability of slopes of cuttings and embankments require additional specialist techniques and equipment which are described in the present code.

Section 3 of the code describes methods of excavating trenches, pits and shafts in various types of ground and methods of forming temporary supports to the sides. Trenches, pits and shafts are excavated to enable some form of permanent construction to be founded at the level of the bottom of the excavation and any permanent support to the ground faces to be formed.

Temporary supports referred to throughout the code are taken to include alternative materials such as timber, steel and reinforced concrete, and alternative methods such as timbering, trench sheeting, sheet piling, diaphragm walls and contiguous bored piled walls.

Figure 21 to Figure 35 are published to illustrate specific technical requirements referred to in section 3; they are not intended to show all details or to prescribe the design in any other respect. Timber has been shown in some of these figures because it is the traditional material, but this does not preclude the use of other materials which may be found more expedient (see **13.1**). Any members omitted (e.g. puncheons, tipping pieces, lacings, ground props) are to be provided where necessary.

2 References

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

3 Definitions

3.1 General

Any earthworks project involves considerations of soil and rock mechanics practice, and the code makes frequent reference to terms used in these sciences. Definitions of terms related to soil testing are given in BS 1377. Engineering geology and rock mechanics terms are defined in various published works [1,2][2)]. For the purposes of this British Standard, the definitions given in **3.2** to **3.4** apply, together with those given in BS 1377 and [1,2].

3.2 Terms used in section 2

3.2.1
anisotropy

the property of having different physical properties in different directions; e.g. the permeability of a soil may be greater in a horizontal direction than in a vertical direction

3.2.2
aquifer

a soil layer containing water in recoverable quantities

3.2.3
berm

a relatively narrow bench or shelf which is provided to break the continuity of a long slope, or as a trap to contain loose material rolling down a slope

3.2.4
colluvial deposits

weathered material transported by gravity, e.g. scree, talus, and landslip debris

3.2.5
creep

barely perceptible movement of a soil or rock mass. There is often a continuous gradation between the stationary and moving material, but this does not occur in the case of creep on a pre-existing slip surface

3.2.6
deviator stress

in a triaxial compression test on a specimen, the difference between the greatest and least principal stresses

1) Formerly known as CP 2001.

2) The numbers in square brackets used throughout the text of this code relate to the bibliographic references, given in Appendix B.

3.2.7
landslide (landslip)

readily perceptible down-slope movement of a soil or rock mass, occurring primarily through shear failure on discrete surfaces at the boundaries of the moving mass

3.2.8
phreatic surface

the level to which the ground water from a given aquifer will rise under the full head

3.2.9
piezometer

an open or closed tube or other device installed downward from the ground surface and used to measure the ground water pressure in the region where the piezometer tip is situated

3.2.10
scree

accumulated rock debris at the foot of a cliff (see colluvial deposits)

3.2.11
slope angle

the angle of any slope expressed either in degrees to the horizontal or as the tangent of the angle to the horizontal (e.g. a slope of 1 in 3 makes an angle to the horizontal whose tangent is 1/3, i.e., 18.5°)

3.2.12
solifluction

the slow downhill movement of soil or scree cover as a result of the alternate freezing and thawing of the contained water

3.2.13
spoil

soil, rock or other excavated material which is not required for filling in embankments or as backfill of excavations, and is surplus material removed from the site

3.2.14
subsidence

downward movement, predominantly vertical in direction, due to removal, consolidation, or displacement of the underlying strata

3.2.15
tectonic movements

the displacement of a rock mass relative to another part of the mass. The scale of tectonic movement can vary from a few millimetres (as in the microfolds of a schist) to tens of kilometres (as in major recumbent folds)

3.3 Terms used in section 3

Trenches, pits and shafts are classified as shallow, medium and deep. The limit of 1.5 m for shallow trenches covers maximum requirements in most schemes for service pipes, cables, ducts or strip foundations, and in addition represents the maximum practicable depth for single throw handwork. The upper limit of 6.0 m for medium trenches corresponds roughly to the present practicable limit of excavation with backacting trenchers or trenchers of the continuous bucket type (see **A.1**).

The depths suggested for the three types of pit (shallow, medium or deep) cannot be applied as rigidly as with trenches, since methods of construction suitable for medium pits may in some cases have to be applied to shallow pits, depending largely on local soil conditions.

Medium and deep excavations of relatively small superficial area include shafts to give access to headings and deep trial holes, but exclude such special problems as deep wells and shafts to underground mineworkings.

3.3.1
trench

an excavation whose length greatly exceeds its width and which has either vertical sides capable of being supported by strutting from side to side or battered sides requiring no support

3.3.2
shallow trench

a trench up to 1.5 m in depth, as used for service pipes, cables, ground beams and strip foundations

3.3.3
medium trench

a trench between 1.5 m and 6.0 m in depth, as used for pipelines and sewers

3.3.4
deep trench

a trench exceeding 6.0 m in depth, for all classes of work

3.3.5
narrow trenches

a class of excavation too narrow to allow the entry of workmen is encountered in narrow trenches of shallow and medium depth. Such trenches are used for cables, small pipes, trench fill foundations and land drains, and require a separate technique for excavation (see **14.9.1**)

3.3.6
pit

for the purposes of this code a pit may be considered as an excavation ranging from that required to receive the foundation base for a pier or column to that required to receive the basement and foundations of a building. Trial pits excavated for site investigation purposes are also included

3.3.7
shallow pit

a pit up to 1.5 m in depth

3.3.8
medium pit

a pit between 1.5 m and 6.0 m in depth

3.3.9
deep pit

a pit exceeding 6.0 m in depth

3.3.10
shaft

an excavation, which may be either vertical or inclined, constructed to give access to underground works. Shallow, medium and deep shafts are defined in the same way as shallow, medium and deep pits

3.4 Terms used in timbering

The following definitions are based on the use of timber as a constructional material. It may, however, be found desirable to used reinforced concrete or steel sections and the definitions should be read accordingly.

3.4.1
back prop

a raking strut used to transfer the weight of timber to the ground in deep trenches; usually placed below every second or third frame

3.4.2
biat (byatt)

a timber bearer giving support to guard rails, decking, walkways, etc. (see Figure 23)

3.4.3
bitch

a fastening of iron or steel used for securing heavy timbers which cross each other. It is similar to a dog but has one of its ends at right angles to the other

3.4.4
blocking, chock or chog

a timber block used as a distance piece or packing, e.g. between a waling and the temporary or permanent lining of the excavation, to permit the insertion and erection of vertical reinforcement in retaining walls or other permanent construction (see Figure 29)

3.4.5
blow or boil

a displacement of sand, silt or gravel in the bottom of an excavation by upward flow of water

3.4.6
bracing

a diagonal member used to stiffen or brace the timbering (see Figure 32)

3.4.7
brob or nail spike

a fastening of iron with its head bent at right angles to the shaft

3.4.8
cap, capping piece or distributor

a piece of timber placed over the joint where two walings butt to take the thrust of the strut

3.4.9
chock or chog

see blocking

3.4.10
cleat

a block of timber fixed to a member to prevent the movement of other abutting timbers

3.4.11
close sheeting or close timbering

vertical or horizontal boards placed in close formation to hold up the face of an excavation (see Figure 23 etc.)

3.4.12
cross poling

short lengths of poling boards placed horizontally across a gap between runners or sheeting and tucked in behind them and used where runners or sheeting cannot be driven continuously and vertically (see Figure 26)

3.4.13
cutting-out piece

a short piece of timber which may be cut out to facilitate the striking of timbering

3.4.14
distributor

see cap

3.4.15
dog

a fastening of iron used for spiking large timbers together and having both ends bent down and pointed

3.4.16
dumpling

the ground temporarily left in the middle of an excavation which may serve as an abutment for the timbering to the surrounding trenches

3.4.17
face waling or face-piece

a waling across the end of a trench supported by the ends of the side walings and which, together with the end strut also acting as a waling, supports the end face of a trench

3.4.18
folding wedges

wedges used in pairs, overlapping each other and driven in opposite directions in order to hold or force apart two parallel surfaces (see Figure 29)

3.4.19
foot block

a timber pad used to spread a load from a ground prop (see Figure 23)

3.4.20
formation

the finish level of the excavation at the bottom of a trench or heading prepared to receive the permanent work

3.4.21
frame

in a trench, any pair of walings on opposite sides of the trench together with the struts that separate them. In a shaft, all the walings and struts at the same level

the word "frame" is often regarded as including the setting of poling boards supported by these timbers

3.4.22
normal poling frame

a frame in which the walings support the poling boards at their mid-points (see Figure 23)

3.4.23
tucking frame

a frame in which the walings support the poling boards at their ends (see Figure 24)

3.4.24
ground or top frame

a timber frame of walings and struts laid about 0.3 m below ground level, used as a guide for the first "setting" of runners or trench sheeting

3.4.25
guide frame

a timber frame erected above ground level to act as a guide for runners or trench sheeting, and as a staging from which they may be driven

3.4.26
ground prop

a prop or puncheon placed between the lowest frame and a foot block on the bottom surface of an excavation and used to support the dead weight of the timbering (see Figure 23)

3.4.27
guide runner

a runner driven ahead to form a guide for driving intermediate runners

3.4.28
hanger

see tie rod

3.4.29
heading

excavation in tunnel

3.4.30
interlocking pile

see steel sheet piling

3.4.31
kicking piece

a length of timber spiked to a waling to take the thrust from the end of a strut which is not at right angles to the waling

3.4.32
king piles

a line of piles to the full depth of the excavation, driven at strut intervals in the body of a wide trench before excavation, serving as supports and abutments for struts shorter than would otherwise be required

3.4.33
lacing

a vertical timber spiked to the sides of struts or walings and tying them together to carry the weight of the lower frames as excavation advances (see Figure 23 and Figure 32)

3.4.34
liner or stretcher

a timber driven between the ends of opposing members of a frame to lock them in position and spiked to the member against which it rests [see Figure 33(a)]

3.4.35
lip, lipping block, lipping piece

a short length of timber fixed and spiked to the top of a strut, and projecting sufficiently beyond its end so as to rest on a waling. It supports the weight of the strut while wedges are being driven (see Figure 23)

3.4.36
open sheeting

used in excavation in which the sides are reasonably firm and not likely to crumble. Generally consists of vertical poling boards spaced at intervals and supported by walings and struts, or horizontal sheeting openly spaced and held in position by soldiers and struts

3.4.37
page

a small timber wedge (see Figure 26)

3.4.38
poling back

the operation of excavating behind timber supports already in position and timbering the new face

3.4.39
poling boards

timber boards generally from 1 m to 1.5 m long and from about 25 mm to 50 mm thick, used in supporting the faces of an excavation (see Figure 23)

3.4.40
puncheon or prop

a vertical prop to support a higher waling or strut from the one below (see Figure 23)

3.4.41
runners

long vertical timbers at least 50 mm thick with their lower end chisel-shaped; used in unstable ground instead of poling boards, and driven downwards in advance of the excavation (see Figure 26)

3.4.42
scantling

a term used to denote the breadth and thickness of a piece of timber

3.4.43
setting

all boards held in position by one frame of timber, or in the case of tucking or piling frames, by two adjacent frames

3.4.44
sheeting

boards, planks or timbers used in conjunction with walings or soldiers and struts to support the sides of an excavation

3.4.45
sheet piling

vertical members of timber, reinforced concrete or steel driven into the soil in a row to retain soil during excavation and to assist in the exclusion of water

3.4.46
shoe

a steel or iron attachment suitably shaped to fit and reinforce the cutting edge of a runner or sheet pile

3.4.47
soldier

a vertical timber or steel taking the thrust from horizontal sheeting or walings and supported by struts across an excavation (see Figure 25)

3.4.48
staging

a working platform supported on the main framing of trenches

3.4.49
steel sheet piling

sheet piling formed of rolled steel sections with interlocking joints and used principally for excavations in difficult or water-bearing soils (see Figure 27 and Figure 28)

3.4.50
stretcher

see liner

3.4.51
strut

a horizontal member in compression, resisting lateral thrusts from the sides of an excavation (see Figure 23)

3.4.52
sub-drains

open-jointed or perforated pipes laid in the trench at the bottom of excavations to drain the ground as the work proceeds

3.4.53
sump

an excavation below the level of the bottom of a trench into which water drains and from which it may be baled or pumped

3.4.54
swinger

a pointed iron bar about 1 m long, used as a lever for moving runners

3.4.55
tie rod or bolt

a steel rod or bolt sometimes used instead of lacings between successive frames to take their weight and prevent movement of the timber

3.4.56
tucking board

a narrow timber used behind walings in tucking frames (see Figure 24)

3.4.57
tucking frame

see frame

3.4.58
waling

a horizontal member supporting the poling boards or sheeting in an excavation (see Figure 23)

3.4.59
wedge

see folding wedge and page

3.4.60
weephole

in wet ground a hole sometimes provided through sheeting to allow the discharge of water so as to prevent the development of a dangerous hydrostatic head of water

3.4.61
yankee brob

a Z-shaped metal strap

4 Safety procedures

4.1 General

Persons responsible for the safe operation of any construction works should not only be conversant with all the relevant legislation, some of which is listed in **4.2**, but should also actively encourage a safe approach to the work in hand. The principal causes of accidents, which apply at any stage of the works, can be categorized under eight main headings:

a) falls of persons into the works;

b) falls of materials (e.g. collapse of the sides of excavations, fall of material into the excavation from spoil tips, etc.);

c) unintentional collapse of the whole or any part of a structure and of structures erected as temporary works (includes the effect of excavations alongside existing buildings and inadequate timbering of excavations);

d) lifting operations associated with the works and unfenced machinery;

e) fires and explosions;

f) electrical, including damage to underground cables and other electrical services;

g) trespass by the public (including children) onto construction sites;

h) miscellaneous accidents due to lifting and carrying equipment and materials, poor means of access for vehicles and operatives into excavations, collisions with obstructions such as projecting reinforcement, mis-placed or inadequate barriers and randomly stacked materials, and reversing accidents or other collisions involving earthmoving equipment and transport.

Unsafe places of work contribute to falls of persons and materials. There is a risk to persons arising from the accidental collapse of the sides of excavations due to insufficient attention being paid to the temporary supports.

The wearing of suitable protective clothing and equipment for the job is a major aid to safety. All persons who work on construction sites should be encouraged to wear safety clothing which includes helmets, boots, shoes and gloves. Injuries to the head, hands and feet are frequent but equally, serious accidents can cause painful damage to eyes. Attention should therefore be paid to laws which require safety goggles or screens for certain types of work, and where necessary, breathing apparatus.

All excavations should be examined daily by a competent person to ensure that it is safe for operatives to work within them. Where the sides cannot be sloped back to a safe angle, as approved by a person competent and experienced in such matters, their continued stability should not be taken for granted. Where the depth of excavation is greater than 1.2 m, the *omission* of supports to a vertical or steep face should be a matter of positive instruction rather than acceptance without instruction.

It should not be assumed that vertical unsupported sides may be safely constructed in rock strata, since the orientation of the geological planes of separation and any soft materials contained in them have to be taken into consideration.

Timbermen erecting the supports should be in a protected position. Consideration should be given to adjustable frames or other devices which can provide support whilst other timbering is erected. Persons should only work within the supported section of an excavation and should not enter unsupported parts. Notices should be posted to this effect.

Operatives engaged on bottoming-up in an excavation should not be required to work near the excavator.

Proper means of access to and from excavations should be provided. Ladders should be secured to prevent slipping. They can be simply tied to the timbering or the top of the ladder can be secured by tying it to a small picket driven into the ground. Operatives should not walk or clamber about the excavation supports since this could weaken them.

Excavated spoil or other materials or plant items should not be placed close to the edges of excavations unless allowed for in the design of the works, and unless the operatives working within the excavation are protected from falling spoil or other objects.

Support works for trench excavations should be designed and constructed to ensure the stability of the surrounding ground and the safety of the operatives and the general public during the construction period. The loads imposed by stationary and mobile plant and construction materials should be taken into account in the design of the support works. Steps should be taken to prevent mobile plant falling into excavations by the use of stop blocks installed with their anchorage points well away from the excavation.

The use of explosives in excavation work should be carried out in accordance with the recommendations contained in BS 5607. See further **10.7**.

4.2 Statutory obligations

Attention is drawn to the requirements of the Factories Act 1961 and to the Regulations made under the Act which apply to construction work and which may affect any of the operations involved in earthworks.

Earthworks associated with the construction of a building or a defined engineering structure constitute a "Building Operation" (BO) or a "Work Engineering Construction" (WEC) by virtue of the definitions in Section 176 of the Act and the associated Extension of Definitions Regulations.

Section 127 of the Act applies a number of its provisions to BOs and WECs and therefore to associated earthworks.

The main Codes of Regulations made under the Factories Act which are directly relevant to earthworks are the four construction codes which apply to all BOs and WECs, i.e.

the Construction (General Provisions) Regulations, 1961;

the Construction (Lifting Operations) Regulations, 1961;

the Construction (Working Places) Regulations, 1966;

the Construction (Health and Welfare) Regulations, 1966.

Some of the other Codes of Regulations which apply to BOs and WECs as well as to conventional factories are:

the Electricity (Factories Act) Special Regulations, 1908 and 1944;

the Woodworking Machinery Regulations, 1922 to 1945;

the Work in Compressed Air Special Regulations, 1958;

the Asbestos Regulations, 1969;

the Ionizing Radiations (Sealed Sources) Regulations, 1969;

the Ionizing Radiations (Unsealed Radioactive Substances) Regulations, 1968.

These are some of the Statutory Regulations which, together with any subsequent amendments, have to be observed.

The Health and Safety at Work Act introduced in July 1974 came into force at the beginning of April 1975. The Act extends the obligations and protection to include all persons at work and members of the public. These obligations are in addition to the duties of employers and others under existing health and safety legislation.

Reference should also be made to the *Guide to the Construction Regulations* published by the Federation of Civil Engineering Contractors and the National Federation of Building Trades Employers; to *Construction Safety published* by the NFBTE; to the *Building and Construction Regulations Handbook* published by the Royal Society for the Prevention of Accidents; and to a Health and Safety at Work series of booklets issued by the Department of Employment entitled *Safety in Construction Work* (now the responsibility of the Health and Safety Executive).

4.3 Precautions against incidental hazards

Expert advice or help should be sought before proceeding with the work if any of the following sources of danger are found or expected. If the work is being carried out in an existing industrial plant the management should be consulted.

a) *Noxious gases*, e.g. carbon monoxide, carbon dioxide, sulphur compounds, air lacking oxygen, town gas or methane: consult HM Inspector of Factories, HM Inspector of Mines, the National Coal Board, the Public Analyst, the local Gas Board or the Medical Officer (environmental health).

b) *Noxious or flammable liquids*, e.g. petrol, alcohol, cleansing chemicals, white spirit, acids or alkalis: consult HM Inspector of Factories or the Medical Officer (environmental health).

c) *Sewage:* consult the regional Water Authority.

d) *Unexploded missiles:* consult the police.

Section 2. Cuttings and embankments, grading and levelling

5 Site conditions and investigations

5.1 General

5.1.1 *Environmental considerations*

5.1.1.1 *Stability of site.* Several factors may affect the stability of a site proposed for earthworks. The site may be unstable in its natural state. The construction of a cutting or embankment may result in instability of underlying or surrounding terrain which was previously in a stable condition. The possibility of buried channels, former watercourses and ponds infilled with compressible material should be investigated.

The natural flow of surface and subsoil water across sloping terrain may be obstructed or diverted by earthworks, leading to changes in water levels or to impounding of flood water by embankments. The effects of these changes in the natural drainage patterns and ground water levels on existing works in the locality should also be considered. Diversion of natural surface or ground water flow towards underground cavities as a result of excavations may lead to sub-surface erosion and collapse of natural arching over these voids.

5.1.1.2 *Pollution.* Precautions may be necessary against pollution of lakes or natural watercourses caused by run-off or pumping from earthworks in made ground or in fine-grained soils such as chalk or marl. If necessary, ditches or piped drains should be provided to intercept run-off water and convey it to lagoons for the settlement of suspended solids before discharge into a watercourse. Chemical dosing may be needed in lagoons or settling tanks to flocculate fine suspended matter or to neutralize acidic waters.

At the construction stage of an earth-moving project airborne pollution by dust, deposition of mud or loose debris on public roads and pollution by noise are factors requiring attention and consultation with local authorities (see **8.6.5** and **8.8**).

5.1.2 *Preliminary assessment of the project*

5.1.2.1 *Assembly of existing data.* Sources of preliminary information are listed in BS 5930. This information includes geology, borehole logs, site investigation reports, previous history of the site and the location of old maps and aerial photographs [3]. Collection of preliminary data is important since it enables a site investigation to be properly planned. If extensive information is available the normal scope of a site investigation may be reduced.

If the initial investigation indicates that the site may be affected by past, present or future underground mining operations, advice should be sought from a mining engineer or surveyor with the appropriate specialized knowledge of the district.

5.1.2.2 *Site reconnaissance.* The procedure for site reconnaissance is described in BS 5930. Much useful information can often be obtained by local enquiries. This reconnaissance is an essential part of the preliminary assessment of a project and no major earthworks should be undertaken without completing this stage of the investigatory work in a thorough manner. Evidence of past or present landslides, creep or subsidence should be sought by observing landforms and studying aerial photographs. A thorough examination should be made of existing drainage patterns which could be affected by the proposed works.

Useful information can be obtained by examining the geological structure and stability conditions in existing road or railway cuttings where the geological formation is similar to that in the area of the project. Where the geology of the site is of significance to the earthworks, it is good practice to summarize all relevant information on a geological map drawn to a suitable scale [4].

5.1.2.3 *Preparation of plan for a site investigation.* The results of the study of available information and of the site reconnaissance, when considered in relation to tentative plans and sections of the earthworks, enable an assessment to be made of the amount of information required from a detailed investigation. A plan should then be prepared for exploring the ground profile to the required depths and lateral extent and for field and laboratory tests on soils and rocks to provide data on ground properties for designing and constructing the earthworks.

The plan should be flexible in its scope and capable of being implemented stage by stage if at all possible. However good the data on which the original plan was based, it is not unlikely that the ground profile, as revealed by the detailed exploration, differs from that anticipated.

The engineer responsible for directing the investigation is to be kept in close touch with the field work and he should adjust the plan from time to time as may be required. Where uniformity in ground conditions is revealed he may take action to reduce the scope of the work; conversely, where very variable ground conditions are disclosed he may need to increase it. The aim should be to avoid the need for further investigations to make good deficiencies in the data provided by the main exploration work.

5.1.3 ***Ground exploration***

5.1.3.1 *General*. The procedure for ground exploration by means of trial pits and trenches, shafts, boreholes and geophysical surveying is described in BS 5930. **5.1.3.2** to **5.1.3.7** give guidance on the relevance of the various exploration techniques to the design and construction of earthworks.

5.1.3.2 *Trial pits and trenches*. Pits and trenches excavated by hand or mechanically are a rapid and economic means of obtaining detailed information on depths of up to 6 m. They are suitable for exploration in areas of shallow cut and fill and for tracing the thickness and lateral extent of superficial deposits of soft marshy soils or fill. Trial pits can be used to examine the foundation conditions for light structures associated with the earthworks and for assessing ground water and support problems for the construction of drain and service trenches. They are also helpful at the planning stage of earthworks, especially in material where the geological structure, as assessed visually, is important to the selection of a procedure for excavation.

Trial pits or trenches are a useful means of exploring, logging and sampling landslides, solifluction or other colluvial deposits of moderate depth, and any associated shear surfaces. In situ tests such as large shear box tests may be made on the shear surfaces exposed in the trenches to provide information on shear strengths in connection with the design of remedial works or to assess the possible detrimental effects of the new construction. Alternatively, undisturbed block samples containing the shear surfaces may be taken for laboratory testing.

5.1.3.3 *Shafts*. Where deep exploration is planned, say for major cuttings, boreholes may not reveal the geological structure in sufficient detail to assess problems of stability. Deep shafts, either sunk by hand excavation with timbered supports or drilled mechanically with the sides supported by steel casings, can be used to provide a means of visual inspection of soil or rock conditions. In situ testing such as vertical or horizontal plate loading tests can be made at the base of the shafts or in headings driven from them. The experiences in sinking shafts and particularly in the rates of pumping ground water provide useful information for the assessment of ground water lowering or ground treatment problems in deep excavations.

Methods of sinking and supporting the sides of shafts are described in clause **15**.

5.1.3.4 *Boreholes*. Augering or cable percussion methods are used for sinking boreholes in soils and very weak rocks as described in BS 5930. The diameters of the boreholes should be sufficient to enable undisturbed soil samples 100 mm in diameter to be recovered.

Where soils exhibit macro-fabric such as fissures or laminations, large diameter samples, say 200 mm or 250 mm in diameter, may be justified to enable special laboratory tests on such samples to be made [5].

Explorations in rocks should be undertaken by rotary drilling to recover core samples as described in BS 5930. The borehole diameter should be such as to obtain complete recovery of the rock cores including all very weak and friable material. The cores should be extracted, labelled, boxed for transport and stored in accordance with the recommendations of BS 5930.

Boreholes should be drilled to a level below the base of proposed cuttings to a depth sufficient to explore all zones of potential instability (see **6.3** and **6.4**). Boreholes located on the sites of embankments should be drilled to a depth not less than 1.5 times the base width of the embankment, unless a layer of strong and relatively incompressible soil or rock is located and proved to be of adequate thickness and continuity at a lesser depth.

Boreholes should be located within the area of cuttings and embankments, but where necessary they should also be located outside the confines of these earthworks to investigate all zones influenced by them.

Deep excavations may involve a risk of uplift of the base due to shear failure in soft to firm clays or to the occurrence of ground water under pressure in pervious layers underlying impervious soils or rocks at the base of the excavations; the possibility of heave of the floor of the excavation due to swelling of clay soils may also need to be considered. Boreholes should be taken to a sufficient depth to investigate the presence or absence of such hazards.

Where ground water lowering schemes for excavations, or installations to cut-off inflow into excavations, are proposed, the boreholes should be drilled completely through the water-bearing formation to locate and prove the thickness and continuity of an underlying impervious horizon.

The extent of variation in the level of the ground water is of considerable importance in analysing most slope stability problems, but reliable information on ground water conditions cannot usually be obtained while drilling boreholes or in the short intervals of time while drilling operations are suspended. Simple standpipes or piezometers should be installed in selected boreholes for periodic ground water observations. These observations should be made daily while site investigations are in progress and thereafter at weekly or monthly intervals for as long a period as practicable before the execution of the earthworks, preferably for at least a year in order to observe the seasonal ground water variations. When ground water is encountered in distinct aquifers separated from each other by impervious layers, the ground water pressures should be monitored by individual piezometers installed in each aquifer and sealed from contact with the over or underlying formation by bentonite slurry or bentonite-cement grout. Field permeability tests can be made in boreholes to assess the rate of ground water flow from the sides of cuttings, or the pumping rates from deep excavations below the water table. Ground water levels should be related to rainfall data.

5.1.3.5 *Detection of underground movements at depth*. The use of trial pits and trenches to determine the location of existing shear surfaces at relatively shallow depths is referred to in **5.1.3.2**. This method becomes increasingly expensive with depth and where there is reason to believe that movement is actually taking place or it is feared that movement is imminent, slip indicators or inclinometers installed in boreholes can be used [6]. The simplest form is a slip indicator, which is best suited to situations where movement is relatively rapid. It has the disadvantage of being unable to detect small movements and it cannot quantify large ones.

The inclinometer, unlike the simple slip indicator, can show the precise direction in which the soil has sheared between any two occasions of measurement. The equipment is suitable for cases where the rate of movement is small or for installation in anticipation of movement. If used when movement is relatively rapid the inclinometer probe is soon unable to pass the shear surface and hence all measurement of movement below this level ceases to be possible. The efficient filling of the annular space between ground and the special casing is most important. Grouting the space is the most positive method. If this is not done properly and the casing is free to flex within the bore the inclinometer will record erratic movements up, down and parallel to the contours, which can be disconcerting until the cause is realized. Apart from measuring the location of shear surfaces, the inclinometer is much used for the general measurement of horizontal movements below the surface of the ground.

5.1.3.6 *Geophysical methods*. Geophysical methods can be used to plot the configuration of soil or rock strata where successive formations have significant differences in their geophysical properties, such as seismic velocity or electrical resistivity. A description of the available methods is given in BS 5930. Geophysical surveys can be executed rapidly and can be an economical method of sub-surface exploration for large-scale earthworks, provided that the ground conditions are favourable, that suitable techniques are employed and that there is calibration and correlation of the survey data by an adequate number of boreholes. Geophysical methods can be used to locate the buried surface of relatively unweathered rock overlain by drift or weathered rocks.

Geophysical logging equipment can be lowered down boreholes to obtain a continuous record of seismic velocity, electrical resistivity, density, moisture content and mineral composition of soils and rocks, from which other relevant physical properties can be deduced. Borehole logging can be a useful means of detailed soil exploration in borrow pits. One instrument based on seismic principles counts the number of signals of sub-audible frequency per unit time generated at the zone of shearing in a rock mass. The rate of emission is an indicator of the rate of movement and hence it gives a warning of the need for any urgent remedial measures in a potentially unstable slope. The signals can be used to locate the shear zone.

Geophysical surveys can also be used in favourable conditions to locate and map cavities below the ground surface caused by mineworkings or solution of rocks.

5.1.3.7 *Safety aspects*. Attention should be given to ensuring safe working conditions while recording observations or making field tests in trial pits, trenches or shafts. Recommendations for safe working practices are made in clauses **4**, **14** and **15**.

5.1.4 *Description and classification of soils and rocks*. A comprehensive system for the description of soils and rocks is given in BS 5930. The classification system and the means of identifying the principal classes of soil are shown in Table 1.

It is most important that such systems should be adhered to strictly for recording ground conditions in boreholes, trial pits, trenches and shafts, and for describing soil and rock specimens on which tests are made. This avoids ambiguity and uncertainty in the interpretation of reports and records at the stages of design, and in subsequent preparation of tenders for execution of the earthworks.

Classification systems are widely used for earthworks design in highway and airfield construction. Empirical relationships have been established between classification groups and factors such as pavement thickness, frost susceptibility, shrinkage and swelling, drainage characteristics and compaction methods. Some of these factors are set out in Table 2 and Table 4. Detailed descriptions should be given of made ground including information on texture, composition and relative density. Whenever possible descriptions of materials in made ground should be in terms of standard soil or rock classification.

It is also important to identify the geological origin of the soils and rocks encountered in the boreholes and trial pits. The stratigraphical formation of the various soil and rock layers should be stated on the borehole records following the practice recommended in BS 5930.

5.1.5 *Field and laboratory testing*. The various field tests for soils and rocks are described in BS 5930. In formulating a programme for field and laboratory tests it is necessary to consider their relevance to the earthworks project and to carry out only those tests which apply to the design and construction problems. The clauses of this code which follow include some guidance on the application of various tests. Where doubt exists on which particular test should be undertaken, that which most closely simulates the anticipated field conditions should be adopted. Where certain parameters can be measured by either field or laboratory methods, it is necessary to consider whether or not the more reliable data which generally result from field tests will justify their additional expense, having regard to the extent of the works being planned and the consequences of failure. Similarly it is necessary to consider the economic justification for the more sophisticated forms of laboratory testing. The relevant sections of BS 5930 give guidance on these points.

Estimates of shear strength, drainage and consolidation characteristics in the mass should wherever appropriate be made by back analysis of field behaviour, particularly where there has been previous instability of a site.

5.1.6 *Report on investigations*. Guidance on the procedure for preparing reports on investigations is given in BS 5930.

5.1.7 *Further investigation during construction*. It should be appreciated that no site investigation, however carefully done, ever examines more than a very small proportion of the ground. It is necessary to check that the soil conditions revealed during progress of the excavations correspond with those forming the basis for earthworks design as interpreted from the site investigation. It may be necessary to undertake further investigation to determine the extent of anomalous conditions. Guidance is given in BS 5030.

5.2 Economic and environmental considerations

5.2.1 *General*. The design of earthworks should take account of their effect on the general landscape and, wherever possible, should avoid disfiguring scars on hillsides or notches in the crests of slopes. The slopes of cuttings and embankments should be varied to blend in with the contours of the adjacent ground, avoiding straight lines and abrupt changes in profile. If slopes of 1 in 5 or flatter are adopted the land can be used for agriculture.

Earthworks may need to be shaped, and fencing designed and sited, to avoid or minimize accumulations of drifting snow. In severe situations, site trials and wind tunnel studies should be made to obtain the optimum profile. It has been found that a slope of 1 in 5 for the top 5 m of an embankment reduces snow accumulation.

A balance of cut and fill quantities within the boundaries of the site is often desirable both economically and as a means of limiting interference with the surrounding areas. Such balances are rarely achieved and the possibility of having to obtain further quantities of suitable material, as well as the disposal of surplus or unsuitable soils, may have to be considered.

The information obtained from ground exploration is necessary for assessing the suitability of the available soils for inclusion within the works. The effect of climatic conditions on the materials during the currency of the works should be considered. Where sites contain large proportions of only marginally acceptable materials, consideration should be given to obtaining less susceptible suitable materials from borrow pits or importing "all weather" materials to alleviate the risk of delay and its financial consequences. Borrow pits should be as close to the project as possible so that plant used on site is able to excavate the borrow without leaving the site and that noise, dust, traffic congestion, mud on roads and damage to roads in the area is kept to a minimum. Tips for the disposal of surplus and unsuitable materials should also be kept as close as possible to the project; it is often feasible to use them for landscaping. Care should be taken in siting spoil tips to avoid local or general slope instability. Careful contouring can often improve the effect of the project on the adjacent landscape, and derelict areas can be brought back into use. These benefits can only be realized if the earthworks are considered as a whole and the designer is able to use all arisings to the best overall effect and not just within the confines of the site.

It is possible to operate earth-moving and compaction plant more continuously and effectively on fills consisting of rock or well graded granular materials than on clays. The latter, however, are affected longer by wet weather, while the former become difficult to use in arid conditions. But these materials may also be suitable for the production of concrete aggregates and for pavement construction, so their use as general filling has to be carefully considered and balanced against the need at a later date to import processed materials.

In order to minimize disturbance to the environment, earth-moving plant and transport should be kept off the public highways unless there is no reasonable alternative. Temporary disturbance can, however, sometimes be accepted if earthworks can be shown in the long term to be providing an overall benefit to the community, such as the removal of an industrial spoil heap for incorporation within the works.

Certain by-products from industrial processes may be suitable for fill (see **7.2.1.6**).

The effect of quarrying good material for fill from local sites should be balanced against the advantage in importing cheaper material over long distances. Although transport costs are often very important, in any economic analysis social benefits and energy savings should also be taken into account [7, 8, 9].

5.2.2 *Cut and fill for roads and railways*. Earth-moving for roads and railways takes place over a relatively narrow band of terrain and a balanced cut and fill is achieved at a preliminary stage by examination of a longitudinal profile. However, constraints to the profile are frequently imposed by the need to provide statutory clearances for bridges under or over existing roads and railways, or to cross them at their existing grades. Computer programs are available for examining the effects on earthwork quantities of adjustment to profiles both longitudinally and in cross section, and computer-controlled plotters can be used to produce perspective drawings of the earthworks as designed to assess their appearance in relation to the environment [10].

5.2.3 *Cut and fill for airfields and other extensive paved areas*. Earthwork contours for airfields are governed by the need to establish profiles for runways and marginal strips to the standards laid down by military or civil aviation authorities [11]. Run-off of surface water from paved surfaces and graded marginal strips on runways can be very large. Therefore care should be taken in contouring "dead" areas between strips or generally in other parts of the airfield, so that the run-off is directed to low-lying areas acting as balancing reservoirs suitably located to receive and store run-off water before discharge to outfall drains.

5.2.4 *Dredging and hydraulic fill*. Although dredging and reclamation in over-water areas are outside the scope of this code, consideration should be given to the economics of excavating for works on land by dredger and disposing of the soil by pumping (hydraulic fill). The use of a dredger can be economical for excavating in pervious solids below ground water level, provided that there is access for the dredger to the working area (see **7.6.3.4**).

5.2.5 *Sand dunes*. The movement of dry sand and the formation of dunes and hollows is related mainly to wind action and is a function also of grain size and density. In addition to the physical characteristics of the sand particles, the wind rose at the locality for a yearly period or greater should be established using data from a local meteorological station or, in their absence, by setting up an automatic recording anemometer. Topography, structures, fences and the slat configuration of fences cause local alteration of the general pattern of drifting and dune formation. Similarly, types of vegetation which thrive in this habitat can progressively affect dune formation as sand is deposited around the continuously growing plants.

Table 1 — British soil classification system for engineering purposes and field identification

First remove material coarser than 60 mm and record as cobbles (60 mm to 200 mm) or boulders (over 200 mm).

Soil groups[a]		Sub-groups and laboratory identification						Field identification
		Description	Group symbol	Sub-group symbol	Fines % less than 0.06 mm	Sub-group name	Casagrande group symbol	
COARSE SOILS less than 35 % finer than 0.06 mm	GRAVELS More than 50 % of coarse material is of gravel size (coarser than 2 mm)	Slightly silty or clayey GRAVEL	G GW GP	GW GP GPu GPg	0–5	Well graded GRAVEL Poorly/uniformly/ gap graded GRAVEL	GW GP/ GU	Particles easily visible to naked eye. Particle shape and grading can be described.
		Silty GRAVEL Clayey GRAVEL	G-F G-M G-C	GWM GPM GWC GPC	5-15	Well/poorly graded, silty (clayey) GRAVEL	GC/GF	Particles easily visible to naked eye. Particle shape and grading can be described. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates the absence of clay.
		Very silty GRAVEL Very clayey GRAVEL	GF GM GC	GM GCL GCI GCH GCV	15-35	Very silty GRAVEL Sub-divide like GC Very clayey GRAVEL (clay of low/ intermediate/high/very high plasticity)	GF GF	
	SANDS More than 50 % of coarse material is of sand size (finer than 2 mm)	Slightly silty or clayey SAND	S SW SP	SW SP SPu SPg	0-5	Well graded SAND Poorly/uniformly/gap graded SAND	SW SP/SU	Majority of the particles visible to naked eye. Feels gritty when rubbed between the fingers. A medium to high dry strength indicates that some clay is present. A negligible dry strength indicates the absence of clay.
		Silty SAND Clayey SAND	S-F S-M S-C	SWM SPM SWC SPC	5-15	Well/poorly graded silty (clayey) SAND	SC/ SF	
		Very silty SAND Very clayey SAND	SF SM SC	SM SCL SCI SCH SCV	15-35	Very silty SAND sub-divide like SC Very clayey SAND (clay of low/ intermediate/high/very high plasticity)	SF SF	

[a] Gravel and sand may be qualified as Sandy GRAVEL and Gravelly SAND where appropriate.

Table 1 — British soil classification system for engineering purposes and field identification

Soil groups[a]		Sub-groups and laboratory identification: Description	Group symbol	Sub-group symbol	Liquid limit %	Sub-group name	Casagrande group symbol	Field identification
FINE SOILS more than 35 % finer than 0.06 mm	Gravelly or sandy SILTS and CLAYS 35 %–65 % fines	Gravelly SILT	FG MG	MG		Gravelly SILT (sub-divide like CG)	—	Coarse particles visible to naked eye. Silt fraction dries moderately quickly and can be dusted off the fingers. Clay fraction can be rolled into threads when moist, smooth to touch and plastic, sticks to fingers and dries slowly.
		Gravelly CLAY	CG	CLG	35	Gravelly CLAY of low plasticity		
				CIG	35–50	Gravelly CLAY of intermediate plasticity		
				CHG	50–70	Gravelly CLAY of high plasticity		
				CVG	70–90	Gravelly CLAY of very high plasticity		
		Sandy SILT	FG MS	MS		Sandy SILT	ML	Sandy silts and sandy clays feel gritty when rubbed between the fingers. Silts and sandy silts dry quickly and can be dusted off the fingers, exhibit marked dilatancy. Dry lumps have some cohesion but can be powdered easily in the fingers.
		Sandy CLAY	CS	CLS etc.		Sandy CLAY: sub-divide like CG	MI	
	SILTS and CLAYS 65 %–100 % fines	SILT (M-SOIL)	F M	M		SILT: sub-divide like C	ML/MI	Clays, silty clays and sandy clays are plastic and can be readily rolled into threads when moist. Dry lumps can be broken but not powdered, but they disintegrate under water. They stick to fingers and dry slowly. Clays feel smooth to touch.
		CLAY	C	CL	35	CLAY of low plasticity	CL	
				CI	35–50	CLAY of intermediate plasticity	CI	
				CH	50–70	CLAY of high plasticity	CH	
				CV	70–90	CLAY of very high plasticity	—	
ORGANIC SOILS		Descriptive letter O suffixed to any group or sub-group symbol Organic matter suspected to be a significant constituent. Example MHO: Organic SILT of high plasticity						Usually dark in colour, plant remains may be visible, often with distinctive smell.
PEAT Pt		Peat soils consist predominantly of plant remains which may be fibrous or amorphous					Pt	Usually black or brown in colour. Very compressible. Easily identifiable visually.

[a] Gravel and sand may be qualified as Sandy GRAVEL and Gravelly SAND where appropriate.

Table 2 — Field characteristics of soils and other materials used in earthworks

Material	Major divisions	Sub-groups	BSCS[a] group symbol	Casagrande group symbol	Drainage characteristics[b]	Potential frost action	Shrinkage or swelling properties	Value as a road foundation when not subject to frost action	Bulk density before excavation		Coefficient of bulking
									Dry or moist	Submerged	
									mg/m^3	mg/m^3	%
Coarse soils and other materials	Boulders and cobbles	Boulder gravels	—	—	Good	None to very slight	Almost none	Good to excellent	—	—	—
	Other materials	Hard: Hard broken rock, hardcore, etc.	—	—	Excellent	None to slight	Almost none	Very good to excellent	—	—	20–60
		Soft: Chalk, soft rocks, rubble	—	—	Fair to practically impervious	Medium to high	Almost none to slight	Good to excellent	1.10–2.00	0.65–1.25	40
	Gravels and gravelly soils	Well graded gravel and gravel-sand mixtures, little or no fines	GW	GW	Excellent	None to very slight	Almost none	Excellent	1.90–2.10	1.15–1.30	10–20
		Well graded gravel sand mixtures with excellent clay binder	GWC	GC	Practically impervious	Medium	Very slight	Excellent	2.00–2.25	1.00–1.35	
		Uniform gravel with little or fines	GPu	GU	Excellent	None	Almost none	Good	1.60–1.80	1.00–1.11	
		Gap graded gravel and gravel-sand mixtures, little or no fines	GPg	GP	Excellent	None to very slight	Almost none	Good to excellent	1.60–2.00	0.90–1.25	
		Gravel with fines, silty gravel, clayey gravel, poorly graded gravel-sand-clay mixtures	GM/GC	GF	Fair to practically impervious	Slight to medium	Almost none to slight	Good to excellent	1.80–2.10	1.10–1.30	
	Sands and sandy soils	Well graded sands and gravelly sands, little or no fines	SW	SW	Excellent	None to very slight	Almost none	Excellent to good	1.80–2.10	1.05–1.25	5–15
		Well graded sand with excellent clay binder	SWC	SC	Practically impervious	Medium	Very slight	Excellent to good	1.90–2.10	1.15–1.30	
		Uniform sands with little or no fines	SPu	SU	Excellent	None to very slight	Almost none	Fair	1.65–1.85	1.00–1.15	
		Gap graded sands, little or no fines	SPg	SP	Excellent	None to very slight	Almost none	Fair to good	1.45–1.70	0.90–1.00	
		Sands with fines, silty sands, clayey sands, poorly graded sand-clay mixtures	SM/SC	SF	Fair to practically impervious	Slight to high	Almost none to medium	Fair to good	1.70–1.90	1.00–1.15	

[a] British Soil Classification System.

[b] Does not apply to in situ surface soils.

Table 2 — Field characteristics of soils and other materials used in earthworks

Material	Major divisions	Sub-groups	BSCS[a] group symbol	Casagrande group symbol	Drainage characteristics[b]	Potential frost action	Shrinkage or swelling properties	Value as a road foundation when not subject to frost action	Bulk density before excavation		Coefficient of bulking
									Dry or moist	Submerged	
									mg/m³	mg/m³	%
Fine soils	Soils having low compressibility	Silts (inorganic) and very fine sands, rock floor, silty or clayey fine sands with low plasticity	ML/SCL MS/CLS	ML	Fair to poor	Medium to very high	Slight to medium	Fair to poor	1.70–1.90	1.00–1.15	20–40
		Clay of low plasticity (inorganic)	CL	CL	Practically impervious	Medium to high	Medium	Fair to poor	1.60–1.80	1.00–1.11	
		Organic silts of low plasticity	MLO	OL	Poor	Medium to high	Medium to high	Poor	1.45–1.70	0.90–1.00	
	Soils having medium compressibility	Silt and sandy clays (inorganic) of intermediate plasticity	CIS	MI	Fair to poor	Medium	Medium to high	Fair to poor	1.55–1.80	0.95–1.11	—
		Clays (inorganic) of medium plasticity	CI	CI	Fair to practically impervious	Slight	High	Fair to poor	1.60–2.00	1.00–1.10	
		Organic clays of medium plasticity	CIO	OI	Fair to practically impervious	Slight	High	Poor	1.50	0.50	
	Soils having high compressibility	Micaceous or diatomaceous fine sandy and silty soils, elastic silts	—	MH	Poor	Medium to high	High	Poor	1.75	1.00	—
		Clays (inorganic) of high plasticity, fat clays	CH	CH	Practically impervious	Very slight	High	Poor to very poor	1.70	0.70	
		Organic clays of high plasticity	CHO	OH	Practically impervious	Very slight	High	Very poor	1.50	0.50	
Fibrous organic soils with very high compressibility		Peat and other highly organic swamp soils	Pt	Pt	Fair to poor	Slight	Very high	Extremely poor	1.40	0.40	—

[a] British Soil Classification System.
[b] Does not apply to in situ surface soils.

5.3 Risks of failure and acceptance of deformation

5.3.1 *General*. The risks of failure should be considered under two headings:

a) movement due to failure of the ground in shear;

b) unacceptable deformation before failure is reached.

The risks of failure in shear of the earthworks are assessed by calculating a safety factor which is defined as the ratio of the available shear strength of the soil to the strength required to maintain equilibrium. This approach is known as the "limit-equilibrium" method [12, 13]. The adequacy of the calculated safety factors is considered in relation to the consequences of failure. For example, in the case of cuttings a high safety factor is required where the results of a slip would endanger a main line railway or buildings. A relatively low safety factor would be acceptable for the slopes of an excavation for a foundation structure which is to be backfilled on completion of the below-ground work, provided that a slip would not cause danger to life or to any buildings in the vicinity. Similar considerations apply to safety factors for embankments.

When considering the deformations of earthworks, it should be appreciated that they themselves can sometimes undergo large deformations without detriment to their own serviceability, although the effect of such deformation on the shear strength may be sufficient to cause failure. In this respect, however, the important consideration is the effect of deformations on structures supported by or adjacent to the earthworks, and whether or not these are likely to be progressive.

5.3.2 *Effect on neighbouring structures*. Buildings close to the toes of embankments may be damaged by lateral soil deformation or heave. Excavation for road cuttings or foundation structures can cause vertical and horizontal deformation in the ground surrounding the excavation which may damage buildings or local services, including gas mains. Upward soil movements beneath a deep basement excavation may damage structures in tunnels at a considerable depth.

As in the case of stability considerations, the effects of deformation are time-dependent, possibly requiring many years before the full effects become manifest. It will usually be found that the critical factor is the serviceability limit state of structures supported by the earthworks or affected by them, rather than that of the earthworks themselves. In such cases the calculations to determine this state are made by conventional methods applicable to structures but based on data obtained from predicted ground deformations.

6 Cuttings

6.1 General considerations

6.1.1 *Environmental factors*. The finished level of a highway or railway may need to be kept low in a cutting so that the noise and visual impact from road and rail traffic is less of a nuisance. The cutting may be combined with a small embankment forming a noise baffle. Aesthetic and amenity considerations may require new transport routes to be in cuttings where existing roads and railways are crossed. Bridges would then cross at ground level without approach embankments. However, the construction of bridges under existing roads and railways is more costly and causes greater disruption to traffic than bridges over them. It may be necessary to construct roads and railways in cuttings adjacent to airports to avoid obstructing flight paths by the embankments associated with bridge crossings.

The slope angles should be considered in relation to the appearance of the cutting in its surroundings. It may be desirable to vary the slope angles or to adopt concave or convex curved shapes to harmonize with the adjacent contours and general landscape.

Forming a cutting in permeable ground having a high water table may alter the drainage conditions in nearby farming land with consequent loss of production. It is essential to record locations of streams and pools and any information of this type available in aerial photographs prior to commencement of work, in order to assist in dealing with subsequent claims for damage.

6.1.2 *Site geometry*. Where the nature of the project allows, the position and alignment of a cutting in plan and the geometry of the cutting in cross section should take the following factors into account:

potentially unstable ground in the vicinity

the stability of the cutting slope itself (see **6.2.2** and **6.3**)

wind effects, including snow drifting, sand deposition and scouring

maintenance

visual and other amenity aspects.

6.1.3 *Economic and safety considerations*. The minimum construction cost of a cutting is achieved by adopting the steepest possible slope angles. However, the economic and human consequences of failure of a cutting slope require careful consideration.

Restriction in the available width of land for a cutting may necessitate a steep slope, but where the consequences of failure would be serious it is necessary to reduce the slope angle by providing a retaining wall over part of the height. It is also necessary to provide sufficient width to accommodate a piped drain or ditch at the toe, and sufficient width at the toe or at intermediate berms to trap debris rolling down the slope (see **6.5.2** and **6.5.4**). The economics of providing additional space for debris traps should be compared with the, alternative of providing a barrier such as a rock fall fence.

As an alternative to a retaining wall, special methods of stabilizing a steeply sloping soil or rock face can be considered (see **6.5.5**, **6.5.6** and **11.4**).

The horizontal and vertical alignment of works such as roads or railways should be considered in relation to the comparative costs of soil and rock excavation. While the cost of excavating soil above ground water level is very much less than that of excavating rock, flatter slopes are required in a soil cutting (particularly in clays) and hence the total volume of excavation for a given depth is greater in a soil cutting. Also, maintaining slopes in a rock cutting normally costs less than maintaining soil slopes. Where cuttings are to be excavated in rock overlain by soil, the slope inclination can be varied to suit the characteristics of each material, with a berm at the change of slope. Similar slope changes can be adopted where soil types vary. For example, the slope can be steeper in a well drained granular soil overlying a stiff fissured clay, provided that the long term overall stability of the slope can be assured.

Economic factors are an important consideration in the design of temporary slopes for foundation excavations. The costs of additional excavation to obtain assurance of complete stability should be weighed against the cost of removing debris from local slips resulting from marginally unstable conditions. The consequences of such local slips for the safety of construction operatives, for possible damage to the partly-completed permanent work and for the existing building and services beyond the crest of the slope should be considered. Time effects are important in relation to the stability of temporary slopes where only short term conditions need be considered (see **6.2.5**).

6.1.4 *A guide for preliminary design of cuttings in rocks.* As an aid to preliminary design of stable slopes for rock cuttings, recommended slope angles for various materials are given in Table 3. The angles are derived from experience in railway cuttings, and are based on the assumption that the rock strata are horizontally bedded or sloping at a relatively shallow inclination and that the rocks are relatively unweathered behind the face.

6.2 Factors governing the stability of cutting slopes

6.2.1 *Materials: soils and rocks*

6.2.1.1 *General.* For the purpose of making a preliminary assessment of stability conditions and for guidance in formulating a field or laboratory testing programme, the broad soil classification systems described in **5.1.4** can be used to obtain an indication of the behaviour of a particular type of soil when excavated to form a cutting.

The parameters used to define shearing resistance should be obtained from back analysis or from appropriate field or laboratory tests which take account of the permeability of the mass of material and also of the stress changes which take place in the material, both in the short and long term, as a result of excavating for the cutting. These aspects are discussed in **6.2.2**.

Weak heavily weathered rocks may exhibit engineering characteristics intermediate between those of a soil and those of a rock. In cases of doubt separate analyses of slope stability should be made assuming that the material behaves either as a soil or as a rock.

6.2.1.2 *Behaviour of cohesionless soils (e.g. coarse soils).* In the case of a cutting slope in dry cohesionless soil, the most critical failure mode is that of a shallow translational slide parallel to the cutting slope. For this case, safety factor, F, is given by

$$F = \frac{\tan \phi'}{\tan \alpha} \qquad (1)$$

where

ϕ' is the angle of shearing resistance, in terms of effective stresses, for the soil at the appropriate range of effective normal stress (which may be quite low), and

α is the slope angle.

This safety factor is thus independent of the slope height.

When ground water is present within a slope of cohesionless material and if erosion by seepage is prevented, deeper-seated, rotational slipping is the most critical failure mode, and the corresponding safety factor is a function of α, ϕ' and the relevant value of the pore pressure ratio r_u [see equation (10)].

6.2.1.3 *Behaviour of cohesive soils (e.g. fine soils).* The stability of a cutting slope in a cohesive soil is a function both of the height of the slope and of the shearing resistance of the soil. The deeper the cutting, the flatter the slope angle. The geological history of the soil can be most important. In the case of intact *normally-consolidated clays*, i.e. clays which have gained strength only by consolidation under their own weight, the short term stability can be determined from the results of simple undrained shear strength tests. With time there is a progressive reduction in strength due to relief of stress consequent on excavation for the cutting, and the slope which is stable in the short term may fail by a rotational slide in the medium or long term (see **6.2.5.1**).

Some types of normally-consolidated clays are sensitive to disturbance, i.e. they lose strength when remoulded; this is because of an unstable structure caused by leaching of minerals under prolonged seepage or other weathering phenomena. Thus collapse of a slope may occur when disturbance is caused by earthquakes or vibrations induced by pile driving.

Over-consolidated clays, i.e. clays which have gained strength by consolidation under a heavy excess overburden pressure or by desiccation due to evaporation or the growth of vegetation, can cause difficult problems in slope stability. This is due to these clays being susceptible to the effects of stress changes caused by excavation for cuttings, which is enhanced if they have a fissured or laminated structure.

Boulder clays or glacial till may be fissured, but for slope design purposes they are usually assumed to act as intact clays. However, because of the significance of fissuring, evidence of this structure should be sought at the site investigation stage.

Fissures and laminations form planes of weakness in the soil mass. After excavating for a cutting the fissures open owing to relief of overburden pressure, with further seasonal opening at and near the surface owing to drying shrinkage of the clay. The open fissures form channels for water seepage resulting in a build-up of hydrostatic pressure behind the face at times of heavy rain and softening of the clay on the fissure surfaces.

Fissures in an over-consolidated clay are sometimes associated with ancient slips when their surfaces are smooth (slickensided) with a "residual" shear strength considerably lower than that of the adjacent intact material.

Instability of cutting slopes in over-consolidated clays can take several forms, such as a rotational shear slide (**6.3.2**), slab and block slides along the weathered softened surface of the slope (**6.3.4.2**), and debris slides (**6.3.4.4**) at the final stage when the slipped masses trap surface and subsoil water resulting in the formation of a liquid mass of slurried clay mixed with stiffer clay fragments. Toppling (**6.3.5**) and falls (**6.3.6**) can occur when a vertical cut is made, e.g. for a trench, consequent on vertical fissures opening behind the face.

Time effects are critical for slopes in over-consolidated clays (**6.2.5**).

Loess is a silt weakly cemented by a calcareous binder. It can stand vertically in a cutting face for very long periods of time. Instability can occur if the cementing medium is dissolved by water seepage.

6.2.1.4 *Behaviour of rocks*. The stability of a rock mass is governed by conditions in the joint system of the mass rather than by the strength of the intact rock. Fissures in rocks are caused by tectonic movements, glacial action or weathering. Thus the fissures, whether they are open or are filled with weathering products of the parent rock or with soil debris washed down from above, form surfaces of weakness which may give rise to the various forms of failure described in **6.3**. The joints are also a means of entry of water leading to the effects described in **6.2.4.2**.

Where rocks are weak and porous the action of water, sun and frost can cause disintegration of an intact or fissured rock (**6.2.5.2**).

The most unfavourable stability conditions occur when excavating for a cutting in a rock scree. This material may act as a loose cohesionless soil or as a cohesive soil, depending on the proportion and consistency of weathered rock or soil acting as a binder. Trial excavations are the best guide to determine stable slopes in the scree material and it may not be possible to form a slope steeper than the natural angle of repose of the least cohesionless material.

Table 3 — Design of slopes in rock cuttings and embankments *(by courtesy of the British Railways Board)*

Type of rock	**Cuttings: safe slopes** (angles referred to the horizontal)	**Embankments: angles of repose** (angles referred to the horizontal)	**Remarks**
Sandstone (sedimentary), strong, massive, of considerable geological age, e.g. Old Red Sandstone (Devonian); Blue Pennant Grit, Millstone Grit (Carboniferous); Bunter Sandstone (Triassic)	Mainly vertical but may in places require cutting back to 70°	38°–42°	Very resistant to weathering, although a block of stone may occasionally be loosened by frost action. Care should be taken where weak beds, e.g. shales, underlie the strong rocks, as the effects of differential weathering may lead to undermining; in such cases protection from weathering should be afforded to the weak beds.
Sandstone, weak, inferior, cemented, usually thinly bedded and of more recent geological age, e.g. Hastings Beds (Lower Cretaceous); Upper Greensand (Cretaceous)	50°–70°	33°–37°	Fairly resistant to weathering, depending on the degree of hardness and the nature of the cementing material. Stone with a silicious cement resists weathering better than one with a calcareous or ferruginous cement.
Shales, e.g. Ludlow Shale (Silurian); Shales of Yoredale Series and Coal Measures (Carboniferous); Shales of Lower and Upper Lias (Jurassic)	45°–60°	34°–38°	Resists weathering to a considerable degree, though the face tends to flake in small fragments. Softening may also occur in time. Particular attention should be given to the relation between the slope of the cutting and the dip of the strata.
Marls, e.g. Keuper Marl (Triassic); Chalk Marl (Cretaceous)	55°–70°	33°–36°	Resists weathering well if care is taken with drainage of cuttings; growth of vegetation should be assisted. These rocks are liable to soften with time.
Limestone, strong, massive, of considerable geological age, e.g. Carboniferous Limestone (Carboniferous); Magnesian Limestone (Permian)	Mainly vertical 70°–90°	38°–42°	Resists weathering well but frost action and weathering of joints exposed in cuttings tend to loosen large blocks, which therefore have to be removed at intervals.
Limestone, weaker, including Oolitic Limestones, e.g. Portland Beds, Coral Rag, Lias (Jurassic)	Mainly vertical 70°–90°	38°–42°	These rocks vary considerably in their ability to resist weathering; the massive Portland Beds are usually very resistant, whilst the Coral Rag and Lias may weather badly, causing slips in overlying strata which may require apron walls or similar protective measures.

Table 3 — Design of slopes in rock cuttings and embankments *(by courtesy of the British Railways Board)*

Type of rock	**Cuttings: safe slopes** (angles referred to the horizontal)	**Embankments: angles of repose** (angles referred to the horizontal)	**Remarks**
Chalk (lower, middle, upper) (Cretaceous subdivisions)	45°–80°	33°–36°	The Lower Chalk is generally massive, homogeneous and therefore more resistant to weathering than the Middle Chalk; it may thus in many cases permit a higher safe slope angle in cuttings. The Upper Chalk is comparatively weak and more fractured; it therefore requires a lower angle of safe slope. Frost action produces much fragmentation, and the growth of vegetation should be encouraged as an aid to binding the surface. The general effects of weathering are towards producing a 45° slope.
Igneous Rocks, e.g. Granite, Dolerite, Basalt, Andesite, Gabbro	80°–90°	37°–42°	Weather-resisting qualities excellent. Cuttings may be left almost vertical after removal of loose fragments. Some basalts may exfoliate to a slight extent after long periods of exposure to weather.
Metamorphic Rocks, e.g. Gneiss, Quartzite, Schist, Slate	60°–90°	34°–38°	Gneiss and Quartzite generally exhibit similar properties to Granite or hard Sandstone, weather-resisting qualities are excellent, and slopes may be left almost vertical. Gneisses are often severely contorted. Schist may vary from strong pelitic material through gradations to talc schist or mica schist which may be weak, approaching the consistency of shale. The weaker schists may weather to a considerable extent and tend to slide along the surface of schistosity. Schists are often severely contorted and may require variations in the safe slope angle in cuttings to allow for the local difference of the schistosity. Slate is generally strong fine-grained rock with excellent weathering qualities, although the effects of weathering may tend to cause sliding along the sleavage planes. Both slates and schists are subject to cleavage along planes generally at a high angle to the bedding plane and may tend to slide not only in the direction of the cleavage or schistosity but also down the dip of the original bedding planes. This is especially dangerous where the dip of either of these surfaces is in the same direction as the slope of the face of the cutting.

6.2.2 *Selection of parameters of soil and rock masses for assessment of slope stability*

6.2.2.1 *Soils*. The fundamental equation which is used to calculate the shearing strength of the material behind a cutting face is:

$$s = c' + (\sigma_n - u) \tan \phi' \tag{2}$$

where

- s is the resistance to shear along an actual or potential slip surface;
- c' is the cohesion intercept of the soil or rock mass in respect of effective stress;
- σ_n is the total normal stress on the plane under consideration;
- u is the pore pressure;
- ϕ' is the angle of shearing resistance with respect to effective stress.

The parameters c' and ϕ' are not uniquely related to the particle size and mineral composition of a particular soil but they depend on the conditions imposed on a specimen of the soil when it is subjected to the shear tests in the field or laboratory. Accordingly, the test location and test specimen should be selected taking into account the frequency, duration and direction of stress changes, the anticipated or permitted deformation of the cutting slope and the predicted pore pressure conditions.

Field tests made in cohesive soils in boreholes are of limited value for determining effective stress parameters since the values which are obtained represent the conditions for the state of stress in the ground at the time of making the test, whereas the values which are needed are those which result from stress changes consequent on excavation of the cutting.

Standard penetration tests (BS 5930) can be made in boreholes in cohesionless soils to obtain a measure of the relative density and angle of shearing resistance using standard published empirical relationships [14]. These values, although they may be affected by borehole disturbance, are likely to be more representative of in situ conditions than tests made in the laboratory because of difficulties in extracting reasonably undisturbed samples of cohesionless soils and preparing specimens for test. Static cone penetration tests (BS 5930) are likely to provide more representative values of in situ conditions than standard penetration tests in boreholes. Again, values of relative density and angle of shearing resistance can be obtained from empirical relationships [14]. These values obtained from field tests can be applied to stability calculations since, because of the relatively high permeability of cohesionless soils, stress changes resulting from excavation for a cutting take place immediately and there is no long term deterioration in overall stability.

Vane shear tests (BS 5930) can be made in soft to firm normally-consolidated clays to obtain values of undrained shear strength from which the short term stability of slopes can be calculated (see **6.4** and **6.5**). However, the results are applicable only to rapid excavations, e.g. for trenches. In the case of highway or railway cuttings which may take days or months for excavation, the stress changes have a significant effect on the shear strength over the construction period, and stability calculations should be based on effective stress parameters obtained from laboratory tests. Moreover, standard vane tests cannot take into account the effects of anisotropy (see below). Correction factors can be applied to undrained shear strengths obtained from vane tests to take account of time effects and anisotropy where stability conditions are required for a period of only a few weeks [15].

Field tests which measure undrained strengths are inappropriate for determining shear strength parameters for use in calculating the medium and long term stability of clay slopes. Tests of this kind measure the in situ undrained strength representative of the stress on the soil at the level of the test; whereas when a cutting is excavated there is a reduction in total stress followed by an increase in shear stress and softening of the clay. Shear strength parameters for medium and long-term stability should therefore be determined from laboratory measurements of the effective cohesion and angle of shearing resistance.

The borehole records and sample descriptions should be studied in relation to the geometry of possible slip configurations (see **6.4**), and layers which are critical to slope stability should be defined at this stage. Specimens for laboratory shear tests should be selected from these critical layers and subjected to a range of tests which will simulate the behaviour of the soil mass in the cutting over the period of time for which stability has to be ensured. The following aspects should be considered in preparing the test programme.

a) *Size of specimen*. The size of the test specimen should take account of the structure and fabric of the soil. Specimens 100 mm in diameter are satisfactory for intact, fissured or laminated clays, or clays with stoney inclusions. Specimens of 200 mm or 250 mm diameter should be considered where soil fabric conditions are critical to stability [5].

b) *Anisotropy*. The effective stress parameters may vary according to the direction of application of the deviator stress to the test specimen, because of the effect of discontinuities in the specimen or of the difference in the ratio of vertical to horizontal stress in the soil in situ [15]. The orientation of specimens cut from borehole samples or block samples should take into account the direction of the potential slip surface in relation to the orientation of the sample.

c) *Drainage*. The drainage permitted from test specimens should take time effects into account (**6.2.5**).

d) *Pore pressure*. The effect of pore pressure changes in the test specimen should be studied in relation to predicted variation in piezometric levels behind the cutting slope (**6.2.4**).

e) *Displacement during shear*. The shearing resistance of most clays rises to a peak value at a small displacement and falls to a lower "residual" value at large displacements. The selected parameters should take account of predicted or permissible deformations of the cutting. The residual shear strength values are appropriate to studies of shear on ancient or tectonic displacement slip surfaces where large displacements have already occurred.

f) *Softening*. Selected test specimens should be tested after softening by absorption of water to investigate effects of heavy and prolonged rainfall on a slope.

g) *Rate and frequency of stress application*. The rate of shearing should take account of creep effects. The effects of repeated cycling of the deviator stress may need to be studied in connection with possible earthquake effects or periodic rise and fall in piezometric levels.

6.2.2.2 *Rocks*. The resistance to shearing of a rock mass depends on the shear strength of discontinuities such as fissures, bedding joints and fault zones (see BS 5930). The strength of the intact rock is of no relevance to stability calculations for a cutting. However, the strength of the intact rock may be used for purposes of classification [2].

The shear strength of discontinuities depends on the roughness and waviness of the two rock surfaces at the discontinuity, the distance between them, and the cohesion and shear strength of any infilling material. Faults may consist entirely of weak slickensided infilling material.

The shear strength parameters at discontinuities should be obtained from direct shear tests on cores or blocks of rocks aligned so that shear takes place along a discontinuity incorporated in the test specimen. Field tests are inappropriate for the reasons given for clays (**6.2.2.1**). Tests should be made in the laboratory to simulate stress changes which take place in the rock mass behind a cutting in a manner similar to that described for clays. In the absence of test data reference can be made to published values of shearing resistance of rock joints [1].

6.2.3 *Structure and fabric of soils and rocks*

6.2.3.1 *Soils*. The structure and fabric of the soil mass behind the cutting slope should be studied in relation to its effect on the permeability of the mass, and the orientation of discontinuities should be studied in relation to the direction of potential failure surfaces. Typical structural and fabric conditions which should be investigated are as follows.

a) *Laminations and layering*. Layers or laminations of silt or clay in a cohesionless soil prevent vertical drainage. Sand or gravel layers in cohesive soils accelerate drainage and consolidation of the soil mass, but flow through these layers may lead to local erosion and undercutting of a slope.

b) *Fissuring*. The influence of fissures has been described. Consideration should be given to the likelihood of widening of fissures due to relief of stress in a cutting, and to the effects of drying shrinkage.

c) *Root holes, rodent holes, etc*. These form channels for drainage and dissipation of pore pressure. They can also permit entry of surface water into a soil mass.

d) *Slickensided surfaces*. These are the smooth and often polished surfaces resulting from movements in recent or geological times. Shear strengths are relatively low on these surfaces due to polishing.

6.2.3.2 *Rocks*. The structure of a rock mass has a dominant effect on the stability of a cutting slope. Factors to be considered are as follows.

a) *Thickness of beds*. Thinly-bedded rocks are more susceptible to weathering and degradation than thickly-bedded massive rocks.

b) *Weak rocks interbedded with strong rocks*. The weak beds weather more rapidly, resulting in undercutting and collapse of the overlying massive rocks in horizontally or near horizontally bedded strata.

c) *Orientation of bedding and joint planes*. Where bedding and joints are inclined in the same direction as the slope, the risks of failure are higher than when they dip into the slope. The location of joint systems should be studied in relation to the various forms of failure described in **6.3**. Account should be taken of the likelihood of joints opening owing to stress relief and to weathering (**6.2.5.2**). The assessment of stability conditions in a rock mass may be facilitated by plotting the direction and inclination of principal discontinuities on a polar diagram.

d) *Conditions at joint faces*. These should be studied in relation to the selection of shear strength parameters obtained from direct shear tests (**6.2.2.2**).

6.2.4 ***Water: surface and sub-surface***

6.2.4.1 *Soils*. Surface water flowing down the slope of a cutting can cause erosion of cohesionless or partly cohesive soils. Erosion is usually concentrated in channels which may eventually erode more deeply to form gullies down the slope.

Cohesive soils are not very susceptible to erosion, except in the case of heavily fissured clays where strong water seepage and softening may result in debris slides (**6.3.4.4**).

Sub-surface water flow, whether in the form of seepage from higher ground beyond the cutting or in the form of surface water entering through fissures or holes, can have a critical effect on stability. Trials should be made with various elevations and shapes of the phreatic surface resulting from seepage flow towards the cutting. From these trial surfaces the pore water pressure at any point on a potential failure surface can be obtained and its effect on shearing resistance can be calculated in accordance with equation (2).

Consideration should be given to changes in the pore water pressure with time. Before the cutting is excavated the pore water pressure corresponds to natural ground water level or to the hydrostatic pressure in permeable layers in a layered soil formation. After excavation of the cutting, the piezometric level falls progressively at a rate depending on the permeability of the soil, with particular reference to any impermeable soil layers which can impede vertical drainage. Eventually the pore pressure reaches an equilibrium value corresponding to steady seepage conditions but subject to any short term variations due to rainfall. If the pore pressures behind the slope are too high to achieve stability, consideration should be given to installing some form of drainage in order to lower water levels and reduce the pressures (see **6.5.4**).

6.2.4.2 *Rocks*. Rock slopes may be subject to erosion by surface water flow if the material is heavily jointed or in the form of a loose scree.

It is impossible to predict the true phreatic levels of sub-surface flow through a jointed rock mass because the water flows through preferred channels following open joints or joints containing permeable infilling material. The effect on slope stability of a range of ground water levels should be studied by means of idealized models covering different flow patterns which may be planar or preferred channel flow [16]. Consideration should be given to the effect of stress relief when the opening of joints and fissures may change the pattern of flow.

6.2.5 ***Time effects***

6.2.5.1 *Short, intermediate and long term stability*. Excavation for a cutting wholly or partially relieves an element of the soil at or beneath the slope or toe of the cutting from lateral and vertical stress. Consequently there is a reduction in total stress accompanied by a change in shear stress on the element, usually leading to a reduction of pore pressures within the slope [Figure 1(a)]. The overall effect of this is generally to increase temporarily the stability of the slope. These processes are discussed more fully in [17].

With time the reduced pore pressures rise toward higher equilibrium values, by seepage of pore water from zones of greater piezometric pressure in the area beyond the influence of the cutting. The increase in pore pressures from the temporarily lowered conditions beneath the slope causes swelling and softening of the soil mass with a consequent reduction in its shearing resistance. The stability of the slope continues to decrease until the pore pressures beneath and beyond the slope have reached their equilibrium values. Thus the most critical conditions for slope stability are in the long term after equilibrium conditions have been attained. In the case of highly permeable soils the pore pressures fall rapidly to an equilibrium value and stable conditions corresponding to the equilibrium pore pressures are thus rapidly achieved. In the case of soils of low permeability, this process may take many decades [18].

In the case of embankments the total stress on an element of soil beneath the embankment increases as the crest of the bank is raised. This results in an increase in pore pressure and a decrease in shearing resistance of the soil forming the foundation. The stability conditions are therefore at their most critical during construction and immediately after completion of the earthworks. With time, the increased pore pressures are dissipated and the stability of the embankment continues to increase until equilibrium conditions are attained [Figure 1(b)].

The governing factor in determining the rate of change of pore pressure and hence the stability of the earthworks is the coefficient of swelling of the soil in the case of cuttings, and its coefficient of consolidation in the case of embankment foundations.

In the case of relatively shallow excavations in clays, say for trenches or foundation pits, any beneficial effects on the short term stability produced by temporary reduction in pore pressures are generally heavily outweighed by the detrimental effects caused by the opening of natural fissures in the clay as a result of stress relief. Short term stable conditions cannot therefore be relied upon in vertical-sided cuts in clay, and where there is any danger to operatives or risks to buildings and services, either support has to be given to the sides of the excavation as described in section 3 of this code or the slopes have to be cut back to an angle at which stability in the short to intermediate term can be assured.

Similar considerations apply to the stability of rock slopes affected by the opening of joints and fissures due to stress relief and by the effects of hydrostatic pressure within discontinuities or the effects of changes of pore pressure within materials infilling the discontinuities.

6.2.5.2 *The effects of weathering*. Unless protective measures are adopted as described in clause **11**, rain, frost, sun and wind cause degradation of soil and rock slopes resulting in erosion and the accumulation of loose material at the toe. Weathering also influences the stability of clay slopes, when drying shrinkage causes cracks to open forming planes of weakness and channels for entry of water. The effects of drying shrinkage are critical to slope stability in normally-consolidated clays. Usually such clays have a crust of firm to stiff material which is over-consolidated owing to the effects of drying and vegetation. This crust usually contains fissures extending through the firm to stiff layer. Hence it may be unwise to rely on the contribution of this layer to the overall shearing resistance of the soil mass when using the methods of analysis described in **6.4**. The presence of vertical fissures in the crust can cause immediate toppling failure in a trench excavated with vertical sides. In these conditions no reliance can be placed on short term stability brought about by temporary reduction of pore pressure as described in **6.2.5.1**.

6.2.6 *Other factors influencing stability*. Other factors to be considered in relation to the stability of slopes of cuttings are as follows.

a) *Surcharge on slopes*. Slopes or the ground beyond the crest may be permanently surcharged by structures such as bridge abutments, traffic signal gantries or buildings. The effects of temporary surcharge by construction plant or stacked materials should also be considered.

b) *Mining subsidence*. Settlement of a cutting may occur as a result of mining subsidence. This may cause weakening of the soils by shear displacements, or changes in the pattern of ground water flow. The effects of movements in joints or renewed movements in fault zones in a rock mass should be considered. Collapse of the surface can occur over old mineworkings.

c) *Seismic effects*. Ground vibrations from earthquakes or construction operations may cause a temporary increase in pore pressure behind a slope. The effects of this on shearing resistance should be considered. Earthquakes may disturb sensitive clays (see **6.2.1.3**), causing a reduction in their undrained shear strength. Joints in a rock mass may be opened or disturbed by earthquakes or blasting vibrations, leading to general weakening of the mass.

6.3 Modes of failure of cuttings and natural slopes

6.3.1 *General*. Instability of the slopes of cuttings on natural hillsides can in many cases be described in terms of different groups or modes of failure. These depend on the height and geometry of the slope, the structure of the soil or rock mass and the surface and sub-surface water conditions. Recognizable and definable modes of failure form the basis for analysing the stability of a proposed cutting slope and for studying the causes of failure of an existing slope. These methods of analysis are described in **6.4**.

6.3.2 *Rotational slides*. Rotational slides (Figure 2) are a form of instability occurring in fairly uniform soils or in structureless or heavily jointed rock masses. Failure occurs when the gravitational forces acting on the mass of the soil or rock above the sliding surface exceed the resisting forces on this curved surface. The curved surface may be circular [Figure 2(a)] or non-circular [Figure 2(b)].

Rotational slides can be considered to act three-dimensionally when a spoon-shaped mass of soil slips on a concave surface [Figure 2(c)].

The movement in a rotational slide is a bodily one resulting in a backward tilt at the crest and an upward and forward movement at the toe. There is little relative disturbance within the mass except at the toe where there may be some rolling and tumbling. Spreading can also occur [Figure 2(c)]. The non-circular slide [Figure 2(b)] is usually associated with anisotropy in the soil or with the presence of a relatively strong layer at a horizontal or shallow inclination, when sliding takes place on this layer.

6.3.3 *Compound slides*. Compound slides are partly rotational and partly translational in character and occur in soils where hetereogeneity results in shear failure taking place on preferred surfaces which may not have any regular pattern in relation to the geometry of the slope. Compound slides are typical of stiff over-consolidated clays where surface shrinkage cracks and natural fissures form multiple sliding surfaces. These multiple slips can occur at random positions and in varying dimensions on a slope (Figure 3), or failure may take place near the toe followed by successive slips working back up to the crest. The latter are known as multiple retrogessive slides (Figure 4).

6.3.4 *Translational slides*

6.3.4.1 *General*. Translational slides occur as a result of weakness in a soil or rock mass at a fairly shallow depth beneath the slope surface. The slide involves the bodily movement of a shallow mass on a planar surface roughly parallel to the slope.

6.3.4.2 *Slab and block slides*. Slab and block slides are forms of translational movement where the sliding mass remains more or less intact. A slab slide (Figure 5) typically occurs in the weathered surface of an existing slope. The weathered surface may be a stiff fissured crust of clay sliding over a fairly large surface on a plane of weakness in the underlying soft weathered clay. Similarly, large areas of rock can slide on a weak clay-filled joint parallel to the slope. A thick layer of top soil may slide as an intact carpet off inclined competent surfaces, particularly after heavy rain. A block slide occurs when a block of relatively strong rock or stiff to hard clay moves down-slope as a unit on a plane of weakness in the form of a fissure or joint roughly parallel to the slope (Figure 6).

6.3.4.3 *Wedge failures*. A wedge failure is essentially three-dimensional in form and occurs when a wedge of rock or stiff clay slides bodily forward and downward on two or three well defined joint planes which intersect behind the slope (Figure 7).

6.3.4.4 *Debris slides*. Debris slides occur when water has access to debris forming a mantle on a slope [19]. The water and debris in a random unsorted form move down the slope either slowly in a creep movement, which may be seasonal in character, or rapidly at times of heavy rainfall or as a result of diversion of surface water onto a slope.

Existing hillside slopes may consist of debris slide material resulting from past periglacial action. These are known as solifluction sheets or lobes and they can occur on flat slopes as a result of freezing and thawing of a relatively shallow mass of soil or rock moving on deep permanently frozen ground.

Debris slides can occur in the disturbed material at the toe of a rotational slide [Figure 2(c)], and on the surface of a slope following internal erosion of permeable layers (**6.3.7**).

Bogbursts occur when a mass of peat or marshy soil becomes surcharged with water at times of heavy rainfall. The liquid mass spreads outwards and if it is sited on a slope it can flow at high velocity down the slope.

6.3.4.5 *Flow slides*. Flow slides occur in loose to medium dense saturated coarse soils as a result of a sudden increase in pore pressure in the mass, e.g. by earthquake action, vibrations from explosives or heavy construction equipment, or by rapid drawdown of water levels [20].

In loose uniform fine sands flow slides can occur on relatively flat slopes, the bodily movement occurring over long distances at a high velocity. Flow slides can also occur with cohesive soils, when they are usually termed mudflows.

6.3.5 *Toppling*. Toppling failures occur in rock slopes where discontinuities behind the face are steeply inclined (Figure 8). They may occur as a result of water pressure behind the slope [21].

6.3.6 *Falls*. Falls occur from steeply cut faces in soils, e.g. in excavations for trenches or foundation pits when only short term stability is required. Cracks open behind the face as a result of stress relief or drying shrinkage. Failure occurs near the base of the free-standing column of soil bounded by the crack system, and the mass of soil falls forward or slides into the cut (Figure 9).

6.3.7 *Internal erosion*. Internal erosion can occur behind the face of a cutting excavated in a water-bearing layered soil formation consisting of interbedded permeable and impermeable soils. Flow of water occurs through the permeable layers and it may emerge at the face with sufficient velocity to cause movement of soil particles. The erosion works backwards into the slope until it reaches the stage when undercutting and collapse of the overlying material occurs. The collapsed materials then move down the slope in the form of a debris slide (**6.3.4.4**).

6.4 Methods of analysis of stability of slopes

6.4.1 *General*. **6.4.2** to **6.4.4** indicate the methods currently in use for the analysis of stability of slopes. In the more commonly used methods the basic equations are given together with references to the original papers. The latter should always be consulted before using the equations.

6.4.2 *Limit-equilibrium methods*

6.4.2.1 *Circular rotational slides*. For simplicity in analysis rotational shear slides (Figure 2) are normally considered as two-dimensional. Thus in the analysis the sliding mass is represented by a thin slice of soil of unit length measured along the length of the toe and aligned at right angles to the toe. The complications involved in analysing rotational slides as a three-dimensional failure are not usually justified, and in normal cases a two-dimensional analysis gives a conservative value for the safety factor.

Under appropriate conditions the short term stability (**6.2.5.1**) of slopes can be calculated by a total stress analysis based on the undrained shear strength of a clay. The soil is considered to be fully saturated with incompressible water and it is postulated that there is no change in the volume or shape of the soil mass due to excavation for the cutting and that the soil shear strength is unchanged as a result of change in total stress. The shortcomings and possible risks involved in this method of analysis have been pointed out in **6.2.1.3**. However, an effective stress analysis may be unrealistic for short term stability because of uncertainty of pore pressure changes immediately after rapid excavation. The total stress analysis should not be used for intermediate or long term stability conditions in clay slopes.

The simpler form of total stress analysis assumes the undrained shear strength, s_u to be constant with depth and lateral extent. A trial slip circle is drawn (Figure 10) and the disturbing movement produced by gravitational forces on the mass above the circle is equated to the restoring moment provided by the shear strength of the soil along the circular arc. In the limit-equilibrium method the shear strength is reduced by a safety factor, F, given by

$$F\,(\text{short term}) = \frac{s_u R^2 \theta}{\gamma A x} \tag{3}$$

where x is the distance of the line of action γA from the centre of the circle, the angle θ is expressed in radians and other terms are as defined in Figure 10.

A number of trial circles are drawn from which the lowest safety factor is established.

When the shear strength varies (laterally or with depth) the mass of soil above the circle is divided into a number of strips, usually vertical (Figure 11) and a summation is made of the disturbing and restoring moments in each strip. The safety factor, F, is calculated from the equation

$$F\,(\text{short term}) = \frac{R \Sigma s_u l}{\Sigma W x} \tag{4}$$

where the terms are as defined in Figure 11.

Equation (3) can be expressed in the form of a dimensionless chart [22]. A chart is also available for the case of undrained shear strength of normally-consolidated clays increasing linearly with depth in relation to the plasticity index of the soil [23].

The intermediate and long term stability of slopes (see **6.2.5.1**) should be analysed by effective stress methods using shear strengths calculated from equation (2). This analysis is again two dimensional. The slice of soil is divided into a number of vertical strips (Figure 12).

Ignoring the forces between the strips, for strips of equal width the safety factor, F, is given, using the Bishop simplified method [23A], by the equation

$$F\,(\text{long term}) = \frac{\Sigma\left\{\dfrac{[c' + (p-u)\tan\phi']}{m_\alpha}\right\}}{\Sigma\, p \sin\alpha} \quad (5)$$

where

u is the pore water pressure at the base of the strip

$$m_\alpha = \frac{\cos\alpha\,(l + \tan\alpha\tan\phi')}{F} \quad (6)$$

and other terms are as defined in Figure 12.

As the term F appears on both sides of equation (5), an iterative solution is required using a grid of sufficient extent to cover all likely centres of rotation.

As a further simplification of the method of slices for use in simple earthworks problems, or where only limited data on shear strength values are available, the safety factor, F, can be obtained from the equation

$$F\,(\text{long term}) = \frac{\Sigma\,[c'l + W(\cos\alpha - ul)\tan\phi']}{\Sigma\, W \sin\alpha} \quad (7)$$

The values of c' and ϕ' for use in equations (5) to (7) should be obtained from drained shear strength tests, or undrained tests with pore pressure measurement. The type of test, the test conditions, and the selection of parameters from a range of tests should take account of the factors listed in **6.2.2.1**, with particular reference to the effects of anisotropy, the rate of applying the deviator stress and the geological history of the site [15].

The pore pressure, u, may be controlled by ground water levels which are independent of stress changes brought about by excavation for the cutting. Thus flow towards the slope may occur in a permeable soil and the pore pressures can then be determined by drawing a flow net for the seepage conditions. In investigating the stability of natural slopes where the flow pattern has reached the stage of steady seepage conditions, the pore pressure can be obtained from piezometers installed in the slope, due regard being given to seasonal changes. In soils of low permeability the pore pressure changes respond only slowly to stress changes as expressed by the equation

$$u = u_o + \Delta u \quad (8)$$

where

u_o is the initial value of pore pressure before any change of stress, and

Δu is the change in pore pressure due to change in principle stresses $\Delta\sigma_1$ and $\Delta\sigma_2$.

The change in pore pressure can be obtained from the equation

$$\Delta u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)] \quad (9)$$

where A and B are pore pressure parameters obtained from undrained triaxial compression tests [25, 26].

In practice a number of trial slip circles are investigated to obtained the lowest value of the safety factor F from equations (5) or (7).

These equations can be expressed in a dimensionless form. For the preparation of the charts it is convenient to express the pore pressure at any point in the form of a simple ratio:

$$r_u = \frac{u}{\gamma h} \quad (10)$$

where

r_u is the pore pressure ratio

γ is the density of soil

h is the height from point under consideration of soil

It has been shown [24] that for a simple slope profile the safety factor of the slope varies linearly with the pore pressure ratio, and it can be expressed by the equation

$$F = m - nr_u \quad (11)$$

The stability coefficients *m* and *n* form the basis of non-dimensional charts [24] from which the safety factor of a slope can be calculated on a trial and error basis. Equation (11) assumes that the pore pressure ratio is constant across the slope profile. The ratio varies with time and reference should be made to published works for design values [18, 25].

The charts do not take into account the weakening effect of a tension crack extending below the ground surface (Figure 13). This may be significant for a relatively shallow cutting. The crack can be allowed for if the method of slices is adopted using equations (5) or (7). Theoretically the depth of a tension crack is given by the equation

$$\text{depth} = \frac{2c_u}{\gamma} \text{ (for undrained, i.e. short term, conditions)} \qquad (12)$$

or

$$\text{depth} = \frac{2c'}{\gamma} \cot (45 - \phi'/2) \text{ (for drained, i.e. long term, conditions)} \qquad (13)$$

The crack depth cannot, of course, exceed the depth of the tension zone, which is probably about one-third to one-half of the slope height.

6.4.2.2 *Non-circular rotational slides.* Non-circular slip surfaces of general shape are usually analysed by the method of Morgenstern and Price [27] or of Sarma [28]. These methods normally require the use of computers.

Where the sliding surface follows a well-defined weak layer the stability can be analysed by assuming failure of a block of soil acted on by the active pressure on the vertical face of the block and resisted by the shear force on the horizontal face and the passive pressure at the toe. Thus from Figure 14, the safety factor against sliding of the block is given by

$$F = \frac{(W' \tan \phi' + c' L) + P_p}{P_A} \qquad (14)$$

where

P_A is the active pressure on the block of soil

P_P is the passive pressure at the toe of the block

W' is the effective weight of the block

and other terms are as defined in Figure 14.

Equation (14) can be applied to rock slopes to analyse the stability of a block of rock sliding on a bedding plane or joint plane [29].

6.4.2.3 *Planar slides.* The planar slide in a long slope can be considered to consist of a number of strips of unit width (Figure 15). The internal forces on the sides of the strip are assumed to be equal, opposite, and colinear and therefore cancel out. Thus the safety factor against sliding at a depth *z* is given by:

$$F = \frac{c' + (\gamma z \cos^2 \beta - u) \tan \phi'}{\gamma z \sin \beta \cos \beta} \qquad (15)$$

This method of analysis, referred to as infinite slope analysis, can be used for cases of slab or block slides in long rock slopes (see **6.3.4.2**). Equation (15) does not allow for the effect of seepage forces.

6.4.2.4 *Wedge failures.* Wedge failures in a rock mass (see **6.3.4.3**) have to be analysed on a three-dimensional basis considering the cohesion, angle of shearing resistance and water pressure on each face. The gravitational force on the slide wedge is equated to the shearing resistance of the faces. Design charts are available for analysis of wedges with up to three intersecting discontinuities [30].

6.4.3 *Stress analysis.* The stresses and displacements in the soil or rock mass behind a cutting slope can be analysed using the principle of continuum mechanics. In the finite element method [31, 32] the mass is sub-divided into a number of structural components or elements the shape of which can be varied to suit the geometry of the slope and the presence of layers or joint systems. The elements are assumed to be connected at a discrete number of points or nodes on their boundaries and a function is chosen to define uniquely the state of displacement within each element in terms of nodal displacement at element boundaries. From a knowledge of the deformation moduli of the soil or rock mass as determined from field or laboratory tests, the strains and hence the stresses at boundaries of the elements can be calculated. The contributions of each element are summed to obtain the stresses and displacements in the whole mass. The calculated stresses can then be compared with available strengths.

For simplicity it is usual to make finite element analyses of soil slopes in a two-dimensional form, but for analysis of a jointed rock mass a three-dimensional analysis may be required. Because of the large number of variables it is doubtful whether the results of such a complex analysis are worthwhile, in view of the irregular character of most rock joint systems. However, the method may be useful as parametric study for analysing the sensitivity of slope behaviour to varying such parameters as slope angle and the cohesion and friction on a joint plane, and in cases where the magnitude of deformations of earthworks is a critical factor in design.

6.4.4 *Physical models*

6.4.4.1 *Phenomenological models*. In analysing the stability of rock slopes where the rock mass is intersected by a well-defined joint system, or where a joint system can be idealized in a simplified form, it may be helpful to study the mode of failure by means of a three-dimensional scale model [14]. Such models can be prepared from wood or plaster with saw cuts to simulate the joints. They can be helpful in deciding on the most favourable alignment of the cutting slope relative to the dip and strike of the principal joint planes (see **6.2.3.2**).

Models of this kind can give a qualitative indication of the mode of failure but they should not be used as a means of quantitative analysis.

6.4.4.2 *Scale models*. The stability of cutting slopes in soil can be analysed by constructing a model of the slope to scale using a soil sample representative of the mass or of layers in the soil mass. The model is then subjected to a scaled regime of the ground water flow and a centrifugal acceleration ng (where g is the acceleration due to gravity) and distortions measured to predict the behaviour of the full scale slope [33].

The advantages of the centrifugal model are that three-dimensional and complex layered soil systems can be modelled. Time effects are also reproduced, which is advantageous in cases where stability conditions intermediate between the short and long term are required. The difficulty in using a centrifugal model study is in deciding on the number of soil samples and variations in a layered soil system which needed to be modelled in order to ensure that the results are representative of site conditions.

6.5 Design

6.5.1 *Assessment of safety*

6.5.1.1 *Local and overall stability*. When preparing designs for the alignment and slopes of a cutting, the possibility of local slips or falls occurring on the face of the slopes should be considered, in addition to the overall stability against the various forms of failure described in **6.3**. Local slips or falls may occur owing to the presence of random pockets of weak, unstable, or water-bearing soils, or thin layers of weak or shattered rocks. In most cases local instability can be dealt with as and when it becomes evident by adopting one or more of the remedial measures described in **6.5.5** and **11.4**. An overall flattening of the slopes due to the occurrence of these local failures is rarely justified.

6.5.1.2 *Safety factors*. Suitable values of safety factors in a particular case can only be arrived at after a careful study of all the relevant factors and the exercise of sound engineering judgment. These factors include the complexity of the soil conditions, the adequacy of the site investigation, the certainty with which the design parameters, e.g. shear strength and ground water pressures, represent the actual in situ conditions, the length of time over which stability has to be assured, the likelihood of unfavourable changes in ground water regime, surface profile or other factors taking place in the future and the likely speed of movement and consequences of any failures. In general, however, it is important to distinguish between first-time slides and slides on pre-existing slip surfaces for over-consolidated cohesive soils.

First-time slides are generally characterized by a degree of brittleness with concomitant possibilities of progressive failure and sometimes rapid run-out, whereas in slides of the latter category these problems tend to be diminished or absent. Current practice would suggest that for first-time slides with a good standard of investigation and for which the consideration that needs to be given to the other factors mentioned in **5.3** is no more than average, a safety factor between 1.3 and 1.4 should be designed for. For a slide involving entirely pre-existing slip surfaces, but otherwise of similar status, a safety factor of about 1.2 should be provided.

6.5.1.3 *Probability*. The design of cutting slopes may be considered from the aspect of probability of failure. This approach involves analyses to assess the sensitivity of the design to changes in the significant parameters. On the basis of the limit-equilibrium method (**6.4.2**), the value of one of the important parameters required to satisfy the equilibrium conditions is calculated for a range of values of the other parameters involved. For example, the value of cohesion required to satisfy a condition of limiting equilibrium can be calculated for a range of friction angles and ground water conditions. Alternatively, each significant parameter can be varied in turn while keeping the values of the other parameters constant. Thus the sensitivity of the safety factor to variations in each parameter can be evaluated. The *rate of change* of the safety factor caused by variations in each parameter is a reliable indicator of engineering behaviour. This rate of change can be demonstrated by presenting the results of the sensitivity analyses in graphical form [34, 35].

6.5.2 ***Slope profile***

6.5.2.1 *Slope angles*. The required slope angles of cuttings should be calculated by the analytical methods described in **6.4** or by empirical methods. Angles flatter than those suggested by consideration of the minimum safety factor instability may be desirable for ease of maintenance, aesthetics, or other reasons (see **6.1**).

The slope within the water-bearing formation should be such as to permit steady seepage from the toe without risk of erosion. The required slope angle for stability under conditions of steady seepage can be determined with the aid of a flow net (see CP 2004).

If the cutting is excavated in layers of soil or rock of significantly differing characteristics the slope angles may be varied to conform to the engineering behaviour of each formation. The slope angles may also be varied in uniform water-bearing previous soils by adopting a steep slope approaching the angle of repose of the soil located above the highest ground water level, and a flatter slope below the water table.

6.5.2.2 *Berms*. Where a water-bearing pervious soil or rock formation overlies an impervious formation, a berm should be provided at the level of the interface between the two formations. The berm is used to accommodate an open channel or piped drain provided to collect seepage from the pervious soil above.

A berm may also be provided to trap falls of rock or other debris from a high steeply-cut face. The vertical interval between the berms and the width of the berms in relation to the height and slope of the cutting should be selected to ensure that boulders do not roll down the slope and bounce off projections to fall at a dangerous distance beyond the toe [36].

The surface of berms, whether in soil or rock formation, should be sloped back from the face, and provided with drainage to avoid the spill of water down the lower slope at times of heavy surface water run-off.

6.5.2.3 *Space for debris*. Sufficient width of space should be provided at the toe of a steep slope in soil or rock to prevent debris accumulating on footpaths, roads and other accessible areas. A space is also required to trap falling rocks or soil which might cause a hazard to life or property. The width of the debris trap is governed by the same rules as berm widths [37]. If insufficient space is available for the calculated width, a fence or wall is required along the outer margin. These structures should be of sufficient height to prevent boulders from bouncing over them.

6.5.3 ***Influence of construction procedure on slope stability***

6.5.3.1 *Sequence and geometry of excavation*. Temporary slopes in cutting excavations should not be cut so steeply that slips or falls endanger the stability of the permanent slope. Temporary slopes may be formed as a series of steps with steeply cut faces. The height of each step and the width of each berm should be such that the required overall slope for the full height of the cutting is not exceeded, and that no danger is caused to construction operatives. At the final stage of excavation the steps are trimmed back to the final design profile. In no circumstances should a face be cut steeply so that it slumps.

6.5.3.2 *Effect of explosives*. When explosives are used to loosen rock in cutting excavations, the blasting vibrations may open fissures and joints in the rock mass. This may be detrimental to the stability of a steeply cut face. Blasting vibrations can be minimized by a suitable blast-hole pattern in conjunction with relays and delay detonators. Opening of fissures and joints beyond the limits of a cutting can, in some circumstances, be minimized by using a controlled blasting technique such as pre-splitting (see **16.3**).

6.5.3.3 *Control of ground water*. The rate of excavation in a pervious water-bearing soil should be controlled so that steady seepage conditions from the slope are achieved and maintained at all stages, so avoiding rapid draw-down of ground water levels behind the slope with consequent risk of instability. The base of an excavation should, where practicable, have appropriate cross or longitudinal falls to prevent water from ponding in the working area. Temporary open channel or piped drains may be required as described in **6.5.4.1**.

Where a cutting in an impervious soil is underlaid by a stratum containing water under sub-artesian pressure, precautions should be taken against a heave or "blow" at the base of the excavation. If normal drainage techniques are inadequate it may be necessary to relieve the water pressure by relief wells or by pumping from well points or bored wells (see CP 2004).

6.5.4 *Drainage*

6.5.4.1 *Surface water control*. Drainage may be required at the top of a cutting slope to intercept surface water flowing towards the excavation, and so to prevent the water from discharging down the slope.

This drainage can take the form of open channels, ditches, or piped drains. The gradient of the drains should, as far as possible, be the optimum for the particular type, and should not be flatter than 1 in 300 unless the drains have the main purpose of providing storage capacity for run-off, when flatter gradients may be acceptable.

The required capacity of drains depends on the nature of the soil, the contour of the ground, e.g. whether sidelong or otherwise, and the existence of springs, agricultural drains, or water channels which may be interfered with by the cutting.

Open channels should preferably be lined. Unlined channels should not be used where interceptor drains are sited near the top of a cutting in a shrinkable clay, since water seeping down shrinkage cracks may cause instability. Open channel or piped drains should be provided at the toe of the slope on both sides of a cutting, and the formation should be trimmed to fall towards the drains. Care should be taken that the construction of the drains does not weaken the toes of the slopes. All drains sited at the tops of cuttings and designed to carry surface water should be lined. Where both surface and subsoil drainage have to be provided it is desirable to install separate conduits for each purpose. Open channels should not be so deep as to render cleaning difficult, 1.2 m being usually a practicable maximum.

Pipe drains should not be less than 150 mm in diameter. When acting as subsoil drains they are laid with open joints and may be bedded on concrete. Alternatively, a porous concrete or perforated pipe may be used. The pipe trenches should normally be completely filled with coarse granular material, broken to 60 mm maximum size. Where the surrounding ground is likely to be washed or squeezed into the drain, a graded filter material should be used between the soil and the coarse filling (see CP 2004). Alternatively the pipes may be wrapped in a durable plastics filter fabric. Where coarse filling surrounding impervious pipes is specified a connection should be made from the filling into the manhole or catchpit at the low end of the run of pipe to avoid a buildup of subsoil water in the filling material. Inspection chambers, incorporating silt traps and enabling a drain to be rodded, should be constructed at intervals of from 50 m to 90 m. The covers to the inspection chambers should permit ready opening for regular inspection.

Where cuttings are constructed in soil containing sulphates injurious to Portland cement, concrete or mortar the use of a sulphate-resisting cement should be considered for all concrete work and earthenware, or PVC pipes should be used instead of concrete pipes (see CP 2004).

6.5.4.2 *Trench and counterfort drains*. Trench and counterfort drains; sometimes described as pillar drains, buttress drains, and batter drains, have been used extensively as a remedy for slips in cuttings and embankments. They are sometimes employed as a design measure when their use may render it possible to work to a somewhat steeper slope than would prove stable without them, and where deep-seated slips are likely to occur. They are rarely constructed in embankments except as a remedial measure after a slip has occurred.

The function of trench and counterfort drains is to reduce pore water pressures in the slope. Counterfort drains, defined as trench drains which are carried into solid ground below a slip surface, provide in addition to drainage some buttressing action. A tentative method of designing trench drains, and the drainage function of counterfort drains, is given in [37].

Auxiliary drains, sometimes described as chevron, vee, or herringbone drains, are often provided between the main counterfort drains in the form of narrow stone-filled trenches extending up the slope from points in the main buttresses at an angle of about 45°, and meeting midway between the main drains. These auxiliary drains are seldom more than 0.75 m × 0.5 m to 0.75 m wide, and may be spaced from 3 m to 10 m apart. Their function is to intercept water flowing down the face of the cutting and to add to the stability of the surface of the earth slope. Like counterforts they are filled with rubble and may in addition be piped or concreted.

Counterfort and chevron drains should be so constructed that neither erosion nor seepage conditions are set up in the bank. Water-bearing outcrops in the face of slopes may require special drainage.

The trench in which the counterfort drain is formed is usually benched. On slopes of, say, about 2.5 in 1, the length of the benches might be 2.5 m, and the height of each step 1 m with a maximum depth of excavation in newly-made slopes of 1.5 m to 2 m. Where constructed in order to remedy slips, the contour should be determined after ascertaining the position of the surface of sliding, the benching being constructed well below this and below the surface of any deeper slip which is likely to occur. The surface of a slip can usually be detected in the excavation. Counterfort drains may be 1 m to 2 m wide and are sometimes made wider at the bottom of the slope than at the top, with width increasing by about 1 m in 10 m of length.

The unsupported width of natural earth or "span" between counterfort drains (edge to edge) may be as little as 3 m or up to 10 m, depending on the conditions on site and on whether auxiliary drains are used.

Excavations should be lined with filter media consisting of hard durable granular material, selected for its permeability and having a grading which permits the entry of soil particles from the surrounding ground.

Excavations should be thoroughly waterproofed at their base to prevent water getting into the body of the slope. Such materials as concrete, plastics sheets, or bituminous coatings may be used. As an alternative, step construction may be adopted. Such steps should be cut to a fall and successive concrete layers should overhang and include a central channel. It is important to prevent clogging of drains in the long term. Erosion of fine soil into the drain-filling material can be prevented by lining the trench with filter media and/or filter fabric. Entry of surface water into trench drains can be prevented by plugging the top metre of the trench with compacted clay fill placed on a layer of filter material.

As an alternative to forming trench drains within the slope, consideration should be given to constructing a deep cut-off drain parallel to and behind the top of the slope.

6.5.4.3 *Face drains*. Face drains or blanket drains can be used to control seepage from the face of a slope in a pervious soil. The drains may consist of a layer of coarse granular material up to 60 mm in size backed by a graded filter (see CP 2004) or by filter fabric, or stone-filled wire mattresses similarly backed by filter material.

6.5.4.4 *Bored drains*. The ground water level in pervious soil or rock formations may be lowered by pumping from vertical bored wells. This may be desirable as a construction expedient to permit rapid excavation in dry conditions, until such time as steady seepage towards the cutting slopes is attained. Bored wells can also be used as a remedial measure to lower the piezometric head causing instability of a slope. The system is uneconomical as a permanent drainage measure unless gravity drainage of a cutting can be achieved. The design of bored wells is described in CP 2004.

Inclined bored drains can be installed from the face of a cutting slope. They are installed by rotary or percussion drilling a cased hole to intercept the water-bearing horizons. The piped drain, consisting of a perforated plastics pipe surrounded by plastics filter fabric or a perforated metal pipe surrounded by a gravel or porous concrete annular filter, is pushed down the casing. The latter is withdrawn, leaving the piped drain in place, when flow from the pipe is under gravity.

Inclined drains are a useful expedient where seepage takes place from relatively thin water-bearing layers, or to lower the piezometric head on a potential sliding surface [37].

6.5.4.5 *Galleries*. Drainage galleries may be constructed by tunnelling from the face of a slope or from a shaft excavated behind the slope. Drainage galleries are costly compared with inclined bored drains but they may be advantageous where seepage takes place from closely-spaced fissures or laminations in a rock formation. The gallery can be tunnelled to intercept the source of seepage and then continued along the water-bearing horizon to the extent necessary to achieve the required lowering of piezometric pressures behind the slope. Drainage galleries provide a means of access for supplementary stabilization measures such as transverse adits, inclined bored drains, or grouting. The galleries are constructed on an upward gradient to permit drainage by gravity towards the portal through a piped drain constructed beneath the floor of the gallery. The drain should have a removable cover for easy inspection and maintenance.

6.5.4.6 *Electro-osmosis*. Fine-grained soils such as sandy silts, silts, and silty clays, are difficult to drain because capillary forces acting on the pore water prevent free flow under gravity to a drainage system. However, flow can be induced under an electrical potential whereby direct current is made to flow from anodes (which are steel rods driven into the soil) to filter wells forming the cathodes. The positively charged water particles migrate through pores in the soil and collect at the cathodes from where they are pumped to the ground surface. The installation and running costs of the system are high compared with gravity drainage or simple pumped wells; therefore electro-osmosis is normally restricted to use as an expedient to arrest movement of an unstable slope until permanent remedial measures can be designed and constructed (see CP 2004).

6.5.5 ***Mechanical methods of support***

6.5.5.1 *General*. Where the available width between the toe of a cutting and the site boundary is insufficient to accommodate a safe slope for the full height of the cutting, it is necessary to introduce support in the form of a retaining wall in order to reduce the slope, or to adopt some other means of artificially creating a steepened slope.

6.5.5.2 *Retaining walls*. Suitable forms of retaining wall to give support to a cutting slope are

- gravity walls in mass concrete, brickwork, or stone masonry
- reinforced concrete walls of L- or T-shape
- reinforced concrete buttress walls
- reinforced concrete counterfort walls
- reinforced concrete diaphragm walls (cantilevered or anchored)
- contiguous bored pile walls (cantilevered or anchored)
- secant bored pile walls
- reinforced earth walls
- steel sheet piling (cantilevered or anchored)
- crib walls, e.g. precast concrete sections or disused rubber vehicle tyres
- gabions (stone-filled wire mesh baskets)

Information on methods of design and construction of the above types of wall can be found in CP 2 and CP 2004. Gravity walls or reinforced concrete walls are suitable if the soil in the lower part of the cutting can be cut back steeply to a temporary slope to allow the wall to be constructed. Any space between the back of the wall and the temporary slope is then backfilled. Alternatively these walls can be constructed in a timbered trench, the soil in front of the wall being removed after completing the retaining structure.

Diaphragm walls, contiguous bored pile walls and secant bored pile walls are suitable for weak, unstable or heavily water-bearing soils where a temporary steep slope cannot be formed or where construction in a trench would cause problems of support or loss of ground. These types of retaining wall are also suitable for sites where construction is to be undertaken in close proximity to existing structures (see CP 2004).

Reinforced earth retaining walls can be formed in the lower part of a cutting slope by excavating at the toe to form a temporary steep slope, then replacing the excavated soil in compacted layers of essentially granular material, each layer being reinforced by horizontal metal or plastics ties. The steeply inclined face of the retaining wall is protected by metal, reinforced concrete, or plastics cladding elements (Figure 19). The design of reinforced earth walls is described in various publications [39, 40, 41, 42].

Reinforced earth retaining walls have the advantage of flexibility and are suitable for soil conditions where appreciable forward movement or heaving of a cutting is anticipated as a consequence of stress relief, or where the site of a cutting may be subject to subsidence from underground mine workings.

Steel sheet piling is used as a remedial measure to restore stability (see **11.4**) or as a construction expedient. It can be used as a permanent retaining wall if consideration is given to some measure of protection against corrosion where a very long life is required.

Precast concrete crib walls are a form of gravity section and may be economical for sites where suitable broken rock or gravel is available as a fill material for the cribs.

Gabions are suitable for sites where broken rock, boulders, cobbles or large gravel are available for filling the wire mesh baskets and where space is available to arrange the baskets in tiers to form a stepped-back retaining wall. A very long life is not possible with gabion walls but the galvanized or plastics-covered wire mesh can provide many years of useful support. Gabions are particularly suitable for construction in conditions where the cutting is temporarily or permanently flooded and subjected to scour from flowing water. The flexibility of a gabion retaining wall is advantageous for sites where appreciable deformation of a slope may occur as a result of stress relief or subsidence due to underground mine workings.

In all cases and for all types of retaining wall attention should be given to drainage at the back of the wall in order to prevent hydrostatic pressure on the retaining structure and to avoid a general rise in pore pressure in the soil or rock mass behind the wall. The drainage layer behind the retaining structure may consist of a layer of granular material, or no-fines concrete. Where there is a risk of loss of fines from the retained soil, a graded filter or filter fabric should be used (see CP 2004) or the granular drainage layer should be backed by a sheet of filter fabric.

6.5.5.3 *Ground anchors*. A steep soil slope can be retained by reinforced concrete slabs in precast or cast-in-place construction, and restrained against movement by steel rods or cables anchored in the soil mass beyond the zone of potential sliding (Figure 16). As an alternative to the design shown in Figure 16, the anchored retaining structures may be designed as wholly or partially buried vertical sleeper walls stepped down the slope to form a series of terraces. Information on the design of ground anchors can be found in CP 2004, DD....[3] and [38].

6.5.5.4 *Rock bolting*. Rock bolts can be used as a means of preventing degradation of a weak or heavily jointed rock face, or as a means of restraint to bodily movement of a mass of rock. Where a good visual appearance is not a requirement, the slope can be covered with wire mesh pinned to the rock face by short bolts or tied back to the crest of the slope by vertical anchor bolts. This technique is suitable only for the retention of relatively light blocks of rock on the surface of a slope, or when the mesh is anchored only at the crest of the slope to control the fall of larger blocks. In the latter case the mesh and anchorage should have adequate strength.

Isolated pinnacles or buttresses of rock which may fail by toppling (see **6.3.5**) can be pinned back to stable rock behind by means of short bolts (Figure 17). Similarly, shallow rock masses which may show instability in the form of slab or block slides (see **6.3.4.2**) can be restrained by short bolts extending below the potential slip plane (Figure 18).

The short bolts used for rock face treatment can be inserted into holes drilled in the rock and then grouted with cement or with a polyester or epoxy resin in the form of an unstressed dowel. Alternatively, stressed anchors can be used (see CP 2).

Larger rock masses can be prevented from showing instability in the form of translational or wedge slides (see **6.3.4**) by means of long rock anchors grouted into the rock mass beyond the zone of potential instability.

Where short or long anchors are used to stabilize rock slopes the effect of grouting for securing the anchors or the drainage of the slope should be considered. Extensive grouting of a jointed rock mass may create a barrier to ground water flow towards the cutting. Where necessary, drainage holes should be drilled into the face to relieve hydrostatic pressure.

Attention should be given to the prevention of corrosion of anchors by means of suitable sleeving in their non-bonded lengths. Anchor heads should be accessible for re-stressing, if necessary, and suitable measures of corrosion protection should be provided to exposed bolt heads, locking mechanisms and reaction plates.

[3] DD . . . . *Ground anchors* (in course of preparation).

6.5.5.5 *Dental treatment of rock faces*. Sprayed concrete with or without steel mesh reinforcement may be used as a construction technique in shattered rock. In this case the concrete holds the rock fragments in position as the slope is formed and prevents further movement and release of lateral stress which could lead to general instability of the slope surface if a large amount of strain were permitted throughout the mass. A rapid set is often required for this purpose and reinforcement is required if the mortar or concrete facing is to be built up to a substantial thickness. Wet or dry mortar systems are available.

6.5.6 *Improvements to soil parameters*. Grouting by physico-chemical methods may be considered as a means of stabilizing a soil mass to permit steep slopes, but its principal use is as a remedial measure. Descriptions of the physico-chemical methods are given in **11.4**.

6.6 Monitoring of slopes

6.6.1 *General*. Where experience or stability analysis gives reasonable assurance of stable conditions in a cutting slope no special measures are required for monitoring stability. However, it is good practice to make periodic inspections, particularly in the early months after completion when the surface may be subject to erosion before grass cover is established. These inspections should include the following observations.

a) *Deformation*. Settlements in the upper part of the slope and bulging towards the toe may indicate incipient failure by a rotational shear slide (**6.3.2**).

b) *Cracking*. A series of cracks in the vicinity of and sub-parallel to the crest of a slope may indicate sliding, as do *en echelon* cracks at the lateral boundaries of incipient movement. Hexagonal or random pattern cracking indicates drying shrinkage.

c) *Fissuring*. Opening of joints and fissures in a rock slope indicates incipient translational or toppling failure (**6.3.4** and **6.3.5**).

d) *Seepage*. Water carrying soil particles seeping from a slope face indicates internal or seepage erosion (**6.3.7**).

e) *Gullying*. Channels eroded on a slope face indicate the need for protection against surface erosion.

Inspections should be made after periods of heavy rainfall, snow or severe frost. Clay slopes should be inspected during or immediately after rainfall following a period of dry weather to assess the effects of water entering surface cracks. Inspection of the position and inclination of pegs driven into the slope is a simple means of detecting gross deformations.

Inspection of slopes of cuttings during the construction period and of steeply-cut slopes for foundation excavations or trenches should be made daily by suitably qualified and experienced persons to ensure safe working conditions for operatives, and to avoid damage to partly constructed works or existing structures around the excavation (see **4.1**).

Where there are doubts concerning the short or long term stability of cutting slopes it may be desirable to install instrumentation to give warning of incipient instability, to enable remedial measures such as the installation of drainage, grouting or anchoring to be undertaken before the stage of failure is reached. Suitable methods of monitoring slopes are described in **6.6.2.** to **6.6.5**.

6.6.2. *Water pressure*. Pore pressures behind a cutting slope can have a critical effect on stability (**6.2**), so it may be desirable to monitor pore pressure changes during and after excavation of a cutting to check the validity of assumptions made at the design stage and to ensure that critical conditions of high pore pressures are not developing.

In homogeneous permeable soils pore pressures can be monitored by plumbing water levels in simple standpipes (see BS 5930). In layered soils or soils of moderate to low permeability the response time of standpipes to changes in pore pressure may be inadequate to detect critical conditions in sufficient time to take remedial action. In these cases pore pressures should be monitored by properly sealed and protected piezometers (BS 5930) with their tips located in each critical soil layer at a number of locations along the slope. Water levels in the piezometers can be monitored by plumbing down the riser pipe or by connecting a series of piezometers to a gauge house by means of pneumatic, hydraulic or electrical transmission and recording systems [43]. Precautions are necessary against damage to a piezometer installation from construction and maintenance operations and from the effects of frost and vandalism.

6.6.3 *Monitoring surface and sub-surface movements*. Monitoring of ground surface movement in both horizontal and vertical planes can be carried out by field survey methods. The particular methods used depend on the accuracy required.

For short term schemes when a high degree of accuracy is not required, simple measurements taken on metal pins or pegs driven into the soil can be taken by normal levelling, tachometric survey methods or short range Electronic Distance Measuring equipment (EDM), with the measurements referred to one or more stable base line stations set some distance from the affected area.

Where a higher order of accuracy is required (± 5 mm or better) and the measurements are expected to be repeated at regular intervals over a long period of time, a properly designed monitoring scheme will be necessary. Consideration should then be given to use of one or a combination of the following methods:

a) precise levelling using a geodetic level and Invar staff;

b) triangulation using first order theodolites (reading to one second of arc);

c) trilateration with special EDM equipment.

These measurements should be taken from stable survey monuments, preferably with fixed centring for the instruments or referred to deep bench marks or datum points. The targets should be designed to provide a unique point to which the measurements can be taken during repeated visits.

Computation should be by normal survey procedures, and the results adjusted by "least squares" or "variation-of-coordinates" to obtain the best mean answer. Sub-surface deformations of a slope may be measured by inclinometer readings in boreholes located at critical points (see **5.1.3.5**). The location of the boreholes at ground level is obtained by methods a), b) and c) above.

Photogrammetry may be used for monitoring purposes, but when a high degree of accuracy is required the ground control would need to be established by methods a), b) and c) above.

6.6.4 *Earth pressure measurements*. It may be desirable to monitor the development of earth pressure on the retaining structures described in **6.5.5**. Earth pressures are measured by means of pressure cells interposed between the soil and the face of the retaining structure or by means of load cells mounted on components such as anchors, struts and shores.

6.6.5 *Seismic observations*. Seismic refraction surveys made at periodic intervals can give an indication of movement of a rock slope since opening or closing of fissures affect the seismic velocity of the rock mass [17]. Micro-seismometers installed in boreholes can give warnings of slope movements (see **5.1.3.6**).

7 Embankments and general filling

7.1 General factors affecting the design of embankments. The cross section of the embankment is determined by the required functional width at the top, the height above ground level and the profile of the slide slopes. At the base sufficient land should be reserved for the location of any necessary drains and services at the toes of the slopes and any other ancillary purposes such as landscaping, including tree or hedge planting. The cross section may also be governed by the stability of the ground on which the embankment is to be constructed, particularly if this is sidelong ground.

In the design of approach embankments to bridges and other structures, the superstructures, substructures and associated earthworks should be studied as a whole and not individually.

Where sufficient width of land is not available to accommodate the full width of the base of the embankment, the provision of earth-retaining structures has to be considered. Types of earth-retaining structure are described in **6.5.5.2**. Selection of a type should take into account the method of forming the embankment and other general factors dealt with in clause **5** of this code.

7.2 Strength and deformation characteristics of foundations and fill materials

7.2.1 ***Materials***

7.2.1.1 *Rock*. Problems associated with strength and deformation of the foundation are unlikely where an embankment is built on rock. Rockfill placed in layers and suitably compacted can form a fill material with excellent strength and deformation properties. However, some weak rocks such as mudstones, shales, marl and chalk can degrade very quickly if exposed to the elements, or if inappropriate construction methods are used. They then behave as cohesive soils. Where embankments are to be formed by dumping rock into water or if they are to act as permeable structures permitting flow of water through them, only strong durable rock should be used (see **7.5.1**).

7.2.1.2 *Granular soils*. In general, owing to their high permeability, granular soils do not allow excess pore water pressures to develop during embankment construction. As a consequence, embankment loading increases the strength of the granular soil and for the most part deformations due to embankment loading occur immediately as construction proceeds.

In a foundation of granular soil for a wide structure such as an embankment there is little risk of shear failure. Difficulties may occur with fine sands which are saturated and/or loosely packed in their natural state. In such a state a "quick" condition can be set up in the foundation by vibration caused, for instance, by traffic, particularly construction traffic, or by the driving of piles.

Granular soil when placed in layers and given suitable compaction forms a high quality fill material, although difficulty can be experienced in the compaction of the final layer of single-sized material unless the grading is improved by adding the appropriate material.

7.2.1.3 *Cohesive soils*. These soils have low permeability, and excess pore water pressures can develop within them when the soil is subjected to a change in loading. A cohesive soil may therefore behave in an essentially undrained manner with very little consolidation occurring during embankment construction, and no significant increase in strength occurs during that period. The major proportion of the consolidation settlements occurs after the end of construction as the excess pore water pressures dissipate. The strength and deformation properties of cohesive soils as both fill and foundation material are largely a function of moisture content, but in the case of the foundation material the structure and fabric of the soil, resulting from its geological history are also important (see **5.2**).

7.2.1.4 *Silts*. Silts are cohesive soils posing particular additional problems. With properties intermediate in character between clays and sands their strength and deformation behaviour are very susceptible to instability caused by disturbance.

7.2.1.5 *Peat*. Peat, which can vary from peaty clay to fibrous peat, is unsuitable as a fill material. Being highly compressible, it should be removed or displaced from beneath an embankment if occurring in layers of significant thickness. Where removal is impracticable, efforts should be made to accelerate settlement by surcharging the embankment, provided that such surcharging does not endanger stability.

7.2.1.6 *Industrial and domestic wastes*. Industrial wastes that are likely to be considered for suitability either as a foundation or as a fill material are as follows: burnt and unburnt colliery shales, various types of slag from the metal-producing industries, quarry waste, pulverized fuel ash from power stations and waste products from the chemical and manufacturing industries. In general, untreated domestic waste is unsuitable for use either as a foundation or as a fill material. However, with the increasing use of incineration treatment, limited supplies of a suitable fill material from domestic waste may be available [7, 8, 9].

When old mining spoil tips are used as borrow pits, the excavated faces should be left sealed to prevent entry of air which might cause spontaneous combustion. Apparently burnt-out tips may contain zones of hot materials, and work should be arranged to avoid falls of dry dusty incandescent material. Before entering tips on calm days tests for the presence of noxious gases, particularly carbon monoxide and carbon dioxide, should be made in low-lying or confined areas.

Care should be exercised in selecting waste products for use either as a foundation or as a fill material, and tests should be made to determine their general suitability, including tests for sulphate content, toxicity, combustibility, and mechanical properties. Careful consideration should be given to the possibility of consequential pollution. Mining wastes or slag, particularly steel slag, which have been stockpiled for long periods, may cause pollution of water-courses or subsoil water in areas surrounding the filled site. Problems with internal erosion or leaching of fines into drainage systems may occur when fine-grained materials such as pulverized fuel ash are used

7.2.2 *Suitability and testing*. The strength, deformation and moisture susceptibility of foundation and fill material should be established by means of:

a) in situ testing as part of site investigation;

b) laboratory tests;

c) instrumented field trials.

In the case of large rock embankments field trials should be carried out in order to determine the best procedures both for excavation and for forming a satisfactory embankment [44].

Some materials, such as silty sands, silty clays and chalk, have a critical level of moisture content above which they rapidly become unsuitable for normal methods of earthworks construction. Laboratory examination should be made of the relationship between moisture content, density and undrained shear strength or CBR values for all types of soil exhibiting predominantly cohesive properties.

7.3 Design of embankments

7.3.1 *General*. Embankments should be designed to provide an adequate safety factor for shear failure and to ensure that any deformation is within acceptable limits. The information required before the cross section of the embankment can be designed includes:

a) ultimate width of top of embankment;

b) loading on top of embankment;

c) geotechnical properties of the foundation and fill materials;

d) restrictions on width of land available;

e) special conditions to which the embankment would be subject, for example, tidal waters, active mining operation and natural cavities, and environmental and other economic factors which may influence the final choice of cross section, e.g. earth banks for sound screening or flattening of slopes to allow them to be returned to agriculture.

7.3.2 *Stability*. Calculation of the stability of the embankment should be undertaken using the methods of analysis described in **6.4**. In some instances, it may be desirable to analyse embankment deformations using, for example, finite element methods described in **6.4.3** to determine whether deformation is acceptable.

Parameters of the shear strength of the fill appropriate for use in the stability calculations are usually obtained from laboratory tests on recompacted samples. Where an embankment is built of rockfill or other granular material with side slopes not exceeding the angle of repose of the fill, it is inherently stable for all heights as long as the foundations are capable of sustaining the loads. However, the angle of shearing resistance of a well compacted granular fill can be considerably greater than the angle of repose and consequently the laboratory determination of this parameter and its use in the stability calculations can lead to a more economic embankment cross section. For rockfill embankments, where laboratory determination of the angle of shearing resistance of the fill material may be difficult, recommended angles are given in Table 3. The shear strength and pore pressure parameters of clays and silts can be measured in laboratory triaxial compression tests.

If the natural moisture content of the material in the field is high but the permeability characteristics are such that it can be readily reduced, the design could take advantage of the resulting improvement in shear strength.

Where embankments are constructed on sidelong ground and a layer of impermeable material underlies a significant thickness of permeable material, a perched water table can form, causing saturation of the coarser material with possible erosion or slumping where the water table emerges onto the side slope.

The stability of an embankment depends not only upon the strength of the fill material from which it has been formed but also upon the strength of the material on which it is founded. An assessment is necessary to check the ability of the foundation material to carry the required superimposed load without shear failure or unacceptable deformations. The factors governing the behaviour of soils and rocks in cuttings (see **6.2**) generally apply also to their behaviour as foundation materials for embankments. If the site contains geological features such as faults or slip surfaces resulting from previous movements, due regard should be taken of these during the evaluation of the stability of the embankment.

Techniques are available for improving the strength properties of fill and foundation materials, and these are described in CP 2004 and [45].

The effects of embankment loading on materials of low shear strength can be mitigated by the use of berms or by flattening the side slopes of the embankment.

7.3.3 *Deformations*

7.3.3.1 *General*. Some deformation of the fill, of the foundation materials or of both may occur and the behaviour of the materials involved should be studied at the site investigation stage to determine their settlement characteristics. The acceptable degree of settlement depends on the type of function the embankment is required to serve, e.g. to carry a highway or railway or for building developments. In some cases, it may be necessary to induce the major part of the settlement before the filled area is required to be used. This may be done either by completing the fill early in the contract and topping up as necessary during the completion stage, or by surcharging the fill by increasing the height to accelerate the settlement, the excess material being removed before completion.

7.3.3.2 *Fill materials.* The different materials which are suitable for fill have varied consolidation characteristics. Consolidation tests may have to be carried out on samples compacted to the anticipated site density in order to calculate settlements. Adequate compaction minimizes but does not necessarily eliminate future settlement. The degree of compaction achieved depends on the moisture content of the fill material and the amount of compactive effort.

Care should be taken to avoid indiscriminate mixing of soils of widely differing characteristics e.g. clay with chalk or drier than average cohesive soil with wetter than average material.

7.3.3.3 *Foundation materials.* Factors which may give rise to settlement problems within the foundation materials include:

a) cohesive soils of high compressibility;

b) peat;

c) changes in ground water levels due to extraction by pumping or natural causes;

d) underground voids such as old mine workings or natural cavities;

e) active mining, including salt extraction by pumping.

The site investigation should be directed towards the discovery of such features and the necessary laboratory and/or field testing should be carried out so that the designer can take account of them.

7.4 Drainage

7.4.1 *General.* Drainage systems to deal separately with subsoil water and surface water run-off are essential, from both the constructional point of view and for the future stability of the embankment or general filled area.

7.4.2 *Pre-earthworks drainage.* Before an embankment can be constructed, existing watercourses, ditches, subsoil agricultural drainage, springs, ponds, etc., have to be dealt with so that the earthworks can be carried out without detriment to the existing ground water regime. Existing field drains should be intercepted by collector drains in the form of open-jointed pipes laid in a gravel-filled trench.

In the case where a new culvert is provided, its size, gradient and invert levels have to be agreed with the appropriate Regional Water Authority to ensure that possible run-off from future areas of development can be accommodated and any future regrading of the watercourse can be carried out both upstream and downstream of the embankment crossing.

Where it is necessary to provide a pipe under the embankment, it is always prudent to provide one of sufficient size to permit blockages to be cleared by working from the ends of the pipe.

To avoid damage by earthworks construction plant to pipes laid at existing ground level or at shallow depths, it may be necessary to protect the pipes by means of a concrete surround or by other methods.

7.4.3 *Temporary drainage during construction.* Adequate temporary outfalls have to be provided for the permanent drainage systems where the permanent outfall cannot be provided immediately.

During the formation of a fill care should be taken to leave areas which are not being worked with as smooth a surface as possible. In the case of cohesive materials the surface should be examined before resuming operations to ensure that a plane of weakness is not created. The surface should also slope to the outside edge of the fill so that rainfall does not pond on the surface and cause deterioration of the material. In areas subjected to periods of intense rainfall, as in tropical monsoons, if the embankment is constructed from erodible material it may be preferred to dish the embankment to a central ditch discharging to a suitable outfall.

In times of heavy rainfall large volumes of water run down embankment side slopes and may cause erosion of the fill. Where the surface of the embankment is to be left for an extended period, the problem can be alleviated by forming temporary grips or ditches leading to a lined outfall at the lowest point to discharge to a ditch or watercourse. In these situations the silting and pollution of existing watercourses should be prevented by providing temporary stilling ponds or by forming filter bunds in the temporary ditches.

7.4.4 *Drainage measures to improve and maintain the stability of embankment or general filling*

7.4.4.1 *Vertical drainage.* In situations where the horizontal permeability of the soil is inadequate to dissipate excess pore water pressures in the time required by the construction programme, the excess pore water pressure in the foundation material resulting from the embankment loading can be reduced by vertical drainage such as sand drains, wick drains or narrow width trench drains backfilled with granular filter material.

Provision should be made to deal with the emerging water, and this can be accomplished by a drainage blanket of rock or granular material in the base of the fill with suitable means of outfall. This drainage blanket can provide a useful haul road for construction traffic.

7.4.4.2 *Horizontal drainage*. Wet cohesive fill can be improved by the provision of horizontal drainage, i.e. layers of granular material placed at intervals within the embankment which reduce the pore water, with the resulting benefit of increased strength and accelerated consolidation of the fill material. If granular material is so placed then provision should be made to collect the emerging flow of water. The possible loss of fines at the interface has to be considered and either a suitably graded drainage layer or a permeable fabric membrane between the fill and the drainage layer should be used.

7.4.5 *Permanent drainage*. Surface water run-off from the surface of the fill and ground which falls towards it should be collected by the means described in **6.5.4**. Drains and services should not be sited close to the toe of an embankment in situations where lateral displacement of the soil due to horizontal ground strain could overstress or displace the pipework or other structures. Pipes with flexible joints which remain watertight under a certain amount of distortion of the pipe alignment are recommended, in order to prevent leakage into the fill in the event of settlement taking place. Particular attention should be given to drainage design adjacent to structures where differential settlement could cause leakage from the drains and wash out fine-grained filling such as pulverized fuel ash or cement flue ash.

7.4.6 *Construction considerations*. When drains are constructed before completing the fill, the remaining fill in the vicinity of the pipes should be placed and compacted in such a way as not to overstress the pipe.

7.5 Special site conditions affecting embankment or general fill design

7.5.1 *Fill deposited in water*

7.5.1.1 *Standing water*. Standing water is the term applied to ponds, lakes, canals and water-filled mineral workings. Where it is impracticable or uneconomical to drain standing water, particular attention in the design of the embankment should be given to the maximum and minimum water levels and to the characteristics of the soil underlying the water. Where practicable, any soft silt, clay or peat should be removed before placing fill, as it is difficult to compact the fill material under water. Fill should be selected from material which remains stable when inundated or when within the zone of a fluctuating water table, particularly in saline tidal water. Broken concrete, brick rubble or granular material should be used to reduce settlement and maintain stability. Where it is impracticable or uneconomical to remove soft materials displacement by end tipping of bulk filling can be adopted. Measures should be taken to equalize water levels on each side of the embankment by means of pipes or pervious blanket drains.

For large areas of standing water, it may be practicable and economical to adopt hydraulic filling using a suitable type of granular material (see **7.6.3.4**).

The slopes of an embankment in standing water should be flatter than those required above water level and they should be protected against wash or wave action as described in **11.4.4**.

7.5.1.2 *Tidal and flood waters*. In tidal waters the effects of the rise and fall of the water level and of wave action on the embankment require special consideration and techniques such as are necessary in the design of maritime structures. Where a sudden rise or fall in the level of the water may occur, precautions should be taken to avoid external erosion and to mitigate the effects of sudden drawdown. This condition can occur where an embankment crosses the flood plain of a river where the embankment is, for most of the time, on dry ground but where, under flood conditions, erosion of the slopes of the embankment in the vicinity of a bridge or culvert is possible owing to the increase in velocity of the flood water passing through the opening. The face of the embankment can be protected by one of the methods described in **11.4.4**.

7.5.2 *Embankments on soft ground*. Methods of constructing an embankment on soft ground include:

a) removal of the soft material by excavation plant or displacement by surcharging with suitable material;

b) providing a wide and deep trench filled with granular material at the toe of the embankment;

c) improving the properties of the foundation material by ground treatment methods, e.g. installation of vertical drains to accelerate consolidation, vibroflotation (the forming of columns of crushed stone within the soft material), dynamic consolidation [45], preloading with fill material;

d) controlling the rate of construction so that there is time for the foundation to consolidate and hence increase in strength sufficiently to remain stable. In these conditions it is desirable to monitor pore water pressures in the foundation material and lateral movements which may take place [13, 46];

e) decreasing the load applied by the embankment to the foundations by the use of lightweight fill, e.g. pulverized fuel ash;

f) providing a permeable horizontal drainage blanket beneath the excavation at ground level. Such a blanket facilitates rapid dispersal of flood water from the base of the embankment, thereby assisting stability. A permeable fabric membrane can be laid over the topsoil followed by a layer of granular material. It may be necessary to restrict the weight of mechanical plant operating on the blanket or on any shallow fill layer placed on the blanket;

g) where brushwood timber is available, the embankment may be constructed on fascines;

h) where sufficient land is available, the slopes of the embankment can be flattened by providing wide berms at the toe and, if required, at intermediate levels;

j) bridging over the soft material at ground level by means of reinforced concrete slabs or beams supported on piles. This is an expensive method of dealing with the problem but in some cases can be economically viable when there are considerable depths of peat to be crossed and the only acceptable alternative is to remove completely the unsuitable material.

7.5.3 *Embankments on sloping ground.* The inherent stability of the natural ground forming a slope should be investigated carefully, particularly in regions known to be prone to landslips. In some cases evidence of existing instability can be seen on the site in the form of undulations, hummocks, lobes and water seepages. Investigations should be made of the geological stability of the slope and the likely re-activation of the existing slips under the loading conditions arising from the embankment construction.

Where an embankment is to be constructed on sloping ground and there may be a danger of a slip developing at the interface, benches or steps should be cut into the existing ground surface to key-in the new construction. Preferably the bottom of the bench should be graded away from the surface of the slope, with provision for positive drainage measures to deal with any subsoil water which may collect at low points of the benching.

In order to deal with instability problems connected with the existing ground it may be necessary to design the cross sections of the embankment to obtain a safe distribution of loading on the ground. The method of building up the embankment may also require to be specified to prevent unbalanced loading. Drainage of the interface between the slope and the embankment and of any potential slip planes is most important and adequate cut-off and subsoil drains should be provided.

7.5.4 *Embankments over quarried ground.* Where an embankment is to be constructed over quarried ground consideration should be given to:

a) differing levels of the floor of the quarry, variable deposited material, and standing water. These problems have been discussed in previous sections of the code (see **7.5.1**). In cases where water is likely to be impounded at different levels on either side of an embankment, large diameter drains should be provided beneath the base of the fill to equalize water levels;

b) the transition between the quarry and the natural ground. In many cases there will be a vertical or near vertical face at this point and even with very careful control of the compaction at the face of the quarry, differential settlement is likely to occur. If practicable the face of the quarry should be graded back or stepped to provide a more gradual transition zone.

7.5.5 *Embankments on existing filled ground.* Where an embankment is to be built over existing filled ground the detailed history of the fill should be investigated in addition to ascertaining the character of the fill from boreholes and trial pits. Old filled areas often prove to be extremely variable and particular attention is required where the existence of domestic wastes is suspected. Extensive treatment may be necessary prior to embankment construction, depending on the depth and nature of the fill. Vibroflotation and dynamic consolidation are among the methods of ground treatment that may be suitable in some instances [45]. Alteration of the drainage pattern within the fill by the construction of an embankment may result in inundation of the fill and consequent leaking and collapse of arched materials.

7.5.6 *Embankments over mine workings and other underground voids*. Where mining operations are carried out adjacent to a geological fault, movement on the ground surface in the vicinity of the outcropping of the fault plane is likely to occur. Drains in embankments should not be carried across the likely zone of movement but should be terminated at each side and led to separate outfalls.

Problems of stability can arise from the presence of the following:

a) old mine shafts;

b) old drift mines;

c) shallow mine galleries;

d) areas of potential subsidence due to currently active and future coal or other mineral extraction by mining or pumping;

e) natural cavities arising from the action of underground water on soluble strata such as limestone, chalk, and gypsum.

Methods dealing with these conditions are described in [47].

7.6 Suitability of materials for fill

7.6.1 *Unsuitable materials*. The following materials are unsuitable under any circumstances for forming load-bearing fills:

a) organic soils, e.g. peat and some alluvial clays and silts;

b) toxic materials, e.g. industrial waste containing soluble compounds harmful to water supply or agriculture;

c) materials containing compounds harmful to other elements of construction, e.g. rejects from gypsum mining which can contain a high concentration of soluble sulphates harmful to concrete;

d) materials listed under **7.6.2** and **7.6.3** when they are in a frozen condition;

e) materials containing substances which can be dissolved or leached or which may undergo expansive reactions in the presence of moisture, e.g. pyritic shales.

7.6.2 *Suitable materials*

7.6.2.1 *Acceptable under any circumstances*. Material of high shear strength unaffected by changes in moisture content, e.g. strong durable rocks, gravel, medium and coarse sands, is suitable as a fill under any circumstances.

7.6.2.2 *Materials suitable within shear strength limits*. Recompacted silty fine sands, sandy silts, silts, clays and some weak or disintegrated rocks all show a reduction in shear strength as moisture content increases above the optimum at which a specified amount of compaction produces the maximum dry density (see Figure 20 and BS 1377). Some grades of chalk suffer reduction in shear strength when water is released from the cellular structure of the rock as the rock is broken down by the operations of excavation, placing and compaction. In the case of predominantly silty soils an increase in moisture content of as little as 1 % or 2 % can result in a very significant reduction in shear strength. Whether the minimum undrained shear strength of the recompacted material is acceptable is dictated either by the design requirements of the earthworks or by the need to allow passage of construction plant [48].

7.6.3 *Materials suitable with the adoption of special construction measures*

7.6.3.1 *General*. The following materials may be made acceptable by the adoption of the procedures set out in **7.6.3.2** to **7.6.3.5**:

a) chalk;

b) wet cohesive fill;

c) hydraulic fill;

d) combustible materials.

7.6.3.2 *Chalk*. It is important that the excavation and subsequent handling of chalk should be undertaken in such a manner as to preserve as much as possible of the natural structure of the rock. Chalk is one of the most difficult materials to handle in earthworks because any crushing of the cellular structure of the rock releases the water contained in the cells, which in turn saturates the crushed fragments of the chalk. The quality of the chalk should be assessed before deciding on the type of plant and methods to be used for excavation, haulage and compaction.

It is possible for the excavation and handling operations to cause degradation of the intact blocky chalk to a "putty" chalk which is unable to support earth-moving equipment. If this stage is reached, either sufficient time should be allowed to elapse for the pore water pressure and the moisture content to reduce, or the degraded material should be stabilized by alternating it with layers of a dry granular material.

The problem increases in severity with increase in the natural moisture content of the chalk and for the higher moisture contents, particularly in the weaker grades of the upper and middle chalk, it may be necessary to:

a) excavate by means of face shovels — these cause much less crushing per unit volume than other excavation equipment;

b) transport by means of limited weight lorries or conveyors [49] — this limits the stress on the placed layers;

c) apply compaction to each placed layer sufficient to close the surface but not sufficient to puttify the material and cause instability in the earthworks.

It is recommended that [50, 51 and 52] be studied before commencing earthworks in chalk.

7.6.3.3 *Wet cohesive fill, including some weak rocks*. The use of cohesive fill with undrained shear strength lower than that defined in **7.6.2.2** can involve considerable construction problems. The type of earth-moving plant that can be used will be restricted. Careful compaction sufficient to reduce air voids to an acceptable minimum is necessary, but over-compaction, which produces high pore water pressures and instability, is to be avoided. It may be necessary to adopt special measures such as the provision of layers of higher strength and/or free-draining fill and a controlled rate of construction.

7.6.3.4 *Hydraulic fill*. This term is used to describe all types of fill material that are carried to the embankment and placed while they are still suspended in water. Excavation is normally carried out by suction or bucket dredger and the fill transported by pipeline to the area of deposition. The discharge of the fill materials onto the fill area has to be controlled to avoid undesirable segregation, because larger particles settle out first and accumulation of the compressive and less permeable fine material settles out further from the point of discharge. Granular hydraulic fill, which can be dewatered by pumping or natural gravity drainage, consolidates rapidly to a reasonably dense state such that pavements or buildings can be constructed on the fill within a relatively short time of completing the deposition. Fill deposited through water settles into a loose state and where the fill is predominantly granular its load-carrying characteristics can be improved when necessary by such techniques as vibratory compaction [45]. A hydraulic fill that is cohesive may continue to settle by self-weight over a long period and where such settlements cannot be tolerated in the completed works pre-loading of the fill may be necessary. Consideration should be given to discharging the dredged cohesive material into lagoons with provision for removal of supernatant water. The material in the lagoons is allowed to dry and consolidate in stages followed by pumping in further layers of dredged soil. Drying is greatly accelerated by the growth of surface vegetation.

7.6.3.5 *Combustible materials*. Fills with a high organic content, such as carbonaceous shales, ash, some slags, coal wastes, timber residue and other organic industrial wastes, are subject to continuous degradation in moist aerobic conditions. Where sufficient air is available the degraded material can ignite. It is essential that the system adopted to place the fill be designed to prevent combustion, as described in **8.11**.

8 Excavation and filling

8.1 Planning

8.1.1 *General considerations*

8.1.1.1 *Extraordinary traffic*. The Highways Act 1959 requires that highways be not subjected to extraordinary traffic within the meaning of the Act. Routes and vehicles should be selected, and loads restricted or distributed, so that no unnecessary damage is caused to highways and bridges by the movement of plant and materials by extraordinary traffic to and from the project. This may require arrangements with the Highway Authority to strengthen the carriageways of bridges of minor roads.

8.1.1.2 *Public utilities*. All works should be carried out in full consultation with the relevant authorities. All services affected by the work should be protected or diverted as required by the appropriate statutory authorities. It may be necessary to carry out investigatory works to locate affected services.

Privately owned apparatus should be treated similarly in full consultation with the owners.

8.1.1.3 *Ancient monuments and archaelogical finds*. The existence or non-existence of ancient monuments or sites of archaeological interest within the earthworks boundaries should be established with the appropriate authority during the planning stages. Unless permission is given for removal, it may be necessary to delay commencement of the work or to halt construction whilst the find is surveyed.

8.1.1.4 *Highways*. Where access is required to and from the public highway, even though only of a temporary nature, the position, form and size of the access has to be agreed with the highway authority.

Where the works affect traffic using a highway it is essential that they be phased in a manner acceptable to the police and the highway authority and that an agreed programme be drawn up. All signs and traffic signals should be in accordance with the regulations of traffic signs.

All existing highways should be kept clean and clear of all dust, mud or other debris and the requirements of the highway authority in this respect should be complied with.

8.1.1.5 *Noise abatement regulations*. Attention is drawn to the Control of Pollution Act 1974.

8.1.1.6 *Location of borrow areas*. As with the disposal of spoil, borrow areas should be sited as near as possible to the area to be filled so that direct access to the site can be achieved as far as possible without use of public roads (see **5.2.1** and **8.11**).

Proper site investigation should be carried out so that the quality of the materials is correctly ascertained and the quantities available estimated to within reasonable limits.

In coalmining areas the possibility of coal seams close to the surface should be investigated. Problems of stability can arise where old mineworkings are present, and there is also a risk of spontaneous combustion. In these conditions reference should be made to the local Area Office of the National Coal Board.

Planning permission should be sought for developing a borrow pit so that all interested parties are consulted and suggestions made by them can be incorporated into the final scheme. In order to lessen the impact of a pit on the environment it is often desirable to incorporate into the scheme at the design stage plans for refilling the pit with the surplus spoil arising from the project and thus reducing the total area of land required.

Land used for borrow should be reinstated. If the land can be made to increase in value by the end of the project, this assists in its acquisition at the outset. Where existing contours prevent agriculture or development taking place, land can be modified after the borrow has ceased. Pits formed below normal water table level may be used for recreational purposes.

During the extraction period and before restoration, borrow areas have to be kept adequately fenced to prevent trespass and illegal tipping. Borrow pits are often dug below normal standing water level and therefore have direct access to aquifers which may be used for domestic water supply. In order to safeguard this supply, it is essential to ensure that neither the utilization of plant nor its operatives increase the pollution hazard. It may be necessary to install temporary drainage with satisfactory outfalls. Rather than cause temporary disturbance by excavating borrow pits in previously undisturbed land, industrial wastes can sometimes be incorporated into embankments. When spoil tips are adjacent to the earthworks use can frequently be made of the material, and work on the tip may be extended to incorporate landscaping. When the tips are at a distance from the site it may be uneconomic to use them unless the opportunity is taken to remove all or part of the tip in a single operation for the overall benefit of the community.

It is important to ensure in the siting of borrow pits that landslides or other movements are not initiated or reactivated. The working of borrow pits is subject to the same regulations that apply to construction and quarrying works. Protection is required in a similar manner and the works themselves should be undertaken in a safe manner. It is essential to avoid danger to the public from standing water in borrow pits (Control of Pollution Act 1974).

8.1.2 *Planning of work*. In British climatic conditions the dominant factor in preparing a programme of earthworks construction is the need to avoid major earth-moving during the winter months. In these months, normally from October or November to the following March or April, rainfall exceeds evaporation from the ground surface so that it may be impracticable to achieve efficient operation of excavating and compaction plant, particularly in heavy clay soils [48]. Granular soils and rocks can be excavated in wet weather but it may not be possible to achieve the specified standard of compaction when the excavated material is placed in fill areas. Earthworks commenced in the summer should not be continued into the winter months if the effect of earth-moving in wet weather is to damage the work already done.

Before commencing excavation all relevant data should be collected and drawings prepared showing the location of the excavation, tipping and filling. On these drawings both the excavation and the filling should be divided into sections, and the quantity of material to be excavated and filled stated in these sections. This information is required in order to plan economic hauls throughout the work.

Where the material to be excavated consists of different types, and if the various types have to be used separately in the fill or run to a spoil tip, the quantities of each class of material in each area should be shown on the drawings. From the nature of the material to be excavated and the method of its disposal, the type of excavation, the length of haul, and the amount of compaction necessary, it is possible to select the most suitable type of plant for excavating, transporting, placing and compacting the material.

8.2 Preparation of site

8.2.1 *General*. Before all or any part of the site is occupied adequate fences should be erected on the boundaries of the site to define its limits, to restrict construction plant to the site of the works, to protect the public and to prevent farm or other animals from straying onto the site or onto adjoining roads. Following this and before any earthworks operations are commenced, pre-earthworks drainage, if required, should be put in hand (see **7.4.2**).

8.2.2 *Site clearance*. All items within the site of the works required for re-use should be carefully removed and taken into store or re-erected immediately e.g. fences, gates, stone walls. Where articles are relatively small they should be clearly marked in order that they may be easily identified. Buildings are not to be left in a dangerous partly demolished state.

Items not required for re-use should be removed and disposed of as soon as possible in order to ensure a clear site for the earth-moving equipment. Care should be taken to ensure that the roots of all trees, hedges and brushwood are removed and disposed of as they can be a source of danger. Buried pipelines, concrete foundations, piles or slabs should be located as shown on site clearance drawings so that they can be removed safely before or during bulk excavation.

8.2.3 *Stripping topsoil*. Care is needed in the location of topsoil dumps to ensure that the soil is accessible and able to be easily transported to the section of the works where it is to be placed. Care should also be taken when stacking large volumes of topsoil that the heap is stable in itself and that the underlying natural ground is strong enough to support the additional weight. It is also important that only good organic topsoil is set aside for use in restoring land for agricultural use, and care should be taken to prevent the development of a nuisance from noxious weeds. There is often a layer of sterile soil which is unsuitable for agriculture, and if it cannot be used for fill in embankments it should be disposed of. If parts of the works are to be turfed and it is planned to obtain it from the site the turf should be cut and re-laid within one or two weeks of cutting, depending upon the time of year. A proper turf-cutting machine or tool should be used which produces either regular rectangles of turf or rolls of constant width and uniform thickness.

In some cases embankments are constructed on the natural ground without first removing the topsoil, for example, where the topsoil and vegetation are required to form a mat to act as a strengthening layer over very poor ground, or where it is impracticable or unnecessary to remove the topsoil.

8.2.4 *Treatment of ditches and watercourses*. Where watercourses are to be maintained through the works it may be necessary to carry out a temporary diversion whilst the permanent structure which is to carry the watercourse is constructed. Conversely, the watercourse may be diverted into a new channel to carry it around or through the works at some other point. In either case the invert and sides of the abandoned watercourse should be cleared of all unsuitable deposits, piped with open-jointed pipes, covered by filter material and carefully backfilled with suitable material deposited and compacted in layers. The object at all times is to minimize the possibility of differential settlements occurring between the line of the abandoned watercourse and the surrounding ground.

If, as a result of the works, disturbance and disruption occur to existing agricultural drainage, appropriate action should be taken to restore or replace the affected drains. See also **7.4.2**.

8.2.5 ***Underground cavities***

8.2.5.1 *Natural cavities*. These cavities are usually the result of flow of underground water in soluble rocks. They may be dry or they may form underground watercourses. In the latter case they can present special problems since measures taken to fill them could interfere with the ground water flow and cause serious trouble some distance from the site. An example is when the underground watercourse forms a supply to a community.

8.2.5.2 *Man-made cavities*. These cavities have been formed in a variety of ways and may be documented, as in the case of modern mining, or be unrecorded. Identification of their position and extent is usually done by a systematic grid of boreholes or trial pits or by geophysical methods. Enquiries amongst local inhabitants can often be helpful and reference should also be made to any old records such as early ordnance sheets the maps and memoirs of the Institute of Geological Sciences and, in the case of coal, records of the National Coal Board. Aerial photographs should be carefully checked for signs of underground workings.

8.2.5.3 *Treatment*. The treatment of cavities depends upon circumstances and it may be necessary to seek expert opinion. Where the cavities are permanently dry the treatment depends on the location of the works relative to the situation and size of the cavities. The location, identification and treatment of underground cavities are detailed in [47].

Foundations, walls of cellars, cesspools, and similar underground structures within one metre of formation level are normally removed and those at a greater depth cleared of material and backfilled with suitable granular fill material or by grouting methods.

Where the backfilled areas are allocated for later building development, the fill should consist of granular material placed and compacted in layers. Demolition rubble containing masses of brickwork, random lengths of timber and plasterboard is unsuitable for filling and, where used, is likely to be a cause of continuing subsidence. A record should be kept of the location of cellars, pits etc. so that future buildings and services can be sited, whenever possible, clear of these underground hazards.

The treatment of deep mineshafts and wells depends on the nature of the backfill, if any, and the depth to bedrock or firm strata surrounding the shaft. Regardless of whether the shaft is backfilled, it is normal to cap or plug the shaft with a well founded reinforced concrete slab, of suitable proportions to carry any load which is to be imposed upon it without relying on support from the backfill. Liaison with the National Coal Board is mandatory in the case of old coal mine shafts.

Where the extent of cavities is not readily determined by drilling or other means, pressure grouting or sand stowing may be adopted, injections should be made in a pattern of grout holes covering the area suspected to contain the cavities.

8.2.6 *Treatment of subsoils*. After completion of the foregoing operations and the removal of any unsuitable material it is advisable where practicable to compact the ground with suitable plant in order to minimize the amount of any subsequent settlement.

8.3 Excavation

8.3.1 *General*. Clause **10** of this code describes the many types of plant which may be employed in excavation. Their suitability for a particular site depends on several factors of which the most important are the nature of the materials to be excavated, the prevailing weather conditions, and the type of transport to be used.

8.3.2 *In cuttings or borrow pit areas*. Unsuitable material excavated in cuttings is normally disposed of in off-site tips or in landscape or long term reclamation works. Off-site tips should be located adjacent to the works in order to minimize haul distances and taxation of vehicles. The provision of landscape areas within the site of the works in which any of the unsuitable material can be placed is obviously both desirable and economical (see **8.1.1**). Care should be taken in the creation of all off-site tips or landscape areas to ensure adequate stability and draingage of the newly contoured areas in relation to their surroundings.

Any unsuitable material from borrow pit workings may be stacked and later used in the reclamation of the area. Here again care should be taken to ensure stability and adequate drainage. Raising the ground level around living trees should be avoided.

8.3.3 *Below water*. When excavation has to be carried out below water level draglines or grabs are generally used. Deep or wide areas of water may necessitate the use of floating dredgers or conventional excavating equipment mounted on a pontoon.

Sheet piling, ground water control, cofferdams, caissons with pumps and other special techniques are frequently employed to enable work which would normally be carried out below water to be achieved in dry conditions, when the ease of excavation may achieve savings in cost which outweigh the cost of these methods of excluding the water.

Working over or adjacent to water involves particular hazards for operatives.

8.3.4 *In rock*. Methods of breaking, loosening and excavating rock and other hard materials vary according to the conditions prevailing, the quantity and hardness, and the equipment available. The rock has to be broken down to a size suitable for excavation, transportation, deposition and possible compaction. It may be necessary to carry out full scale excavation trials if the excavated material is to be produced to a suitable grading, e.g. a free draining material or one which is well graded [44].

Clause **10** details types of plant suitable for excavating in rock after it has been treated by either blasting or mechanical techniques. Blasting methods are described in clause **16** of this code.

A ripper/rooter may be used to break up heavily jointed or weak rocks and large pieces may be broken down with a heavy weight. Where ripping techniques are employed a limiting factor may be the ability of the machine tracks to grip the rock surface.

Where ripping is impracticable, and particularly in trenches and other restricted areas, the rock can be broken and loosened in one of the following ways:

a) by the use of pneumatic, hydraulic or mechanical paving breakers;

b) quarrying it out by hand using rock wedges and hammers, steel bars, picks etc., and breaking with rock hammers;

c) drilling suitable holes with pneumatic drills and then breaking up the rocks with plugs and feathers with limited blasting and splitting devices, or hydraulic bursters.

8.4 Earth-moving. Earth-moving is the removal of excavated material from the point of excavation to a permanent or temporary point of deposition. Unnecessary rehandling of material adds to the cost of this operation but it is not always possible to deal with excavated material in one operation.

The excavated material should be moved as quickly and economically as possible to its final destination, which may be on or off the construction site.

The method of earth-moving should take into account:

a) the alternative distances and routes between the points of excavation and deposition;

b) the type and quantity of material to be moved along each of the required routes;

c) the economics of using existing roads, constructing specific haul roads, or merely using existing surfaces;

d) the availability of the most appropriate earth-moving equipment;

e) the weather conditions which may affect the condition of the haulage roads [48];

f) any environmental considerations, such as noise, which may limit the selection of plant or the route it may follow.

The movement of earth-moving plant across the site has an influence on the sequence and method of construction of other works on that site. It can affect the condition and stability of the ground over which the plant travels, and the routes should be designed to take account of this. Some of the plant commonly used on construction sites is forbidden to travel along public highways.

The excavation of an area of cut should be so timed that the formation level is not exposed to the deteriorating influence of the weather for longer than can be avoided. Exposed soils in the formation can be adversely affected by swelling due to rain or frost action, or by cracking due to heat or drying winds. Relief of overburden pressure caused by excavation may result in ground heave. With certain soils this may lead to fissuring, thus permitting easy access of water and consequent softening of the soil.

Formations should, therefore, be protected as soon as possible after they are exposed. This protection may take the form of concrete blinding where the following work will be of reinforced concrete construction, a surface dressing with a sealer such as bitumen or tar, or covering with a sub-base material in the case of road works. It is desirable, with both cuttings and embankments, that bulk earth-moving be carried out to a level slightly higher than the finished formation level and that the remaining material be removed at the last possible moment.

8.5 Deposition and spreading. The method of deposition and spreading depends upon the available plant and the means used to transport the fill material.

The most satisfactory way in which to form embankments or other deep areas of fill is by depositing the material in layers, spreading it as evenly as practicable and compacting it. The thickness of these horizontal layers depends on the type of material and its behaviour under compaction and on the compacting equipment available (see clause **9** and **10.6**).

Suitable material should be used as soon after excavation as possible so as to avoid moisture changes due to the weather. It is generally impracticable and uneconomical to adjust the moisture content of soils during earth-moving operations other than on a relatively small scale and in favourable conditions.

The amount of tolerable settlement of a compacted fill depends upon the intended utilization of the fill area. For example, on landscape areas a large degree of settlement could be accepted providing it does not adversely affect drainage or the stability of the fill. Materials should be selected so as to ensure that filling is used most effectively to take account of its varying strength and deformation characteristics, e.g. high strength materials should be placed where the stresses are highest. In some instances it may be possible to improve the mechanical properties of soils by the admixture of more stable granular material or the mixing into the soil of stabilizing cement, lime or similar materials. Such procedures should be evaluated by laboratory testing and field trials.

Site conditions may necessitate stockpiling material for subsequent use. This requires the introduction of additional handling operations of excavation, transportation, deposition and compaction. There are a number of cases where this double handling may be cheaper than the disposal of excavated material and its subsequent replacement with imported fill, for example, where the fill area is inaccessible due to the construction of a bridge or where selection of excavated material is required in particular areas. Allowance should be made for the possible deterioration of the stockpiled material due to weathering.

Care is necessary in siting stockpiles in accessible areas on or off site so as to reduce the amount of further work to a minimum. Precautions should be taken to ensure stability, taking into account the strength of the underlying ground and the effect on drainage. It may be necessary to carry out partial compaction of the stockpile and provide protection to maintain the quality of the material.

8.6 Control

8.6.1 *Preliminary trials.* Where there is no previous experience to guide the selection of equipment and construction methods for particular soils, it is desirable to carry out compaction trials on the different types of soil which may be incorporated in the fill using varying layer depths and compaction plant to confirm or establish an economical method of work. These trials should also determine the suitability of the proposed filling materials for use in different weather conditions and take into account the likely variability of these filling materials as they may arise in practice (see **7.2.1**).

8.6.2 *Methods of monitoring deformations and stresses.* It may be necessary to monitor the settlement and stability of both the fill and the foundation during and after construction.

The extent and the method of monitoring depend on the nature of the ground, the significance of the operations, and the accuracy required. This may range from simply levelling at regular intervals of time a series of points on the fill and thereby assessing when settlement has effectively ceased or become acceptable, to a full range of instruments installed to measure accurately the pressures and deformations occurring in the sub-soil and embankment. Steel pins set in concrete blocks make suitable reference points for simple levelling (see **6.6**).

Observations of settlement, lateral displacement and dissipation of pore water pressure during construction may be used to govern the rate of filling, to ensure continued stability and also to assess the accuracy of the predictions (based on conventional laboratory tests) which have been used in the design. Stability charts on which the measured deformations and pressures are marked can be prepared for use in site control.

The following instruments are commonly used:

- piezometers to measure pore water pressures;
- settlement gauges to measure the settlement;
- inclinometers to measure lateral movement (**5.1.3.5**);
- earth-pressure cells to measure total pressure.

Computer programs are available to analyse stability and settlement at all stages of filling and to produce rapid translation of instrument readings into safety factors for use in the control charts.

8.6.3 *Earthworks adjacent to structures*. Earthworks operations adjacent to structures are frequently carried out separately from the main earthworks operations and can be considered in the following categories:

a) filling over large pipes and culverts of either concrete or metal construction. In these cases it is important that fill is brought up equally on each side of the structure to prevent unbalanced loading and that great care is taken with the first layers of fill over the top of the structure;

b) against abutment and wing walls of bridges and retaining walls of all kinds;

c) around and between skeleton abutments, buried piers and bank seats.

Because satisfactory compaction of fill adjacent to structures is often more difficult to achieve owing to the restricted nature of the operation, it is usual practice to specify special types of fill, such as selected granular materials or pulverized fuel ash, in the immediate area of the structure. Satisfactory compaction to reduce to a minimum differential settlement between backfill and structure is important enough to warrant the additional expense. Both the type of compaction plant and the method of compaction may have to be modified from those used in general embankment construction. The possible development of horizontal forces on foundations or on piles should be considered.

8.6.4 *Shallow cut and fill*. In general most types of soil, when in suitable condition, can be used for fill up to about 5 m in height to carry superimposed loads, provided due care is taken with the foundations, compaction and side slopes. In the cases of fills of the order of 2 m in height special precautions may be necessary for the following reasons.

a) It is very often the case that the zone of soil immediately below the topsoil is at a lower shear strength than the underlying subsoil. It is also likely to have a varying moisture content which will influence its behaviour under operational conditions.

b) In the larger embankments a weaker material can either be placed at locations where it is not detrimental to stability or be mixed with better material. In the case of shallow cut and fill this may not be possible and problems can arise with the use of such material.

c) Special care has to be taken in the selection of earth-moving and compaction plant and in the organization of the deposition and compaction of the fill in order not to overstress weaker subsoil.

It is essential not to disturb or damage any existing field drains or drains incorporated in the fill. These may require protection, or interception by a collector drain (see **6.5.4.1** and **7.4.2**), particularly if the fill is to be used as a construction road before completing the construction of the permanent pavement or other works.

Consideration should be given to adopting a minimum height of fill to avoid differential movements due to the causes described in **8.6.4**.

8.6.5 *Earthworks adjacent to properties*. Care has to be exercised when carrying out any works adjacent to existing properties to minimize damage and nuisance. Particular problems associated with earthworks are dust, vibration and noise.

a) *Dust*. In dry periods water spraying should be employed to keep down dust. Excessive watering, causing slippery conditions on haul roads, should be avoided.

b) *Vibration*. Vibration due to compaction or heavy plant may cause damage to adjacent property and should, therefore, be avoided by using static compaction and suitable earth-moving plant.

c) *Noise*. Appropriate measures should be taken to comply with any statutory or local requirements concerning noise emission.

To avoid litigation with the owners and tenants of properties adjacent to earthworks it is recommended that they are informed and every effort is made to minimize damage and nuisance. Whenever possible work should be confined to normal working hours and night-time and weekend working avoided.

Surveys of all properties likely to be affected should be carried out before work starts and records agreed between the property owners, the contractors and the employers. In particular, any existing cracks or deformations should be recorded.

8.7 Inclement weather during construction

8.7.1 *Wet weather*. Areas of cut and fill should always be formed with a sufficient crossfall and surface smoothness to shed surface water which could otherwise accumulate from rainfall or from surrounding areas. Where necessary, trenches, either temporary or permanent, should be formed to drain the water from the earthworks. This is especially important in road and airfield construction for which drains should be formed at the earliest possible time and so arranged to maintain the formation or subgrade free from water. Continuous compaction during filling operations produces a surface which discharges the bulk of this water.

In excavating for cuttings, it is advisable to maintain a longitudinal fall and provide an outlet for surface water.

The construction of a drainage channel to take surface water clear of the excavation during the progress of the work may be advantageous. Grips or ditches should be cut in the bottom of the excavation to drain into the drainage channel or the sump and these should be kept clear at all stages of the work. Where during the construction of a cutting existing drains or water courses are interfered with, means should be adopted for conveying the water to the permanent drains in the cutting. Alternatively, longitudinal drains may be constructed at the top of the cutting where they are not detrimental to the long term stability of the slope. The excavation should be carried out so that water will not be trapped but will easily get to the drainage channels provided.

Rainfall may so affect the exposed surface of cohesive soils as to result in interruptions both in the use of excavating plant and in the transport of materials. Road and airfield works, with their extensive use of pneumatic-tyred plant over shallow cuts and fills, are particularly affected by intermittent wet periods.

8.7.2 *Freezing conditions*. It may be difficult or uneconomical to excavate soil in periods of severe frost or to achieve satisfactory compaction of filled earth. However, earthworks can be carried out in frosty weather provided that material which is frozen is set aside in both the cut and fill areas. In the case of rock fill and coarse grained granular materials, frost has little effect and should not unduly interfere with the progress of work. Those areas of fill which have been subjected to frost action should be recompacted before further materials are placed. Increase in volume and consequent loss of strength caused by freezing pore water evidenced as frost heave are most troublesome in chalk, silt, silty clay and fine sands.

8.8 *Haul roads*. Haul roads should be constructed to provide all weather access to the works for construction plant. They should be so constructed as to balance the initial construction cost against the cost of maintaining a standard of riding quality suitable for the plant using them. They may thus vary from the graded earth road to a metalled surface.

The maintenance of a uniform running surface on a haul road is essential to cut down mechanical wear and tear and enable full advantage to be taken of the maximum travel speed of the earth-moving equipment. In the case of an earth road, regular trimming by means of a grader is necessary to prevent deterioration under adverse conditions.

Dust clouds may be a hazard where haul roads are used by general site traffic. Dusting and deterioration of surfaces can be minimized by frequent spraying with water or by surfacing with a bitumen spray coat or a fabric.

Where an existing highway is crossed by the haul road it may be necessary to strengthen the highway to carry the construction traffic.

8.9 *Advanced earthworks*. Where an embankment is to be constructed over soft or otherwise unsuitable foundation soils it may be possible to avoid removal of these materials if the embankment can be constructed well in advance of other earthworks, providing that access onto the land can be arranged. This either causes displacement of underlying soft soil or induces consolidation, each of which reduces the settlement taking place after construction (see **7.3.3**).

8.10 *Stage construction*. An embankment constructed on a moderately weak foundation soil may, if stability is questionable, be built in stages.

Piezometers may be installed in an embankment to monitor the dissipation of excess pore pressures, thereby providing a means of controlling the rate of fill placement. Alternatively, if no monitored control over the rate of fill placement is to be exercised, a calculated waiting period between successive layers of fill should be incorporated in the earthworks programme to allow for dissipation of pore water pressure.

8.11 *The disposal of spoil*. On any large project the disposal of spoil forms an important part of the earthworks programme. Normally the spoil consists of materials which, owing to their physical characteristics, cannot be incorporated into the embankments, although occasionally it may derive from an excess of cut over fill requirement. For economic reasons, tips should be located close to the areas of cutting so as to minimize haul distance. Discussions should be held with the occupiers of nearby land, as it may be possible to use surplus spoil to improve their fields by levelling hollows. In the case of topsoil, which is usually needed for cutting and embankment side slopes or reinstatement of farm land, the contractor may have to arrange for areas of land outside the area of the works to be made available for topsoil tips. When the soil tip has been removed it may be necessary to aerate the existing topsoil and if demanded add to it topsoil from the site of the works. The opportunity should also be taken to utilize spoil for landscaping purposes or reducing embankment slopes.

If a spoil tip is to be permanent, licensing and planning permission is normally required. All interested parties should therefore be consulted. If delays or extensive modifications to the original plan are to be avoided, schemes for tips should be presented carefully and in detail.

In rural areas land for tips should be sought in locations which are currently underproductive so that agricultural benefit is eventually derived by the deposition of the spoil. Tipping can be used to restore previously derelict land, for example old quarries. It can also be used to fill up old excavations or to raise the level of low lying areas with a view to future development. Where no such easy solution exists, ground contours should be so planned that agricultural development can continue after tipping ceases and the land has been resoiled to the appropriate depth.

Where spoil is deposited on tips which will be used for building development the material should, where practicable, be compacted in layers as described in clause **9**.

Spoil containing a substantial proportion of combustible material should be so placed as to avoid future underground fires. The infilling below water level of quarries, rivers, and canal basins or any other available locations with a permanently high water table is possible, subject to soluble products not polluting the surrounding water. For the majority of cases however, spoil is deposited above the water table and in order to reduce the risk of combustion minimum air voids should be achieved.

Any potentially combustible materials should be placed and compacted in layers between impervious incombustible materials. The combustible layer should not exceed 1 m in thickness. The top should be covered at the end of each day's work with 500 mm minimum of non-combustible material (cohesive or fine granular) to prevent fires on the surface igniting the mass below. Slopes should be similarly covered with 300 mm minimum of fine sand or crushed stone sand to control fires and air access.

Before final restoration takes place tips should be provided with adequate temporary drainage, so that surface water does not affect adjacent land or pollute water courses, either chemically or by the action of suspended solids. Dust and noise from earth-moving plant should be kept to a minimum consistent with the works taking place, so that their impact on the environment is not too severe. Restrictions placed upon the work by the planning authorities should be strictly observed.

Disposal of spoil off the site should be considered as part of the design and arrangements should be made before construction commences. It is particularly important to ensure in the siting of spoil tips that landslides or other movements are not initiated or reactivated by the placing of the spoil, and that existing surface and subsoil drainage systems are not adversely affected.

9 Compaction

9.1 General. Soil compaction is the process whereby soil particles are constrained to pack more closely together through a reduction in the air voids by rolling or other mechanical methods. The engineering properties of soils and rocks used in filling, e.g. their shear strength, consolidation characteristics, permeability, etc., are related to the amount of compaction they have received.

A high degree of compaction assists in:

a) reducing the cost of subsequent maintenance;

b) reducing the risk of slips;

c) permitting permanent structures such as roads or buildings to be constructed without delay;

d) permitting the use of higher bearing pressures in the design of foundations for permanent structures.

The increase in the dry density of soil produced by compaction depends mainly on the moisture content of the soil and on the amount of compaction applied. With a given amount of compaction, there exists for most soils a moisture content termed "the optimum moisture content" at which a maximum dry density is obtained.

The degree of compaction necessary is determined by the engineering properties required for the fill to carry out its design function. It can be specified in terms of any of the following properties of the compacted material:

e) minimum dry density;

f) maximum air voids associated with a maximum moisture content;

g) minimum percentage of the maximum dry density obtained from a standard laboratory test;

h) minimum shear strength.

Alternatively, if the properties of the materials to be used in the fill have previously been related to the compactive effect of the various types of available plant, the degree of compaction may be controlled by specifying the thickness of layers and the number of passes of specific categories of plant.

9.2 Test for compaction of earthworks. The tests which are normally carried out in connection with earthworks compaction are detailed in BS 1377. They enable shear strength and density/moisture content relationship both in the field and in the laboratory to be established.

9.3 Methods of compaction. The objective of the operation is to achieve the required degree of compaction by the most economical means.

The method of compaction employed depends upon:

a) soil type, including its grading and moisture content at the time of compaction;

b) specified compaction requirements;

c) total quantity of material and rate at which it is to be compacted;

d) geometry of proposed earthworks;

e) environmental restrictions (e.g. noise).

Earth-moving plant is continuously being developed and improved. The various types of compaction plant which are commonly available are described in clause **10** of this code and a guide to their suitability for different soil types is given in Table 4.

To establish the method of compaction to be adopted for a site, field compaction trials, should, where appropriate, be conducted to determine the most suitable compaction equipment for the conditions which are likely to pertain during the construction period, taking into account:

f) soil moisture content;

g) thickness of layer;

h) number of passes;

i) variability in the grading of the material.

Economic considerations should be taken into account. For example, it may have to be decided whether it is more economical to compact in 100 mm layers with a light roller or in 300 mm layers with a heavy roller. Again, consideration may have to be given to the number of passes required with smooth or tamping rollers to produce the specified end product.

A combination of two or more types of equipment may give the best results.

9.4 Soil compaction characteristics

9.4.1 *General*. For compaction purposes, soils can be classified into the groups described in **9.4.2** to **9.4.5**.

9.4.2 *Rockfill*. Rockfill should be compacted in thick layers with the maximum diameter of the rock fragments not exceeding two thirds of the layer thickness. The material obtained from excavations in sound rock is normally spread by heavy crawler tractors in layers up to 1.5 m deep. This process is beneficial in that it re-mixes the material should segregation have taken place during haulage, and provides a relatively flat and even surface for the compaction plant to work on. Vibration techniques give the best results in the compaction of rockfill and a heavy vibrating roller with 10 tonnes to 15 tonnes static weight normally compacts rockfill efficiently in layers up to 1.5 m thick.

It is desirable to place rockfill obtained from excavations in strong or moderately strong rock with as high a moisture content as possible in order to prevent collapse settlement due to subsequent saturation of the material. To this end the material may be sluiced with water after placing and before compaction.

Fill material obtained from a weak parent rock can degrade rapidly by compaction and problems can arise if the material has a moisture content sufficiently high to produce excess pore pressure causing local instability to develop during the compaction operation. It is therefore important that the standard of compaction to be adopted avoids the creation of these conditions. Generally, layer thickness should not exceed 500 mm.

9.4.3 *Granular soils*. Granular soils are generally defined as non-cohesive or coarse soils with high permeability containing a small percentage of fines (say less than 10 smaller than 0.06 mm). The percentage of fines which can be accepted depends on whether they are cohesive or non-cohesive. Under pressure from compacting plant, water can be forced out and a high degree of compaction can be achieved even if the material has initially a high moisture content.

In the compacted state, granular soils possess high load-bearing capacity and are not usually susceptible to frost action unless they contain a high proportion of fines. They are, therefore, to be preferred for fill purposes and are generally easy to compact. However, where the material contains an appreciable quantity of fines, high pore water pressures can develop during compaction if the moisture content of the material is high, resulting in lack of stability. With uniformly graded granular material it is difficult to achieve a high degree of compaction near to the surface of the fill, particularly when vibrating rollers are used. This problem is normally resolved in that when the next superimposed layer is compacted the loose surface of the lower layer is also compacted satisfactorily.

Improved compaction of uniformly graded granular material can be achieved by maintaining as high a moisture content as possible by intensive watering and by making the final passes at a higher speed using a non-vibratory smooth wheel roller or grid roller.

9.4.4 ***Cohesive fine soils***

9.4.4.1 *Silts*. The moisture content has a great influence on both the strength and compaction characteristics of silty soils. In particular, an increase in moisture content of as little as 1 % or 2 %, combined with disturbance created by spreading and compaction, can result in a very significant reduction in shear strength, making the material intractable and impossible to compact.

9.4.4.2 *Clays*. The compaction characteristics of clay are highly dependent on moisture content in that a greater compactive effort is required as the moisture content reduces. It may be necessary to adopt thinner layers and more passes by heavier compaction plant than required for granular materials. Where clays are compacted at the upper limit of moisture content feasible for compaction, there is a danger of instability as a result of excess pore water pressures caused by such compaction.

Where cohesive soils are excavated in large lumps the material should be broken down by tamping or grid rollers after spreading, to bring it to a size suitable for obtaining the required compacted density.

9.4.5 *Special fills — waste materials*. Such materials are by their nature extremely variable and should always be subject to site trials to determine the compaction specification.

9.5 Control of compaction in the field. The following operations, which influence the degree of compaction, should be monitored:

a) forming the correct depth of layer for each material type;

b) segregation of materials where different compaction required;

c) routeing of earth-moving plant to avoid uneven compaction of any area of the fill;

d) correct number of passes being given to each layer of fill by compaction plant;

e) filling being placed correctly to enable full compaction of the edge of the embankment to be achieved;

f) correct operation of compaction plant, particularly vibratory plant;

g) maintenance of a free-draining surface during fill placement.

In addition, the moisture content and the density and/or shear strength of the resulting earthworks should be periodically checked.

9.6 Compaction adjacent to structures

9.6.1 *Culverts*. Filling should be brought up and compacted uniformly on both sides of culverts to avoid unequal loading of the structure. Filling over the top should be carried out under close supervision to avoid unacceptable point loadings due to construction plant or fill containing material which may damage the structure.

The method and means of compaction should be approved in relation to the design and construction sequence of the culvert. Certain types of culverts, e.g. those formed of corrugated metal plates, require the use of selected fill materials immediately adjacent to the structure.

9.6.2 *Abutments and retaining walls*. One of the most frequent maintenance problems which occurs in connection with embankments is due to the differential settlement of fill at the rear of abutment and wing walls. It is therefore essential that the material placed behind structures is of a type that can be compacted to such a degree that only insignificant differential settlement occurs subsequently. This frequently necessitates the exclusion of naturally arising cohesive materials and certain weak rocks such as chalk. In these circumstances, it is necessary to import selected granular material or pulverized fuel ash. Where the embankment adjacent to the structure has already been constructed it is essential that when filling the space in between, steps are cut into the face of the existing fill to enable full compaction in horizontal layers to be achieved over the infill area.

Because this area is usually restricted in space and irregular in shape, small hand-operated compaction plant has to be used and it is important to ensure that the thickness of layers of fill is not too great for the compactive ability of the plant being employed (see also **8.6.3**).

9.6.3 *Obstructions*. These can take the form of manhole shafts, piers for bridges, columns etc., or temporary obstructions such as poles or towers supporting overhead cables or settlement monitoring equipment.

In order to obtain the required compaction around the obstruction, it is usually necessary to employ suitable supplementary compaction plant which is able to operate efficiently in these conditions without damage to the obstruction.

10 The operation of construction plant for earthworks

10.1 Classification of plant. The construction plant employed for earthworks is designed to perform one or more of the following operations:

a) breaking up the soil or rock into suitable condition for it to be excavated;

b) removing the soil from the excavation and depositing it directly into spoil heaps or onto transporting equipment;

c) handling and transporting the excavated material to the point of disposal or the area for filling;

d) levelling of deposited materials in fill areas;

e) compaction of materials;

f) maintenance of haul roads;

g) ground water control.

Earth-moving comprises any process of excavation and disposal of excavated materials. In many situations the excavated material is divided into materials suitable for use as backfill or for compaction in fill areas, and unsuitable materials which have to be disposed of in some way or other. Material to be disposed of is, in most cases, transported directly to its final position, whether this is on or off site.

Construction plant is an expensive investment requiring a high rate of utilization. Most smaller plant items for excavation comprise a basic excavating machine which can be equipped with a variety of attachments enabling the prime mover to be employed as near continuously as possible. Such equipment is likely to be employed all the year round.

By contrast, the larger pieces of excavation plant can only be used economically on large projects. It is essential that plant operators and drivers are fully trained and competent to carry out their duties in accordance with the Construction Regulations. The various types of plant are described in Appendix A in relation to the conditions and soils most appropriate to their use.

10.2 Particular factors affecting earth-moving plant. In order to make the appropriate decision regarding the excavation plant and the associated equipment for transportation and compaction, the total quantity of material requiring to be excavated should be considered in terms of the following:

a) quantities arising from individual excavations and the times at which these excavations can be carried out;

b) the individual destinations for the material from each excavation;

c) the haulage routes and the likely variation in condition of each haul road from the point of excavation to the point of deposition;

d) the nature of the material, whether it is in a loose or dense state, whether it is weather-susceptible and whether it will be necessary to lower the water table in order to excavate it;

e) the feasibility of excavating material by equipment standing or running across the area, or only by equipment on prepared foundations or on stable soil at ground level;

f) the suitability of the material for rubber-tyred plant (the ground may be abrasive to tyres or so soft that only tracked plant can operate);

g) the depth of the excavation, access into the excavation and methods of support, if any, around the perimeter of the excavation.

10.3 Preparation for excavation. The extent of the work should be assessed, the perimeter details decided and then the excavation area and perimeter set out. Reference pegs and profiles should be established clear of the excavations and associated works so that the position of the operation is never in doubt, and in such a way that the operators of the plant and those in attendance on them can readily assess the formation level and other details of the shape to be achieved. Account should be taken of any particular requirements regarding the formation, whether it should be initially excavated to a higher level to offer protection as long as possible, or whether care is necessary in trimming to avoid excessive overbreak. It may be necessary to carry down the setting out profiles from ground levels to near formation level so that the necessary accuracy can be readily achieved.

The term "excavating plant" does not include rock-breaking plant. Rock excavation requires preparation by drilling and blasting or, if the rock is weak, by the use of rooters, rippers, or scarifiers which break the material down to a size that can be handled by the excavating plant.

Table 4 — Typical compaction characteristics for natural soils, rocks and artificial materials used in earthwork construction

The information in this table should be taken only as a general guide. When the material performance cannot be predicted, it may be established by earthwork trials.

This table is applicable only to fill placed and compacted in layers. It is not applicable to deep compaction of materials in-situ (see **10.6**).

Material	Major divisions	Subgroups	Suitable type of compaction plant	Minimum number of passes for satisfactory compaction	Maximum thickness of compacted layer (mm)	Remarks
Rock-like materials	Natural rocks	All rock fill (except chalk)	Heavy vibratory roller not less than 180 kg per 100 mm of roll Grid roller not less than 800 kg per 100 mm of roll Self-propelled tamping rollers	4 to 12	500 to 1 500 depending on plant used	If well graded or easily broken down then this can be classified as a coarse grained soil for the purpose of compaction. The maximum diameter of the rock fragments should not exceed two-thirds of the layer thickness.
		Chalk	See remarks	3	500	This material can be very sensitive to weight and operation of compacting and spreading plant. Less compactive effort is needed than with other rocks.
Artificial	Waste material	Burnt and unburnt colliery shale	Vibratory roller Smooth wheeled roller Self-propelled tamping roller	4 to 12 depending on weight of plant	300	
		Pulverized fuel ash	Vibratory roller Self-propelled tamping roller Smooth wheeled roller Pneumatic tyred roller			Includes lagoon and furnace bottom ash
		Broken concrete, bricks, steelworks slag, etc.	Heavy vibratory roller Self-propelled tamping roller Smooth wheeled roller			Non-processed sulphide brick slag should be used with caution

Table 4 — Typical compaction characteristics for natural soils, rocks and artificial materials used in earthwork construction

Material	Major divisions	Subgroups	Suitable type of compaction plant	Minimum number of passes for satisfactory compaction	Maximum thickness of compacted layer (mm)	Remarks
Coarse soils	Gravel sand gravelly soils	Well graded gravel and gravel/sand mixtures; little or no fines Well graded gravel/sand mixtures with excellent clay binder Uniform gravel; little or no fines Poorly graded gravel and gravel/sand mixtures; little or no fines Gravel with excess fines, silty gravel, clayey gravel, poorly graded gravel/sand/clay mixtures	Grid roller over 540 kg per 100 mm of roll Pneumatic tyred over 2 000 kg per wheel Vibratory plate compactor over 1 100 kg/m^2 of baseplate Smooth wheeled roller Vibratory roller Vibro-rammer Self-propelled tamping roller	3 to 12 depending on type of plant	75 to 275 depending on type of plant	
	Sands and sandy soils	Well graded sands and gravelly sands; little or no fines Well graded sands with excellent clay binder				
	Uniform sands and gravels	Uniform gravels; little or no fines Uniform sands; little or no fines Poorly graded sands; little or no fines Sands with fines, silty sands, clayey sands poorly granded sand/clay mixtures	Smooth wheeled roller below 500 kg per 100 mm of roll Grid roller below 540 kg per 100 mm of roll Pneumatic tyred roller below 1 500 kg per wheel Vibratory roller Vibrating plate compactor Vibro-tamper	3 to 16 depending on type of plant	75 to 300 depending on type of plant	

Table 4 — Typical compaction characteristics for natural soils, rocks and artificial materials used in earthwork construction

Material	Major divisions	Subgroups	Suitable type of compaction plant	Minimum number of passes for satisfactory compaction	Maximum thickness of compacted layer (mm)	Remarks
Fine soils	Soils having low plasticity	Silts (inorganic) and very fine sands, rock flour silty or clayey fine sands with slight plasticity Clayey silts (inorganic) Organic silts of low plasticity	Sheepsfoot roller Smooth wheeled roller Pneumatic tyred roller Vibratory roller over 70 kg per 100 mm of roll Vibratory plate compactor over 1 400 kg/m^2 of base plate Vibro-tamper Power rammer	4 to 8 depending on type of plant	100 to 450 depending on type of plant	If moisture content is low it may be preferable to use a vibratory roller Sheepsfoot rollers are best suited to soils at a moisture content below their plastic limit
	Soils having medium plasticity	Silty and sandy clays (inorganic) of medium plasticity Clays (inorganic) of medium plasticity				
		Organic clays of medium plasticity				Generally unsuitable for earthworks
	Soils having high plasticity	Micaceous of diatomaceous fine sandy and silty soils, plastic silts Clay (inorganic) of high plasticity, “fat” clays				Should only be used when circumstances are favourable
		Organic clays of high plasticity				Should not be used for earthworks

NOTE If earthworks trials are carried out, the number of field density tests on the compacted material should be related to the variability of the soils and the standard deviation of the results obtained. Compaction of mixed soils should be based on that subgroup requiring most compactive effort.

Bearing in mind the factors listed in **10.2**, the excavation plant is selected, with the advice of the plant manufacturer where appropriate, from the equipment available so as to provide the most economical solution for digging, hauling and placing. Any particular type of plant may appear more attractive if it reduces the number of individual operations. The output of the excavation plant should be matched to the ability of the transporting machines or other equipment to remove the excavated material to the point of deposition. An economical balance should be found between the numbers of excavating units, the numbers of transporting and compaction units and the overheads associated with the total time for the excavation and filling operation.

In order that such plant, which is selected in the hope of continuous operation, may proceed with a minimum of interruption, it is necessary to take steps to avoid stoppages. These stoppages may occur because of the presence of obstructions in the ground, usually at higher level, or the accumulation of water. Possible obstructions include tree stumps and heavy roots, foundations of buildings or other structures which have been demolished, large boulders, and waste deposits. It is usually preferable to take steps before commencing the main excavation to remove these obstructions so that the plant may proceed without hindrance.

Trees can be removed by mechanical equipment specifically designed for the purpose or by bulldozers fitted with special attachments for dragging out roots. Alternatively, they may be felled with power-operated saws and the stumps tackled separately. It is most common to cut trees to a height not more than 1 m above the ground and to remove the stumps by grubbing out with winch gear, by the use of bulldozers, or by the use of high velocity explosives (see clause **16**).

Small boulders may well be disposed of by the excavating plant it is intended to use. Larger boulders may need to be broken down to a size which enables them to be handled by the excavating plant and then directly incorporated in the area within which they are to be deposited. Occasionally, large boulders may be removed to one side of the works and used as part of a landscape feature. When boulders have to be broken down, one of the following methods may be used:

a) *drilling required:* cartridge explosives, plugs and feathers, hydraulic and gas, burster cartridges;

b) *drilling not required:* blaster charges, boom-mounted hydraulic hammer, drop balls, pneumatic or hydraulic picks.

It is sometimes most convenient to break old foundations into sections which can be removed. The breaking up may be achieved by the use of pneumatic breakers, plugs and feathers, hydraulic cartridges, thermic lances or water jets. The presence of steel reinforcement in these foundations may require the use of a burning torch. Where the foundation consists of a prestressed concrete member, the advice should be sought of a specialist with a knowledge of the design of the member before it is attacked, in view of the possible release of the large amount of energy stored within the member.

10.4 Excavation and earth-moving plant. Construction plant suitable for excavation is described in Appendix A. The selection of the earth-moving plant should not be a separate choice from the selection of the excavation plant. In some instances the same plant is used for both purposes. The most commonly employed method of earth-moving is by the use of lorries, dumpers and the like. The choice is influenced in part by any need to travel along public highways and/or the need to dispose of material away from the site from which it arises. For hauls over 1 000 m, large dump truck or lorry transport are usually employed. The establishment and maintenance of good haul routes are a prime feature in the economics of the operation, affecting the round trip time and wear and tear on equipment and operatives. The economics of haul route preparation have to be considered in conjunction with the operational costs for the plant involved.

Conveyors and pipelines can be used economically in certain situations [49]. The installation cost and maintenance and replacement costs of both conveyor belts and conveyor pipes are comparatively high, particularly when abrasive or clogging materials are being moved. With granular materials and the availability of plentiful supplies of water, the use of placing methods employing pipelines can often come into consideration. Conveyor belts are particularly attractive where large quantities of material have to be shifted between two areas when haul route access between them is difficult to obtain and maintain. Where the gradient may be too severe for a conveyor, the introduction of elevator equipment should be considered.

Graders are frequently employed for the maintenance of haul routes and large areas of formation. The grader blade trims at a preset height above tyre surface level and so removes loose material which accumulates on haul routes, and by employing variable blade levels can trim the haul route or the surface of a formation to a required level or crossfall. Also associated with earth-moving operations are powered brushes or sweepers, either purpose-made or with the brushes fitted to tractors. Water bowsers with spray bar attachments may be required to control dust on exposed surfaces or haul routes in dry weather. In exceptional circumstances it may be necessary to increase the moisture content of the soil to improve compaction.

Earth-moving can also utilize transport by rail, barge or ship. When barges or ships are used the material is normally unloaded by grabs. Bottom-dumping barges are economical for placing fill in stranding water. Purpose-made rail equipment running on standard or narrow gauge track is available. Narrow gauge track and electric-battery-operated locomotives are used in tunnel excavation. Diesel-operated locomotives and side-tipping trucks can be used overground on monorail systems. Standard gauge bottom-dumping rail trucks are used to bring fine materials such as pulverized fuel ash to a site.

10.5 Compaction equipment. Compaction is the application of load cycles to soil and rock by a process of rolling tamping, and/or vibration. The extent to which compaction can be achieved is a function of the soil properties and the energy applied. Unless these are harmonized it could be difficult to achieve the compacted density with any speed and economy under practical conditions.

The earth-moving plant employed to take the excavated soil to the deposition area is capable of applying a compactive effort to the fill, provided that the plant can easily traverse it and is systematically routed. This is normally considered additional to that provided by the compaction plant.

To minimize turn round times on the area being filled the deposition of fresh material should be adequately controlled to avoid the creation of temporary soft areas, steep gradients and obstructions to maintain reasonable access. When wet weather or the use of wet fill is anticipated, appropriate measures such as self-draining crossfalls should be employed to avoid the accumulation of water on or in the fill.

The types of compaction plant in common use are described in **A.2**.

10.6 Specialist plant and equipment. Specialist equipment and techniques for ground water control by pumping, grouting and the construction of coffer-dams, methods of improving the characteristics of the ground by deep compaction and other in-situ operations are described in CP 2004.

10.7 Safety measures. The maintenance of safe conditions both for people employed on the site and for the general public, particularly children, have to be considered at all stages of designing, planning and executing earthworks. All aspects of current legislation have to be adhered to, in particular the Factories Act 1961 as amended by the Health and Safety at Work etc. Act 1974, and the associated Construction Regulations.

All earth-moving plant is potentially dangerous and the operator may often be unable to see people or obstacles close to his machine. The noise of the plant may prevent him from hearing shouted warnings. Audible reversing warnings can protect people working in the vicinity of lorries tipping their loads. When unattended, plant should always be left in a safe position (e.g. scraper bowl lowered, excavator bucket resting on the ground) and the power units immobilized.

Earthworks should be designed and supervised to be stable at all stages of construction and to constitute no risk to the construction operative or to the public (including trespassers). This may necessitate the placing of restrictions on the method of working or the provision of certain temporary works. Excavation should always be surrounded by safety barriers when a hazard exists.

All bridges and crossings, temporary or permanent, should be adequate to allow the safe passage of fully laden plant, or alternatively adequate warnings and restraints should be supplied to avoid overloading such bridges and crossings. Likewise, where plant has to pass through openings of restricted width or height, frameworks should be provided ahead of these openings to indicate the maximum acceptable plant dimensions. Within these openings suitable provision has to be made for the safety of pedestrians and well secured sleepers or similar devices installed to restrain the wheels of the plant to the required access way. Where plant is required to operate close to an existing cutting large timber baulks should be securely fixed along the crest of the slope to act as a safety barrier.

The existence of public utility apparatus, i.e. gas, water, electricity, sewers or overhead cables, and the presence of pipelines should be thoroughly checked and the protection of such apparatus and the safety of plant operatives discussed with the owners and proper preventative measures initiated to avoid accidents.

Proper stop logs should be provided to prevent lorries and dumpers from reversing into a dangerous position when tipping their loads over the edge of a tip or excavation.

No plant should be overloaded and no load disposed such that material is dropped and causes a hazard. All haul roads should be kept clean and where appropriate the wheels of plant should be washed or otherwise cleaned before travelling on a highway, in compliance with Section 127 of the Highways Act 1959.

11 Maintenance and protection of slopes

11.1 General. This clause deals with factors causing slopes to deteriorate from their designed condition and with the methods which may be adopted to prevent erosion of the surfaces. Methods of protection against wave forces in open sea conditions are not considered.

11.2 Maintenance of slopes

11.2.1 *Factors leading to instability after construction*. The factors leading to various forms of instability of cutting slopes in the short and long term are described in **6.2**. Many of these factors are also relevant to the slopes of embankments.

11.2.2 *Inspection*. Regular inspection of slopes is an essential basis for programming the frequency and extent of maintenance work. Factors to be considered in periodic inspections are described in **6.6.1**.

11.2.3 *Check and maintenance of drainage systems*. A description of drainage systems for slopes is given in **6.5.4**. Regular inspection is required to ensure that the drains are working effectively and that they are not becoming silted up or blocked as a result of pipe fractures or slope deformations. Where access to a drainage system is available through manholes, the manhole covers should be lifted at regular intervals, silt traps cleaned out and pipes examined for blockage and rodded and flushed as necessary. A watch should be kept for infiltration of soil into open or closed joint piped drains, and remedial measures taken if there are signs of appreciable internal erosion of soil into the pipes, or indeed if they are not carrying any water at all. In this latter case it may be that water is being discharged into the slope at some point where the drain is broken and this may present a threat to its stability. In addition water seeping out of the ground may also indicate a pipe fracture. The fracture should be located and repaired with as little delay as possible. Special inspection should be made at times of heavy rainfall to check whether or not any of the drains are surcharged or are carrying eroded soil.

Outfalls of drainage systems should be checked to ensure that pollution or damaging erosion of water courses is not occurring, and to check that the water courses have adequate discharge capacity for the run-off of the drainage system at times of storm. They should also be checked in freezing weather in case water is impounded in the system by icing.

Coarse backfill to drains may become clogged in time and require replacement. When this is done the insertion of a filter fabric to surround the backfill will keep clay and coarser particles out of the collector system.

11.2.4 *Check and maintenance of structures*. Retaining walls should be checked for signs of tilting or bodily forward movement and the appropriate remedial measures taken if these movements are judged to be excessive. The growth of vegetation on structures should be kept in check and growths removed if they cause damage to joints in concrete or brickwork, or blockage of the drainage systems. Weepholes should be inspected to ensure that they are working effectively and are not discharging soil as a result of erosion behind the retaining structure.

Gabions should be inspected for signs of corrosion of the metal baskets. Any discharge of water through the infilling material should be inspected for signs of soil erosion. If soil particles are being carried by the discharge water, the gabions at the point of seepage should be removed and replaced on bedding and backing material consisting of a properly designed graded filter (see CP 2004) or a porous plastics filter cloth.

11.2.5 *Check and maintenance of anchor systems.* Where maintenance of a specific amount of prestress is an essential factor in the design of temporary or permanent stressed anchors for retaining structures, a regular check should be made on stress levels in all anchors, and re-stressing carried out as required. It may be desirable in particular situations to incorporate a load cell in the anchor head so that the anchor load can be monitored continuously, with a permanent record if desired. Any re-stressing work has to be done under experienced supervision.

"Dead" anchors used as dowels or rock bolts for stabilizing rock slopes should be inspected for signs of looseness or distortion.

All stressed or "dead" anchors should be inspected for signs of corrosion at anchor heads and associated components. Removal of corrosion products and application of protective coatings should be undertaken as required.

11.2.6 *Control of development.* Control should be exercised on all forms of development on or beyond the crest of slopes. Tipping of spoil or waste material on slopes, berms, or above retaining walls should be prohibited or kept under control.

Horticultural activities such as allotment gardening should be inspected and controlled to ensure that terracing of slopes or the discharge of water does not endanger stability. Any building developments proposed for the land above the crest of a slope should take into account the possible effects of surcharge, and when necessary piled foundations should be adopted to carry the new foundation loadings below zones of critical stressing of the mass of soil or rock behind the slope.

Discharge of water from agricultural drains or surface water from roads, buildings, or other paved surfaces, should not be permitted onto the face of a cutting slope. If it is not possible for such drains to be intercepted before reaching the cutting slope then a drainage system giving adequate protection of the slope surface is required down the face. Any soakaways from surface water drainage systems should be sited in such a manner that water absorbed into the soil or rock mass behind a slope does not have a detrimental effect in the short or long term on slope stability.

Animal grazing or access by the general public should be restricted to ensure that damage to the grass cover does not lead to erosion of bare soil.

Control should be maintained on ground clearing operations at the toe of the slope to ensure that the toe is not undercut or toe drains disturbed or blocked by dumping soil onto them. A small retaining wall or kerb to define and maintain the boundary provides a measure of control.

In areas subjected to mining subsidence the cutting slopes and surrounding areas should be inspected at regular intervals to ensure that any tension cracks or pressure ridges in the soil or rock are not detrimental to stability.

11.3 Sources of erosion

11.3.1 *General.* Before adequate steps can be taken to prevent erosion, it is necessary to define the normal sources of erosion. Though not exhaustive, **11.3.2** to **11.3.5** cover the sources most commonly encountered in practice.

11.3.2 *Water.* The most common source of erosion is that resulting from water action. Rain, which constitutes the most significant eroding agent, affects the slopes of earthworks both during construction and in the finished condition and should be considered as a serious threat to stability in both cases. Heavy downpours of rain initially loosen the surface material and can thereby allow the earthworks to absorb the water into the surface, producing saturated conditions. Generally, the action only affects the outer surfaces of the earthworks and usually if these are shaped correctly the water will run off into either permanent or temporary drains. In so doing however, and depending upon the type of material in the earthworks, large washouts may occur and the arisings may cause serious hazards to adjacent property. Absorption of heavy rainfall within the body of the earthworks can increase the moisture content of the material to unacceptable limits and the subsequent seepage of this excess water from the earthworks in the long term can cause surface erosion and slips. In most cases, these slips are fairly shallow, usually being confined to the outer surfaces only, and although not structurally damaging in themselves, they are unacceptable both aesthetically and in general to the maintenance of the slope.

Streams or other water courses running along the foot of a slope may erode the toe and in times of flood may immerse part of the slope, lessening its stability. Increased runoff from the new works themselves may affect the behaviour of existing streams.

When water stands against earthworks, soft or loose material may in time be eroded by wave action and this action should be considered separately from general flood conditions. Existing surface protection may be loosened by suction action of the waves. In tidal waters the constant raising and lowering of ground water levels may cause migration of soil particles from slopes.

Erosion by solution may occur from chemical waste tips and other forms of industrial waste, particularly during the construction stage when large areas may be exposed for long periods.

11.3.3 *Frost*. As an eroding agent frost is normally considered in conjunction with water action. Alternate freezing and thawing loosens the surface of rock cuttings and opens cracks and fissures that may have been caused by the construction processes or occur naturally. Frost action has to be considered also in connection with cuts and fills in soil when surfaces previously compacted are lifted and loosened, thereby giving easier access for water, once a thaw has taken place.

11.3.4 *Wind*. As an eroding agent wind action combined with dry weather is usually at its most troublesome during the construction period, particularly in areas of excavation and filling in coarse soils. In areas of earthworks where ground conditions indicate the presence of fine granular materials, the risk of erosion should always be considered carefully and action taken during the construction period to stabilize exposed surfaces.

11.3.5 *Other sources*. Changing patterns of agriculture, particularly those leading to increased run-off, can lead to erosion. Habitual passage of pedestrians and animals along restricted routes especially at the boundaries of earthworks, also leads to erosion.

Erosion of earthworks may be accelerated or started by excavation in the completed earthworks and by the passage of construction traffic over defined routes without due regard to the care of the works. Structures produce an impervious surface from which water may flow on to the earthworks. Careless backfilling adjacent to structures, poorly compacted drain trenches and the like all cause the concentration of water to erode previously adequate earthworks. With the increasing use of artificial materials for fills, combustion either spontaneous or caused by the lighting of fires may occur which can progress internally and therefore go undetected for long periods.

11.4 Methods of protection

11.4.1 *General*. In addition to recognizing the factors causing erosion it is important to choose the method of protection best suited to the site conditions. Economic considerations should take into account the balance of costs between adopting an ideal form of protection which will prevent any erosion of a slope, and a less costly solution which will involve periodic maintenance.

Allied to the engineering and economic factors is the environmental factor in the choice of protective measures. Protection should be considered as part of the design of the earthworks, and can be used as a feature to lessen the environmental impact of the project.

In **11.4.2** to **11.4.4** methods are described which are in general use for protecting the slopes of earthworks from eroding forces.

11.4.2 *Vegetation*. The most widely used and effective form of protection is that which falls into the general category of vegetation. It is only effective in climatic conditions where the selected vegetation is able to thrive and self-propagate. Most forms of vegetation can be chosen so as not to impair the environment unnecessarily, and if correctly designed can be shown to be of positive environmentally benefit.

Of this group, a grass sward is the most generally used. It is formed by either seeding on a layer of topsoil, of thickness varying from 100 mm to 200 mm, spread on the slope, or using a nutrient mulch mixed with the seed and sprayed directly onto the natural material of the slope.

Various types of mulch are available, and specialist services exist to provide mixes designed for different seed bed materials with additives to hold the mulch to the bed. In general the type of seed chosen should be one which is locally established and which does not produce a long stemmed grass. Regular cutting of the sward is desirable to reduce weed growth, but economic or other circumstances may prevent cutting of other than noxious weeds. In some cases it is desirable, both from an environmental and from a conservation point of view, to allow freedom of development to the vegetation in order to encourage native flora and fauna. It should however be remembered that grass banks provide a useful haven for small animals, so that only controlled cutting and spraying should be carried out, unless environmental or other factors determine that the growth should remain very short.

Where surface protection is required quickly, turfing of the slope can be carried out. Generally, turfing is more expensive than sowing and seeding. After placing the turf it should be well beaten and if necessary held in position with netting. Turf placed in times of hot dry weather may shrink exposing the underlying surface. The space between the turves then acts as a channel for rain and water, so that observation of turfed areas after placing should be made and if necessary action taken either to prevent shrinkage by watering, or to fill the cracks with fine soil.

Trees provide better cover and deeper root growth than that provided by grass. In addition to their aesthetic value they may have an economic value in so far as some long term benefit may be derived and the adjacent surface need not be mown. The type of tree should be suitable for the surroundings and care should be taken that infiltration by roots does not damage drainage or services. The dangers of mature trees falling should be recognized and a suitable cutting programme initiated. In clay soils, growing trees can cause settlement of paved surfaces and structures due to extraction of moisture from the clay by roots extending beneath the paving or foundations. When trees are cut down re-absorption of water by the clay can result in heave of paved surfaces or foundations [53].

Bushes and plants provide a quick and very satisfactory ground cover in areas where topsoil and grass may be difficult to establish and maintain, and add to the aesthetic qualities of the earthworks.

11.4.3 *Drainage*. Drainage is an important method of preventing erosion. Methods of drainage are described in **6.5.4** and **7.4**.

11.4.4 *Other forms of protection*. Pitching or paving forms a very effective protection to slopes, particularly those subjected to water action. The method tends to be expensive and should be used only when no reasonable alternative exists. Care should be exercised to ensure that the toe of a pitched slope cannot easily be eroded or excess water pressures be allowed to build up behind the pitching. Pitching should comprise heavy stones well bedded with the longest face into the slope. If joints are mortared weep holes to allow escape of water should be provided. Paving has also been used extensively, either in the form of interlocking precast concrete blocks or in random stone, in areas where vegetation cannot be established; it is also used cosmetically. Wherever paving is used, the bed has at all times to be fully compacted, preferably blanketted with a weak sand/cement mixture. Paving joints should be sealed to prevent the ingress of rain water. Once the sealed surface of a paved area is broken, erosion can be rapid and remedial works prove very expensive.

In cuttings in fractured rock, the removal by hand of localized unstable areas and filling by masonry can be adopted, although it should only be considered for small areas. This procedure is sometimes referred to as "dental" work (see **6.5.5.5**).

Gabions, although used structurally as retaining walls, have wide application in the protection of slopes, particularly in preventing water erosion (see **6.5.5.2**).

Where stone filled mattresses are used to protect a slope at the junction of the protected length and the natural ground, care should be exercised to ensure that the mattresses are fixed into stable material. In order to prevent erosion of material behind the mattress a short length of gabion may be fixed at right angles to the main protective mattress. The top edge of the mattress should also be fixed into reasonably solid material to allow the mattress to rotate if some erosion does take place at the toe.

Synthetic fabrics can be used to provide rapid protection against erosion where growth is either difficult or delayed. A programme of mulch treatment of the surface of the soil should be undertaken before laying the fabric [54].

Bitumen, either hot-applied or more usually as an emulsion, can be used on slopes in loose materials to provide temporary binding of the surface until the vegetation is established.

Where industrial waste is used as fill, consideration should be given at the design stage to the environmental hazards that may arise, particularly through the leaching action of water. A clay blanket gives adequate protection although the cost of importing and separate handling of the material may well nullify the savings arising from the use of industrial waste. Alternatively additional filling to allow sacrificial erosion may be provided outside the structural limits of the embankment.

On cutting faces with a periodical tendency to weep, a blanket of granular material is an alternative to extensive herring bone drainage systems. In this case the granular fill becomes a continuous filter drain and therefore acts as a protective layer on the surface.

Wiremesh, plastics and rope netting may be used on crumbling rock faces, although this could be regarded not so much as a protection of the face but more as a protection of the adjacent land users. The use of mesh is described in **6.5.5.4**.

11.5 Remedial works after slope failure

11.5.1 *General*. Movements which occur in the course of slope failures of the type described in **6.3** frequently result in temporary restoration of stability. Thus the rotational shear slide (**6.3.2**) has the effect of achieving an overall flattening of the slope with a restoring force provided by the debris at the toe and a reduction in the disturbing force at the crest of the slope. However, in the longer term there may be renewed instability as a result of softening and slumping of the debris pile at the toe and retrogressive slipping of the steep scar face at the crest.

Consideration should be given to the possibility of tolerating the movement. This can be done only if no transportation route is blocked and no danger to life or property is involved. Before deciding that no action is required it is necessary to consider the probable future behaviour of the slope. One slip is likely sooner or later to lead to another. If this appears to present no threat to life or property or if it is considered that any subsequent movement could be dealt with safely when it occurs, the slip may best be left alone, provided that the visual appearance is acceptable.

If space is available in the cutting the most economical form of remedial work is not to attempt to restore the original slope but to adopt measures to stabilize the slipped profile by improving stability conditions at the toe and cutting back the steep scar face at the crest.

Where the slip overflows on to a road or railway, removal of material from the toe may re-activate the slip. In the short term, therefore, only a minimum of clearance should be undertaken. If space is available on the downslope slide of a cutting, consideration should be given to diverting the road or railway around the slumped material at the toe of a large-scale slip. This may well be more economical than adopting methods of supporting a restored slope.

In considering a remedial scheme after the slip it may be necessary to undertake the work in two stages, the first stage being immediate measures to re-open a road or railway or to safeguard a structure, and the second stage being long term measures to ensure reasonably permanent stability. Where no interference with other works is entailed the least expensive solution may be to acquire sufficient land to allow the slopes to be reduced to a stable angle.

11.5.2 *Immediate measures*. The first step should be to make a general appraisal of the mode of instability so that the risks of attempting to restore the original profile can be assessed. Many slips are caused by the ingress of water and this has to be controlled as soon as possible. All open cracks should be sealed to prevent water entering the slope. Surface water or shallow subsoil water should be diverted away from the slip area by open ditches or piped drains. Any water ponded in the slipped mass or flowing from the scar slope should be drained or pumped away. Temporary deeper drainage measures by pumping from timbered shafts or boreholes should be considered, including the use of exploratory boreholes as pumping wells [55].

The methods to be adopted for clearing the slipped debris should take account of the geometry of the slide, the means of access for earth-moving plant, and the risks involved in clearance at the toe. In the case of rotational slips (**6.3.2**), if space is available, immediate restoration of stability can be achieved by removing material from the top of the slipped-off masses and placing the excavated material on the toe of the slip.

It is often desirable to flatten the rear scarp as well, but in the case of translational slides (**6.3.4**) such mass shifting is less effective. In any case care should be taken that further slips are not brought about by either the excavation at the head or the loading at the toe of the slide. This practice of toe weighting is effective when the weighting material is placed on a horizontal ground surface, but it may cause further slipping if the weighting material is tipped onto a sloping surface.

Mechanical methods which can be considered as immediate measures are driving sheet piles to support the toe of a slip, the use of gabions backed by porous filter fabric, driving old rails or timber piles at close centres or setting timber planks or sleepers between the flanges of H-section piles driven into the slumped material. Where a flow slide (**6.3.4.5**) or mud flow is endangering an important structure, stability can be restored immediately by freezing the sliding mass. This can be achieved by driving tubes with closed ends into the soil at close-spacing. Liquid nitrogen or other refrigerant is then injected into the tubes to freeze the surrounding soil. Although this process is effective it is costly to maintain in operation over an extended period of time.

Temporary slopes for foundation excavations can be stabilized against surface erosion by spreading tarpaulins, polythene sheeting, or porous filter fabrics over the soil. Where a clay slope is exposed to drying shrinkage it is often helpful to maintain stability during later rainy periods by covering the exposed clay with tarpaulins, polythene sheeting, or a layer of mass concrete. For best effect these measures should be taken before the opening of shrinkage cracks. Temporary slopes in soft clays and silts can be stabilized by electro-osmosis (**6.5.4.6**).

Immediate measures to restore stability of a rock slope include drilling holes to relieve water pressure, pinning unstable blocks of rock by bolting or dowels, and packing open cracks with concrete or mortar [56, 57].

11.5.3 ***Long term measures***

11.5.3.1 *General*. Before deciding on any long term measures to restore stability to a cutting slope, the causes of movement should be studied. A topographic survey and a general geological appraisal of the site should be followed by sub-surface exploration in trial pits or boreholes including measures to locate the surface of sliding (see **5.1.3**). Pore pressures and ground movements both in the slip area and the adjacent stable ground should be monitored using the methods described in **6.6**.

Having obtained all the necessary information on the ground conditions and completed an analysis of the causes of instability, the long term remedial measures should be decided in the light of the economics of construction, the importance of assurance of stability (e.g. to the maintenance of communications in a road or railway), the life of the works, and the existence of buildings or other works which may be endangered by continuing instability. Environmental considerations such as the appearance of the restoration measures may influence the scheme adopted. The long term measures given in **11.5.3.2** to **11.5.3.7** may be considered.

11.5.3.2 *Slope profile adjustment*. Slopes can be trimmed to a shallower profile if land at the top is available, or berms can be formed on the slope with the excavated material placed at the toe. These measures may affect stability elsewhere as new areas of underlying ground are loaded; appropriate consideration should be given.

The removal of vegetation cover during trimming may seriously reduce stability at the onset of wet weather. Measures therefore have to be taken to replace vegetation cover (**11.4.2**).

11.5.3.3 *Drainage*. Reducing pore pressures within the slope can be an effective and economical measure. One or a combination of the methods described in **6.5.4** may be adopted.

11.5.3.4 *Mechanical methods*. A retaining wall or an anchorage system may be required as described in **6.5.5**.

11.5.3.5 *Rock bolting and dental treatment of rock faces*. Stability of shallow masses of rock can be restored by rock bolting (**6.5.5.4**), or by spraying concrete or mortar onto the face to fill open joints or cracks or to support overhanging blocks. Larger rock overhangs can be packed by building walls in masonry or concrete. If necessary these walls or pillars can be anchored to stable rock behind by means of bolts or simple dowels. Means should be provided in the form of weep pipes to prevent build-up of hydrostatic pressure behind these retaining structures. In severely corrosive conditions, stainless steel rock bolts can be used.

Where the appearance of a rock face is an important consideration, the "dental" concrete (**6.5.5.5**) should be faced with stone rubble to match the surrounding rock. Where blocks of rock are anchored the bolt heads should be set into recesses cut in the rock face and the recesses then filled with cement mortar textured and coloured to match the surrounding rock. Consideration can also be given to spraying areas of weak disintegrated rock with resins or mortar similarly coloured and textured.

11.5.3.6 *Grouting*. Two forms of grouting may be considered as a remedial measure: penetration and hydrofracture. Penetration grouts of the suspension or colloidal solution type may be used to stabilize open textured granular soils or loose rock masses. The grout penetrates the voids between the soil or rock particles and sets to form a rigid mass. The techniques and materials for grouting are described in CP 2004.

The method of grouting should not be such as to cause further deterioration of an already dangerous slope. Advice should be sought from the Health and Safety Executive before using a chemical grout.

Hydrofracture of "claquage" grouting is an injection technique whereby the soil or rock system is displaced as the grout shears its way through zones or planes of weakness to form a seam. The required grouting pressure is a function of the soil strength.

A grout of higher viscosity is required than that for penetration grouting, so suspensions of Portland cement, sand, mineral fillers and wetting or aerating agents are generally used for economy. There is little penetration into the surrounding fissures or voids unless these are large. As the distance of travel of grout from the injection points is limited by its relatively high viscosity a grid of injection holes is required to deal with the mass to be treated. Stability is probably achieved by a chemical bonding action of the grout at the soil-grout interface involving a base exchange mechanism between the cement and the clay minerals.

Once started, a claquage process for treating a slip should be continued until completion of the scheme; the process cannot be interrupted for a period much greater than a week as otherwise stability is not achieved. Although used successfully on a few cutting slips the method is more applicable to embankment slips [55].

11.5.3.7 *Electrochemical methods*. Electrochemical methods of stabilization function on the principle of introducing chemical solutions into the soil such that there is an exchange of the mineral particles and hence an increase in shear strength of the soil. Electrochemical stabilization can be applied to soils such as silts or clayey silts which are too fine for stabilization using cement or conventional chemical grouts.

Because of the low permeability of these soils there are difficulties in introducing the electrolyte into the soil mass.

Two methods are possible:

a) in conjunction with the electro-osmosis system (**6.5.4.6**), when the electrolyte is fed into the anodes, and

b) by a mix-in-place method, where drilling equipment is used to mix the soil with the electrolyte in powder, slurry or solution form.

In cases where the soil minerals are favourable for base exchange, the latter method can be adopted to form close-spaced columns of soil stabilized with hydrated lime to support steep slopes [58].

Section 3. Trenches, pits and shafts

12 Design considerations

12.1 Site investigation. A general appraisal of the project and a site investigation should be undertaken as described in **5.1.2** to **5.1.7**.

Ground water levels may vary seasonally or in conformity with changes in level of adjacent tidal or river waters. Therefore it is desirable to monitor fluctuations in ground water level for as long a period as possible before commencing construction work, by means of observations in piezometers or simple standpipes.

The ground water observations made during the sinking of the site investigation boreholes are often misleading owing to the rate of drilling, the necessity to add water to assist drilling, the installation of temporary casings in the boreholes, and the nature of the ground.

Site investigation information should be made available to all tendering contractors and, wherever practicable, the ground conditions and water levels should be shown on the construction drawings.

12.2 Ground conditions

12.2.1 *General*. Knowledge of the ground conditions and land-surface features gained from the site investigation is used for the following purposes:

a) to determine the appropriate method of excavation and related plant requirements;

b) to determine the appropriate form of support to the sides of the excavation and to ensure its adequacy;

c) to determine suitable means of maintaining the excavations free from ground water;

d) to ensure that any potentially buoyant structures which may be constructed within the excavations will not be subjected to water pressures sufficient to cause uplift forces at any stage of their construction.

12.2.2 *Influence of ground conditions on construction methods*. The methods of excavation and types of temporary or permanent support depend on the following factors:

a) the variations in the geological structure, and whether the ground consists of cohesive or non-cohesive superficial deposits, or solid deposits;

b) the depth of the excavation;

c) the existence of a groundwater table;

d) the type and extent of the excavation;

e) the topography of the site and the proximity of existing land features such as roads, buildings, and buried services.

Excavations in non-cohesive soils, such as sands, present problems of confinement, since granular soils readily collapse unless they are cut back to a stable slope or restrained in position by continuous sheeting. The problem of confinement of granular soils is aggravated by the presence of ground water. Under no circumstances should the apparent cohesion imparted by water surface tension in damp granular soils be relied upon for the support of vertical faces in excavations. If, however, the lateral distances required for side-slopes are available within the site, then open excavation methods may be employed in these soils. In some instances, a combination of side-slopes and vertical sheeted excavations is advantageous. If the formation of side-slopes in granular soils is demonstrated to be economical but a high ground water table exists then the problem is often solved by using some suitable method of dewatering (see CP 2004).

Excavations in cohesive soils, such as sandy and silty clays and clayey silts, generally present little problem of confinement. The main problems are those of stability in the short and medium term whether of a vertical-faced excavation or of side-slopes. Stiff cohesive soils or weak rocks often have sufficient strength to permit open excavations to be readily formed by mechanical plant, provided that there are no restrictions on the magnitudes of ground movements which inevitably occur both within and immediately surrounding the excavations. The problem of the stability of unsupported vertical or steeply sloping faces involves many uncertain factors, one of which is the period of time during which the stability of the face can be assured (see **6.2.5**).

Silt-size soils present the greatest difficulties in excavation work particularly where a high water table exists. Such deposits are difficult to dewater and are often of such a consistency that they require to be completely confined by sheet piling. The installation of a lateral support system prior to excavation is a safe practical method of forming excavations in water-bearing silts.

It should not be assumed that vertical or near-vertical faces can safely be formed in excavations in rocks since unfavourable orientations in the bedding planes and joint systems can result in dislodgement of the rock blocks soon after the formation of the exposed rock face. Weak layers of rock strata within inclined rocks having relatively high intact strength, soft clay filling in fissures, and ground water under pressure in the geological planes of separation also significantly affect the stability of exposed rock faces (see **6.2.1.4** and **6.3.4**).

Rock strata inclined inwards and downwards into the exposed rock face should not be unquestioningly regarded as stable since the joints between the rock blocks can form failure surfaces if the bedding planes of the rock strata are inclined at steep angles.

Soils containing boulders or large cobbles can be hazardous where the face is left exposed even for short periods, because of the risk of falls from the face. Excavation in domestic or industrial wastes containing toxic or asphyxiating compounds or gases can also be hazardous to operatives.

These considerations should be studied carefully in relation to the safety of persons required to enter or work in close proximity to the excavations (see **4.1**).

12.2.3 ***Sources and control of ground water***

12.2.3.1 *General*. Ground water may enter an excavation by the following ways:

a) as surface or shallow subsoil water due to rainfall on the site and on the surrounding catchment which diverts water to the site;

b) because the excavation extends below the ground water table at the site;

c) when an excavation in relatively impermeable ground exposes a permeable layer containing ground water under a hydrostatic head, or where such a layer exists below the base of the excavation;

d) accidentally, e.g. by the bursting of a water main.

12.2.3.2 *Surface or near surface water*. Flow of surface or shallow subsoil water towards an excavation is inevitable in sloping sites during periods of heavy rainfall. Problems of ingress may be avoided by the provision of some suitable form of interceptor drainage or dam partly or totally surrounding the excavation.

Field drains or service trenches carrying subsoil water encountered in the excavation work should also be intercepted and diverted from the excavation. Excavation should, wherever practicable, commence at the lowest point and progress in an uphill direction in order to facilitate the maintenance of a relatively water-free working surface.

Deep excavations may alter the local hydrology by transferring surface water into permeable strata causing flooding in adjacent excavations, cellars, or basements.

12.2.3.3 *Ground water*. Ground water may be significantly excluded from an excavation which extends below the water table by surrounding it by a cofferdam constructed of interlocked steel sheet piles or of in-situ concrete walling. Permeable soils can be injected with cement, chemicals, or clay slurries to form an impermeable curtain around the excavation (see CP 2004).

If the interlocked steel sheet piles or concrete walling of a cofferdam can be taken down to a thick relatively impermeable deposit, the ground water in the overlying permeable ground will be excluded, apart from leakage through the interlocks or joints which is readily removed by normal pumping methods. Considerable care has to be taken where the relatively impermeable deposit at the base of the excavation is either thin and is underlain by a water-bearing permeable deposit containing ground water under hydrostatic pressure, or is thick and contains thin water bearing permeable soil layers and considerable hydrostatic head. In such cases, instability of the base of the excavation can be experienced due to hydrostatic uplift forces. The hydrostatic pressure may be relieved by deep filter wells.

If a relatively impermeable deposit does not exist within reasonable depths of penetration of the steel sheet piling, it is still possible, with considerable pumping, to achieve a stable relatively water-free excavation. Alternatively, open excavations with side-slopes may be formed beneath the ground water table by utilizing appropriate methods of lowering the water table. Reference should be made to CP 2004 which gives details of methods of dewatering excavations.

12.2.3.4 *Accidental flooding*. Accidental flooding may arise from causes such as prolonged heavy rainfall or bursting of a water main and often results in considerable damage. Consequently there is a statutory requirement to provide ready egress from excavations (see **4.1**). Where the excavation is formed within low-lying land or water mains exist in close proximity to the site, adequate measures should be taken to guard against the risk of accidental flooding, particularly where the effects would endanger life and property.

During periods of heavy rainfall loss of stability may result from a rapid increase in the elevations of the ground water table and the corresponding increase in hydrostatic pressures.

12.2.3.5 *Excavation in sloping ground*. Consideration should be given to the overall stability of the site where excavations are made on side-sloping ground. Particular care is necessary on sites with a past history of instability. The stability conditions at various stages of the excavation work can be analysed by the methods described in **6.4**.

12.3 The design of stable slopes and supports to excavations

12.3.1 *General*. In general the application of soil mechanics theories using strength parameters obtained from field and laboratory investigation work to the design of excavation supports or the determination of stable angles of slope is unnecessary in the case of relatively shallow trenching for services or shallow excavations for structural foundations. In such cases, the design of the earthworks and support systems is based on experience and reference to standard published works.

For trenches having depths not exceeding 6 m a simple investigation of the ground conditions normally provides sufficient information to permit the selection of the most suitable system of support without the need for the application of soil mechanics theories. It is essential, however, that even a simple site investigation should be carried out with sufficient care and attention to ascertain as far as is reasonably practicable that the ground conditions have been properly defined and that all variations which would affect the stability of the excavation are investigated in nature and extent.

The selection of suitable types of sheet piling or timbering is likely to be governed by considerations of avoidance of damage to these components during installation and by the requirements of re-use. Modifications to the selected system may be required in the light of changes in ground conditions revealed during excavation. The choice of construction method usually lies between open excavation with the formation of stable side-slopes, or the use of temporary or permanent support to a vertical face. Guidance on the design of safe side-slopes for most excavations can be found in **6.5**. It should be noted that the design of steeply sloped or vertical unsupported faces in cohesive soils presents considerable difficulties and the assessment of the rate of decrease in the safety factor after excavation cannot be determined by theory. Even in stiff glacial till or other stiff over-consolidated cohesive soils the presence of permeable layers or fissures within the soil can have a dominating influence on stability, as positive pore water pressures may develop rapidly behind the slope for which negative pressures are required to maintain stability.

It may be possible to rely on the stability of an unsupported vertical face of a trench excavation less than 1.2 m deep, where precautions are taken against overloading the ground surface close to the edge of the trench and against risks to operatives from local falls, say from the dislodgement of boulders. Reliance should never be placed on the ability of the ground to remain stable when excavated to a vertical face over the period of time between completing the excavation of a length of trench and setting and bracing the support system. A vertical face deeper than 1.2 m may be adopted in a large open excavation where falls or dislodgements from the sides would not cause a danger to operatives or damage to permanent works or neighbouring structures, and where the excavations are suitably fenced for the safety of operatives and the general public.

The short term and long term stability of the unsupported slopes and base of deep excavations should be verified by calculations.

12.3.2 *Magnitude and distribution of lateral soil pressures*. Lateral pressures on the support systems of excavations should be calculated in the manner described in CP 2. The earth pressure distribution on the sheeting or other form of supporting wall should be in accordance with conventional practice for the design of retaining walls. The maximum loads on struts or anchors can be represented by an envelope of rectangular or trapezoidal form (Figure 21 and [59]). This envelope should not be assumed to represent earth pressure distribution for calculating bending moments in the supporting walls. Bending moments and forces in the walls and bracing members should be calculated for each successive stage of excavation and installation of the support system and its subsequent removal.

The Institution of Structural Engineers has produced a report [60] on the design of deep basements and reference should be made to this publication for information on the procedure to be adopted for the design of temporary works for structures of this type.

12.3.3 ***Stability of base of excavation***

12.3.3.1 *General*. Base failure may be experienced in deep excavations either by uplift of granular soils due to large seepage forces caused by high hydrostatic heads, or by shear deformation of soft saturated cohesive soils due to overstress.

12.3.3.2 *Water-bearing permeable soils*. For supported excavations below the ground water table in granular soils it is customary to use interlocked steel sheet piles driven to a prescribed depth of penetration below the base of the excavation. Where the sheet piles do not have sufficient depths of penetration below the base of the excavation to reach an impermeable stratum which would provide a cut-off to the inflow of water beneath the toes of the sheet piles, then upward seepage occurs at the base of the excavation. If the head of water outside the sheet piling is such as to cause a steep hydraulic gradient over the length of the seepage path, then the velocity of upward seepage may be such as to cause instability in the form of uplift or "boiling" of the soil particles (Figure 22).

Boiling may also occur as a result of strong flow from a permeable layer underlying less permeable soil at the base of the excavation.

The design of deep excavations in water-bearing granular soils should consider this possibility and if necessary, ground water lowering methods should be used to lower the external head of ground water, or alternatively the sheet piling should be driven to a deeper penetration to lengthen the seepage path so decreasing the hydraulic gradient (see CP 2004).

12.3.3.3 *Soft cohesive soils*. Failure by heaving may occur in deep excavations in soft cohesive soils through overstressing of the soil in the region of the base of the excavation. The likelihood of base failure by shear deformation may be predicted by conventional methods of stability analysis; for supported excavations the analytical method of Bjerrum and Eide may be used [61]. A safety factor of at least 1.5 should be employed against overstress.

12.3.4 *Movements at base of excavation*. The magnitude and rate of upward movement which occurs at the base of an excavation depend on the reduction in vertical stress caused by the removal of soil from within the excavation, on the nature of the strata underlying the base of the excavation, and on the ground water conditions.

The upward movements which take place are caused by immediate elastic strain which occurs simultaneously with the deepening of the excavation and by long term volumetric strains due to moisture content changes. In stratified cohesive deposits which display high horizontal permeabilities, heave caused by volumetric strain can be rapid.

The magnitude of upward movement is generally greater at the centre of the base of the excavation than at the periphery, but uplift can take place beyond the periphery and thus act as a counter to the settlement which may occur as a consequence of yielding of the excavation supports. The magnitude of heave may be predicted by elastic theory but the rate of heave cannot be reliably predicted on the basis of theory and few field measurements have been made.

12.4 Practical considerations

12.4.1 *Methods of excavation and types of support*. It should be appreciated that no matter what method of excavation is used, ground displacements occur both within and immediately surrounding an excavation. These ground displacements depend partly on the geological structure and are principally due to elastic strains. In cohesive soils, volumetric strains due to changes in moisture content also take place. If the method of excavation and the type of support are unsuitable for the particular ground conditions, then shear deformations or shear failures of the soil or failures due to hydrostatic pressures may occur. Vibrations from construction equipment may cause consolidation of cohesionless soils or have a detrimental effect on existing structures in a weak condition. The sequence of excavation and installation of lateral supports has a significant effect on the stresses and strains induced in the ground.

It may not be practicable to prevent significant vertical and lateral ground displacements immediately beyond the limits of an excavation, so the effects of the inevitable movements on any adjoining structures have to be considered. It may be necessary to underpin adjoining structures before commencing an excavation, in order to protect them from the ground displacements. Alternatively, and with the agreement of their owners, damage to the adjoining structures may be accepted and repaired after completion of the permanent work. However, the damage should not be such as to cause any danger to the occupants of these structures, or to the general public. A simple construction procedure is desirable since alterations to a complex construction sequence, when unexpected variations in ground conditions are encountered, is often difficult. The work should not be undertaken without experienced supervision, and inspection should be made several times each day to ensure that stable conditions are being maintained.

Narrow trenches are sometimes excavated with unsupported vertical faces, depending on rate of construction, soil type and strength and depth of trench excavation. Where trenches are deeper than 1.2 m they should be supported where men are required to enter them. Stability conditions should be regarded as unfavourable even in firm and stiff clays and in fissured and closely jointed rocks.

It may be economical to incorporate support systems such as steel sheet piling, concrete diaphragm walls, or contiguous bored pile walls in the permanent construction.

No lateral supports for any part of an excavation should be altered or dismantled except under the direction of the designer or a competent person possessing adequate experience. A store of suitable supports should be kept on site to provide immediate strengthening, if found necessary.

12.4.2 *Existing buildings, buried structures and services*. The age, types of construction and the type and depth of foundations of existing buildings which would be affected by the excavation should be ascertained before commencing work on site. An appraisal of the dead and superimposed loads from the foundations of existing buildings should be made since the stability of the excavation may depend on an accurate prediction of imposed loading on the selected system of retention for the excavation. It is advisable that all buildings and buried structures which are likely to be affected by the excavation work should be surveyed with the representatives of the owners, and dilapidation reports prepared and signed by all interested parties. The dilapidation report should contain photographs of any building defects. Significant structural cracks should be instrumented by simple strain gauge devices and levelling points installed to permit any building movements to be monitored [62]. Where ground anchors used for excavation supports pass beneath existing buildings and highways the effects of the drilling and grouting processes used to install the anchors should be considered. It is of course necessary to obtain the permission of the building owners or highway authority for the installation of anchors beneath their property.

The design and construction of excavations and their retention systems should also take into consideration the prior location and safe support of all services such as water and gas mains, and buried structures such as underground tunnels and stormwater and foul sewers.

12.4.3 *Working space.* On confined sites the whole available working space may have to be within the boundaries of the excavation. Certain items of the plant may be supported at or near ground level on temporary stagings, or may at a later period be carried on portions of the completed work. It may be necessary to operate other plant, or to store it temporarily, in the bottom of the excavation. In this case the working space varies as work proceeds. A careful preliminary study should be made of the use to which the working space may be put at each stage of the job to ensure that concurrent or successive operations are not obstructed unnecessarily by plant, spoil heaps, materials, roads, temporary trenches, sumps etc. Care should be taken that heavy plant, spoil and materials are never placed so as to endanger adjoining buildings, buried structures, services, the stability of the excavation, or the safety of operatives.

12.4.4 ***Disposal of spoil***

12.4.4.1 *General.* The method of handling excavation spoil depends on such factors as the proportion and quantity to be used as fill material and on the working space available within the site.

The specification and drawings, should, where necessary, give clear instructions as to the disposal of all spoil so that the procedure for excavating and transporting material can be planned.

The amount of material which can be used as fill depends on the nature of the structure and the suitability of the spoil. It may be necessary to separate spoil suitable for filling from that which is unsuitable. Topsoil, which is valuable as vegetable soil but unsuitable as fill, should be stripped and either used or stored separately. The work should as far as possible be arranged to avoid the necessity for temporary spoil heaps and the consequent double handling. Transport and, to a lesser degree, excavation methods may be influenced by the distance from and access to disposal sites (see **8.1**).

12.4.4.2 *Temporary spoil and material heaps.* Temporary spoil and material heaps should be sited to interfere as little as possible with the work to be carried out. For convenience in handling it may be necessary to place them near excavations but the following points should be borne in mind.

First, they should not interfere with free access to the excavation (in trench work it is desirable to place the material which is to be used for backfilling on one side of the trench only). Secondly, they should be so disposed that there is no danger of the spoil slumping in wet weather and entering the excavation. Thirdly, the spoil heaps should not be placed in such a position as to endanger the stability of existing works above or below ground or of the excavation, the sides or side supports of which should be so designed as to be capable of withstanding the additional stresses due to any superimposed load.

Spoil heaps should be graded to safe slopes taking into consideration the nature of the material and the effects of wet weather. With coarse sand or clean gravel the natural angle of repose of the tipped materials should remain substantially unaltered in wet weather, but with materials that soften and slump, e.g. clays, silts, mudstones etc., a substantial reduction in slope has to be anticipated and an adequate distance maintained between the periphery of the spoil heap and the edge of excavation.

The clearance between the toe of the spoil heap and the edge of the excavation should give sufficient working space at all times, and for this purpose a minimum width of 1.50 m is recommended.

13 Construction procedure

13.1 Temporary support of excavations

13.1.1 *General.* Temporary supports can make use of timber, steel sections or precast reinforced concrete. The same general principles apply irrespective of the material employed for the supports.

The selection of the method of support is largely influenced by the type of ground encountered but the plant used for excavation and the amount of re-use expected from the components are also factors to be considered. All supported excavations should be provided with guard rails, hand rails, walkways, staging and ladders in accordance with the Construction (Working Places) Regulations, 1966, as may be required to safeguard operatives working within or around the excavations.

The following types of support are commonly used.

a) *Poling boards.* Placed in a vertical position against an excavated face standing vertically to a height of 1 m or more while the boards are set in position (Figure 23 and Figure 24). They can also be used in a top frame to facilitate the pitching of runners [Figure 26(d) and Figure 33(a)].

b) *Horizontal sheeting*. Used to support ground standing up to a face of 300 mm to 600 mm while the sheeting members are being placed and wedged against the face. The method is advantageous for supporting very deep excavations (Figure 25), particularly where drainage of the soil can be allowed, thus avoiding hydrostatic pressure on the supports.

c) *Runners*. Used in soft or loose ground which requires continuous support at all stages of the excavation. They are driven down slightly in advance of the excavation to avoid flow of the soil below the toe (Figure 26). Runners are also used in better ground where their employment as open or close sheeting is considered more practicable and economical than other forms of face support.

d) *Sheet piling*. Used for soft or loose soils, and in water-bearing soils where it is desired to avoid drawdown of ground water levels outside the periphery of the excavation and also to prevent "boiling" of the base of the excavation under conditions of a high external head of ground water (**12.2.3**). The sheet piling is driven to the full required depth and is supported by walings and struts or ground anchors in the required stages as the excavation is taken down (Figure 27 and Figure 28).

e) *Movable shoring systems*. Proprietary shoring systems consist of sheeting panels or face "boards" which can be lowered into a trench or pit and jacked against the soil by means of hydraulically-operated struts or rams. The whole assembly of sheeting members, walings, and struts may be towed along a trench in the form of a sled. Alternatively, individual components, for example a pair of faceboards linked to a hinged adjustable strut, can be lowered into a trench or pit and subsequently lifted out without the need for an operative to enter an unsupported excavation.

While timber has been shown for strutting in Figure 23, Figure 24, Figure 26, Figure 27 and Figure 28, adjustable steel screw struts may be more economical in many cases.

The methods used for bracing the sheeting members described above, whether by walings or vertical soldiers, with struts or ground anchors, depend on the type of structure to be constructed within the excavation and the site conditions around the excavation. The drawing of sheeting members requires consideration since timber which has to be left in rots and forms a void, and for economic reasons the materials should be re-used as far as possible. Struts can nearly always be drawn out but sometimes walings can be extracted only in short lengths and with considerable difficulty, especially if reinforced concrete work is being carried out in a restricted working area. Poling boards or horizontal sheeting often have to be left in when work is carried out on confined sites where a working space outside the permanent structure cannot be provided. When it is known that the supports have to be left in, particularly above standing ground water level, pressure creosoted timber, precast concrete, or steel sheeting should be used where the formation of a void has to be prevented. Consideration should also be given to pressure grouting with cement where components are left in support of public highways or adjoining property.

When constructing reinforced concrete retaining walls in a restricted working space the relationship of the bracing system to the reinforcing steel has to be allowed for. Where the vertical steel bars are fixed in front of the walings the space available for removing the walings and drawing the timbers has to be considered. The spacing of the bars should also allow for positioning and withdrawing the struts. Space can be saved by packing out the walings from the sheeting members thus allowing the bars to be fixed between the two (Figure 29). The use of diaphragm walls or contiguous bored pile walls can often be more economical in these confined situations (see **13.3**).

The use of ground anchors as an alternative to strutting to support walings or to give direct support to diaphragm walls or bored pile walls may not be feasible because of the presence of existing basements, sewers, or tunnels around the excavation, or the inability to obtain wayleave to install anchors beneath adjacent property.

The faces of excavations in rock can be given temporary support by mesh-fabric or short walings restrained by anchors grouted into the rock behind the face.

13.1.2 *Effect of width*. The method adopted for supporting the sheeting, walings, or soldiers depends on the width of the excavation.

a) Trenches up to about 6 m wide can be strutted from side to side by single timber or steel struts.

b) Trenches exceeding about 6 m in width require the use of king piles or soldiers to permit the use of two or more single lengths of timber to span the width of the trench thereby reducing the effective length of the strut. Alternatively trenches 6 m or more in width can be strutted with steel or reinforced concrete members with or without the need for restraint by king piles.

c) For excavations which are so wide that support of the sheeting by walings and struts would be uneconomical one of the following methods can be used.

1) Cutting of earth faces to a slope so that they are stable without timbering.

2) Supporting the sheeting by raking shores taking their reaction from the ground slab concrete previously constructed within the interior of the excavation.

3) Tying back the walings or soldiers by means of anchors restrained by the ground behind the excavation face. Where cast-in-place concrete walls are used the anchors can be connected directly to the walls without the need for horizontal walings.

4) Constructing the permanent earth support in a trench around the perimeter of the excavation. This acts as a retaining wall subsequently allowing the ground in the interior of the excavation (the dumpling) to be removed without further temporary support. The retaining wall should be designed to be self-supporting at this stage, and should not rely for stability on a basement floor or ground floor slab constructed at a later stage of the work.

5) Constructing permanent support across the full width of the excavation in the form of the ground floor slabs or intermediate lower ground floor slabs of a deep basement. Strutting by this method is followed by removing the soil from below the slabs through openings left for this purpose. Restraint to buckling of the slabs is normally required and this can consist of the permanent columns of the building, or temporary bracing can be provided. This method is most economical when used in conjunction with permanent support to the sides of an excavation by means of in-situ concrete, diaphragm walls or contiguous bored and cast-in-place piling.

13.1.3 *Support by poling boards or steel trench sheets* (Figure 23 and Figure 24). Poling boards in timber range in size from 1 m long by 32 mm thick to 1.5 m long by 50 mm thick. A common size, suitable for excavations to 10 m depth is 1 m by 32 mm, but for deeper excavations where the ground swelling pressures can be high thicker boards will probably be required. The upper supporting frames are often sheeted by 1.2 m × 50 mm boards. The levels of the frames and supporting walings are frequently determined by the dimensions of the permanent work and it is undesirable to have to adjust the positions of supporting members during the progress of the work to conform to these dimensions.

Normally, and provided that safe working conditions are obtainable, the ground is excavated to a depth equal to the length of the boards or trench sheets and the sides are roughly dug to a vertical face, care being taken to leave some ground to be trimmed off by the timberman. The length of the waling having been decided, the timberman carefully sets a board to plumb and line at each end of the length of face to be covered by the waling. The intermediate poling boards are then fixed to a line stretched between the end boards. The small amount of earth which has to be removed before each board is set is cut away by the timberman who should take care to remove only sufficient earth to permit the boards to have a good bearing against undisturbed ground.

When both faces have been poled, the walings are placed against the boards and temporarily propped off the bottom of the trench at their intended level. The struts, which have been cut to length outside the trench with their lip blocks already fixed, are dropped into position and held by the lip blocks (Figure 23) until the strut is tightened against the waling by wedging. Alternatively, the strut can be tightened by cutting it slightly over-long and then driving it at one end to bring it square to the waling which is forced back by the strut. Adjustable screw struts can also be used.

After tightening all struts the waling should bear against each individual poling board and generally the frame should be well-designed and have a neat workmanlike appearance. The frame can then be ground propped and puncheoned and laced to any frame already set above it (Figure 23).

In firm or stiff clays or in weak rocks with widely spaced joints, close support of excavated faces may not be required, and the poling boards or trench sheets can be placed at a wider spacing with gaps of one or more board widths between them. Usually the walings are set at the top and bottom of the poling boards or trench sheets. When this is done the boards in each lower setting are tucked behind the bottom waling, using tucking boards for this purpose (Figure 24). The advantage of the "tucking-frame" system is that when the hole has to be filled with concrete and the boards are left in position the earth face is supported at all times as the filling proceeds and as each waling is drawn; also, that the boards are always supported at the top and bottom. In the alternative middle board system the walings are set only at the middle of the boards (Figure 23).

13.1.4 *Support by horizontal sheeting* (Figure 25). Deep excavations are supported by first driving steel H-section soldier piles to the full required depth around the excavation. If the ground conditions are such that the piles cannot be driven to the full depth or if driving has to be avoided from considerations of noise or vibrations, a drilling rig is used to form a hole into which the pile is lowered. The bottom of the pile, below excavation level, is fixed in position by surrounding it by lean concrete or cement-stabilized sand. Excavation is started and the horizontal timber sheeting boards are set in position as the excavation is taken down. In reasonably good ground the boards are set between the flanges of the H-section and are held tightly against the face by wedges driven between the ends of the board and the inner flange of the pile. This necessitates exposing the face below the board already set by a depth equal to the width of two or three boards.

In weak or loose ground which cannot stand up unsupported over this depth, the soil is exposed for a depth equal to only one board. The latter is then placed against the inner face of the inner flange of the H-section and held to it by clips.

As the excavation is taken down the soldier piles are strutted or anchored at the required levels. The struts or ground anchors may be provided for each soldier pile. Alternatively groups of soldiers may be supported by horizontal walings with single or pairs of struts or anchors to each waling (Figure 25).

In very deep excavations the soldier piles may be provided in two or more settings.

The soldier piles are unlikely to be set truly vertically or at constant spacing, particularly when they are installed by driving. Timber is therefore the best material for sheeting since it can be readily cut to the required length for insertion between the soldiers. However, steel sheeting can be used and the ends burned off where adjustments in length are required. Where a concrete facing wall is desirable this is best cast-in-place in short lifts between the soldiers rather than constructed from precast concrete planks that cannot readily be cut to length.

13.1.5 *Support by runners*. Runners are pitched and driven down as excavation proceeds to form open or close vertical sheeting (Figure 26). In bad ground close sheeting is adopted and the ends of the boards are "toed-in" below the bottom of the excavation to prevent the loss of ground. Timber runners are square edged and can be provided with splayed chisel ends. The shape shown in Figure 26 keeps the timber hard against the ground and the adjacent board. The edges can be tongued and grooved for watertightness but this practice is rarely adopted if interlocking steel trench sheets are available for use as runners.

Timbering commences by fixing a waling at the top level [Figure 26(a)]. In determining the spacing of the waling from side to side of the excavation, allowance is made for the insertion of wedges (pages) between each runner and the waling. To facilitate spacing and proper alignment of the waling frames short timber uprights are set behind the walings opposite the ends of each strut and hard against the earth face. The runners are then set behind the walings between each upright, and the main excavation can be commenced [Figure 26(b)]. In good ground the excavation can be taken below the toes of the runners and an open spacing can be adopted. When excavating below the runners the latter are held from dropping by their wedges, which should be kept firmly in place at all times. On completing the stage of excavation, the wedges of each runner are released in turn, the board is driven down by maul to toe it into the ground below the bottom of the excavation and the wedges are then replaced and tapped into position. In bad ground the toes of the runners are kept below excavation level at all times. They are driven down in stages of 150 mm to 200 mm working progressively along the cut. To fill the space beneath the uprights at the strut positions short boards (cross-poling) are packed behind the adjacent runners. On completion of the upper section of the excavation a second waling frame is set in position and strutted [Figure 26(c)].

In good ground an unsupported excavation can be made to a depth of 1.2 m in which the two frames of walings and a setting of runners are installed. In bad ground it may be advantageous to timber the upper 1 m to 1.2 m with poling boards fixed as described in **13.1.3**, except that the first stage of excavation is taken only down to top waling level and the short poling boards are driven down in a manner similar to runners. When the excavation has reached just above the bottom of the poling boards the runners are fixed inside the top waling frame, which acts as a guide in conjunction with inner guide timbers to keep the runners in alignment as they are driven down to support the deeper section of the excavation [Figure 26(d)].

This deeper stage of excavation is supported by a second setting of runners pitched within the walings supporting the first setting. Uprights are again used to align the waling frames. The upper end of the upright is tucked behind the bottom waling of upper setting, and the bottom end is set above the bottom of the next waling to give space for tucking in the next upright below it.

In planning the dimensions of deep excavations allowance should be made for setting in the timbering of each stage of excavation with successive settings of runners installed as described above.

13.1.6 *Sheet piling of excavations.* Sheet piles for supporting excavations are rolled steel members either trough or Z-shape in cross section and with interlocks to enable them to be driven to form a watertight wall. Guide timbers are first laid on the ground together with a guide trestle to control the alignment and verticality of the piles. A pair of piles is pitched within the guide timbers, carefully plumbed and then partly driven down with adjustment to plumb as required after this first stage of driving. The adjoining piles are then interlocked with the first pair, a panel of ten to twenty being pitched within the guides. The last pair of the panel is driven to two-thirds of its full penetration, then the remaining piles are driven to their full depth working backwards towards the first pair to be pitched. The next panel of piles is pitched and driven and the procedure repeated until the area of the excavation is surrounded completely by the wall of sheet piles. In the case of small excavations it may be advantageous to pitch the piles around the full perimeter of the excavation before driving them. The piles in each panel should be corrected for alignment, as required, by pulling the panel by steel ropes. If this is not done an initial tendency to lean becomes cumulative, requiring a special taper pile to close the interlocks when completing a box or ring of sheet piles.

Sheet piles are driven by air or steam-operated double-acting hammers, or by diesel hammers, or by vibrators held by grips to the top of the sheet piles. Drop hammers or single-acting hammers may be used but these require the use of a pile frame or a set of hanging leaders with which to guide the hammer and keep it in alignment with the pile. The noise of driving steel sheet piles can be greatly minimized by enclosing pairs of piles and the hammer in a sound-absorbent box, by using a hammer or vibrator designed for low noise emission, or by using a proprietary jacking system [63, 64].

The sheet piles are driven to their full intended depth before commencing the excavation. As the excavation is taken down the sheet piles are supported by successive frames of walings and struts (or ground anchors). The frames should be prevented from falling into the excavation by puncheons or props, suspending them from steel rods hooked over the tops of the sheet piles or by setting them on brackets fixed to the face of the piles (Figure 27 and Figure 28).

Any soil which is hanging up in the troughs of the sheet piles should be removed as the excavation is taken down. If this is not done it is liable to become dislodged, creating a hazard to operatives working at the bottom of the excavation.

13.1.7 *Adjustments to supports to enable the permanent work to be constructed.* It is often necessary to adjust the position of support frames during the construction of the permanent work. When the permanent structure does not completely fill the excavation, at least one face has to be supported during the progress of the work. A typical example is the construction of a retaining wall in a trench.

The walings which are to be left in position until the supports are finally removed have to be held in a manner which allows the struts to be taken out. This is normally done by soldiers placed vertically between the strut positions and held at the lower ends by packing off the permanent structure and at the upper ends by struts across the excavation. The permanent structure should have attained sufficient strength and should be in a condition of stability adequate to resist the thrust from the soldiers.

In the interests of safety and to minimize the movement of the adjoining ground, alterations of the struts in the support system should be kept to a minimum.

When a waling has to be removed as the work proceeds it is the normal practice to bring the permanent work up to the underside of the waling before removing it.

Where space has to be provided for reinforcing steel projecting from partly constructed work, the positions of walings can be adjusted by moving them inwards and supporting the poling boards or sheeting members by blocking or chogs (Figure 29). The reinforcing steel is then accommodated between the walings and the poling boards. Temporary walings are used while adjusting the previously placed members.

13.1.8 *Maintenance of supports.* Constant inspection of the support system is necessary to ensure that all components are working effectively and are not showing signs of overstressing as indicated, for example, by struts punching into walings or bowing of sheeting. Loosening or distortion of frames can occur owing to drying shrinkage of previously wet timber, loss of ground from behind the sheeting, shrinkage of the retained soil in dry weather, and accidental blows from excavation machinery or crane skips.

Heavy pressure on the supports can occur owing to swelling of timber, originally installed in a dry condition but then becoming wet in the excavation, or swelling of a previously dry soil.

Sheeting should be kept firmly in contact with the excavated face and any "runs" of soil through gaps should be stopped immediately. Folding wedges and wedges to runners should be inspected periodically and tightened as necessary. If struts show excessive punching into walings additional struts should be introduced or hardwood facing blocks should be inserted between the ends of the struts and the walings. To prevent accidental displacement of timber at any stage in the construction the rubbing boards and lacings should be fixed between settings of struts, and if required between walings also.

Dry rot is likely to occur if timbering is required to be kept in place for long periods. The faces in contact with the soil are the first to be affected and the rot may eventually extend to walings and struts. When observed rapid action should be taken to prevent infection of the adjacent sound timber by the dry rot fungi. The infected timber should be removed and burnt. If replacement is required the use of precast or in situ concrete or steel sheets should be considered. Creosote should be applied liberally to timber left in position. Where it is anticipated that timber will remain in place for a long period pre-treatment with a fungicide is desirable.

13.1.9 *Striking of supports.* A well-designed support system should be capable of being struck easily without risk to the operatives or damage to the permanent work. Unless the operations are properly planned and undertaken to a prescribed sequence the removal of the supports may involve hazard to the operatives. Operatives should not be allowed to work in confined spaces to remove components in situations where collapse or yielding of the ground would trap or bury them. The scheme should permit the operatives to work from ground level or from properly supported ground while performing the striking operations. The supports are removed in the reverse order of installation, i.e. the struts are removed first and the walings are strutted temporarily off the permanent work as may be required. Then backfilling should be placed between the permanent structure and the sheeting. This gives temporary support to the sheeting and allows the walings to be removed. Finally the sheeting members are drawn.

In order to remove a strut it has first to be relieved of pressure, which may be done by one of the following four methods:

a) by driving one end sideways (where folding wedges have not been used for tightening);

b) by extracting folding wedges;

c) by the temporary use of a screwed steel strut or a jack at the end of a strut;

d) by cutting out the wedges or by cutting a short piece of strut away from the end. This expedient may be necessary where heavy pressures have developed and methods a) to c) cannot safely be used.

Where poling boards have to be left in, the concrete of the permanent structure, or backfill material, or other construction should be brought up to the underside of the waling so as to support the boards over at least one-third of their length before removing the waling.

Where the ground is stiff or compact and open poling can be used the timbering can be withdrawn before placing the backfill, but the filling should be performed immediately after drawing the timbers.

Horizontal sheeting can be removed, board by board, as the backfill or permanent construction is rammed against the earth face. Finally the H-section soldiers are withdrawn by jacking, or by piling extractors. It may be more economical to leave them in position.

Operatives should not enter narrow trenches or confined spaces in wider excavations to perform the operations of withdrawing timbering if this involves their working in unsupported ground.

13.2 Backfilling. With the possible exception of trenches in open unpaved ground, backfilling needs as much care both in planning and construction as other portions of the permanent works, and unless satisfactorily carried out can be the source of much future trouble and expense.

The general objectives in backfilling are to permit safe removal of the supports, to restore the stability of the earth sides and bottom (where this is not done by the permanent structure) and to restore the ground surface with a minimum of future settlement.

The degree of compaction given to backfilling depends on the amount of settlement which can be tolerated in the restored ground surface and the support necessary for any pipes or other services which may be buried in the backfill or in the adjacent unexcavated ground. The degree of compaction which can be attained depends on the nature of the backfill and on the methods used. Generally, if excavated material is used for backfilling its moisture content and density, after compaction, should be as near as possible to that of the undisturbed soils, particularly with clays which may dry out or conversely become wet and soft when in the spoil heaps. Such material cannot be compacted satisfactorily and it may be desirable to replace it by granular backfill or lean concrete wherever load-bearing capacity or minimum settlement are important. The compaction of the imported backfill has to be undertaken with care, each layer being well rammed or vibrated as described in clause **9**. Spaces left by drawing timbers should be properly filled in.

Where it is necessary to place fill through water, only coarse granular material should be used. Concrete placed in bags can be used where load-bearing capacity is required from underwater fill.

In water-bearing ground the space between the backfilled excavation and the permanent structure may act as a reservoir in which ground water can accumulate. The effect of such impounded water on the permanent structure or on existing adjacent works should be considered, and if necessary drainage should be undertaken to control ground water levels. Existing land drainage systems intersected by the excavation should be properly reinstated or diverted around permanent underground structures before the backfilling is completed.

Where new pipes are to be laid or existing pipes have to be re-supported, particular care is needed in backfilling if future troubles from bursts or leakages caused by differential settlement are to be avoided. The backfilling should therefore be undertaken in a manner which gives uniform support to the pipework, and in the case of existing pipes the support should, as nearly as possible, be equal to that of the adjacent ground into which the pipe passes.

The necessary uniformity of support can be achieved by backfilling with selected material placed and compacted by hand and carried high enough above the pipes to protect them from damage by the mechanical equipment used to place and compact the remaining backfill. If fill of suitable quality is not available it may be advisable to use lean concrete instead. Consideration should also be given to the use of flexible joints in new pipework or in connections to existing pipework to accommodate settlement in the backfill or from the permanent structure.

13.3 Support of excavation by diaphragm walls or cast-in-place bored piles. Permanent structures can be used as excavation supports. They may take the form of concrete cast-in-place in a trench supported by bentonite slurry (diaphragm walls), or walls formed from touching bored and cast concrete piles (contiguous bored piles), or interlocking bored and cast-in-place concrete piles (secant piles). These methods are useful for constructing underground structures in confined situations where insufficient space is available for conventional timbering outside the permanent work; they can also be used for constructing permanent retaining walls close to existing foundations. Because of their rigid design and the avoidance of ground disturbance caused by installing and striking conventional timbering, these forms of support are advantageous where the yielding and settlement of the adjacent ground surface or property have to be reduced to a minimum.

Yielding of the sides of excavations for deep basements can be further minimized by combining support from cast-in-place retaining walls with the rigid strutting provided by the ground floor and any lower suspended floors of the basement. These floors are cast on the ground abutting the permanent retaining walls. Excavation within the retaining walls is then taken out beneath the floor, the spoil being removed through openings left for the purpose. Although these methods greatly minimize inward yielding of the sides of the excavation and hence settlement of the surrounding ground surface, such movement cannot be prevented entirely. Inward movement of the retaining walls occurs as they deflect under the external earth pressure, and as the strutting provided by the ground floor slab or basement floors compresses under load or shrinks with the maturing of the concrete in the substructure. Where ground anchors are used temporarily to support the cast-in-place concrete walls, stretch in the anchors and yielding and creep of the soil or rock into which the anchors are grouted can also cause deflection of the retaining walls.

14 Trenches

14.1 Construction methods. The design of a trench and the method of excavating, supporting and backfilling it should take into account the following factors.

a) *Purpose and location of the trench*.

b) *Size of the trench*. The depth and width have to be sufficient to allow the permanent work to be properly constructed.

c) *Trench opening*. The length of trench to be open at any one time and the period it is to remain open are governed by the requirements for installation, inspection and testing of the permanent works. No trench should be left open longer than necessary.

d) *Nature of ground*. Trial pits and intermediate boreholes, taken to a level below the bottom of the trench, provide useful information, including ground water levels. It should be appreciated that ground conditions can vary widely and frequently along a trench and investigations before excavation are at best only a guide.

Excavation alongside an earlier backfilled trench is to be avoided if possible. If a trench in such a position is essential, extra care is necessary with trench supports.

e) *Removal of ground water*. See **12.3** and **14.6**.

f) *Statutory obligations*. See **14.2**.

g) *Obstructions*. These may be above and below ground level. Some information on obstructions below ground can be obtained from site inspection, enquiries of public utility authorities and trial holes.

14.2 Statutory obligations. The highway authority should be consulted at an early stage in the preliminary investigations for a trench to be excavated along a highway. If the work is to be carried out by a statutory undertaker, it is necessary to give notice of intention to the highway authority and to receive its prior approval for the position and depth of the trench, the width of carriageway to be kept open for traffic and the procedure for reinstatement of the surface. The highway authority may require the undertaker to excavate works in controlled land abutting the street or road instead of in the street, if, for any reason, it may be necessary to close a road to through traffic, early application should be made to the highway authority.

If the trench on a highway or other land is to be opened on behalf of a non-statutory undertaking, approval has to be obtained either in the form of licence or by a wayleave. Under the provisions of the Public Utilities Street Works Act, 1950, a statutory undertaker has to give notice of proposed excavations to other statutory undertakers when their apparatus is likely to be affected, to afford them reasonable facilities for supervision, and to comply with their reasonable requirements.

Where the public has access to an excavation, the site has to be adequately fenced, and guarded and lighted so as to give proper warning to the public during the hours of darkness. Traffic signs are to be placed, operated and lighted in accordance with directions given by the highway authority or the police.

Legislation which may affect the excavation of trenches on a highway includes the Highway Acts, the Public Utility Street Works Act, 1950 and the Road Vehicles (Legislation and Licensing) Regulations, 1971.

Measures to be taken to ensure the safety of workmen, trenches and adjoining works are governed by the Construction (General Provisions) Regulations, 1961 and other Construction Regulations (see also clause **4**).

In the vicinity of certain ancient monuments, historical buildings and beauty spots, private Acts of Parliament may be in force regulating or prohibiting any excavation work. The planning authority should be consulted where restrictions apply. Permission is required from the British Railways Board whenever excavation work is to be undertaken beneath or adjacent to a railway.

14.3 Excavation procedure

14.3.1 *Methods available.* The principal ways in which a trench may be excavated are:

a) a short section of trench is opened at a time and the work is carried out by a single gang. This is the normal method for small pipes and sewers;

b) a longer section of trench is kept open to enable a number of gangs to carry out all stages of the work concurrently. This spread method of working applies to pipelines in open country;

c) the excavation for the full length of the trench proceeds stage by stage from top to bottom, frames being added as required until the final depth is reached. This is the normal method employed in digging for foundations at a considerable depth on a confined site, particularly in areas where there is surrounding property which must not be disturbed, when consideration should also be given to the need for underpinning the existing foundations.

It is sometimes advisable for a part of the work to be carried out in heading or by thrust boring between open trenches or between pits sunk on the line of the work. This course may be adopted when obstructions on the line of trench cannot be disturbed, or where the depth is too great for open trenching to be practicable.

14.3.2 *Trenches with sloping sides.* Any width or depth of trench may be constructed with sloping sides, provided that:

a) the nature of the ground is suitable and the sides of the trench can stand up at a stable angle without support for the required time;

b) dewatering of the ground can be effectively carried out to prevent the sides slipping or the trench flooding. This applies especially to water-bearing granular soils; and

c) the permanent work can be installed safely in the trench. In this connection it should be appreciated that the permanent work, e.g. a pipeline, may be subject to greater vertical loading and have less lateral support in a trench with sloping sides than in one with vertical sides.

If these conditions apply, all the excavation can be carried out mechanically by dragline, backacter, or scraper. This type of trench involves excavating and storing considerably more spoil than trenches with supported sides, but savings in cost and time can be made when using such mechanical equipment.

A support-free trench is advantageous where long pipes are to be laid or where large cast-in-situ culverts or similar works have to be constructed. Figure 30(a) shows a trench of this type up to 5 m deep, and Figure 30(b) a similar trench between 5 m and 10 m deep. The dewatering system shown in this illustration would be adopted only if the trench were excavated below ground water level in a granular soil.

14.3.3 *Trenches with vertical sides.* Supports should be provided for all vertically sided trenches more than 1.2 m deep which men are required to enter. Support may be necessary for shallower trenches where the system of work requires operatives to work substantially below surface level, e.g. kneeling or lying. Supports should be inserted in a manner that does not involve risks to operatives due to instability of the sides of the trench. Details of various methods for supporting vertical sides are given in **13.1**; in bad or wet ground it may be necessary to drive sheet piles or poling boards before commencing the excavation. In addition, movable shoring devices have been developed for use in trenches. These are moved along or lowered into the trench as required and provide a shield for the men working in the trench. If spoil is to be deposited alongside the trench as the work proceeds, it should be done in such a manner that it does not form a hazard in itself, or cause the trench sides to become unstable.

14.3.4 *Bottoming of trenches.* Whatever the permanent work, some hand trimming is usually necessary in the trench bottom. In most soils, therefore, the main excavation should cease above formation level, leaving the remainder to be trimmed and shaped accurately to line and level. During this operation it is desirable to avoid trampling or otherwise disturbing the soil at the formation level, particularly in clays, silts and fine sands. Depending upon the purpose of the trench it may be helpful in such soils, therefore, to lay gravel, broken stone or weak concrete as soon as the formation is exposed to form a protective layer. Ground water should meanwhile be kept below formation level.

In rock it is usually necessary to excavate to below formation level and then to place uniformly compacted sand or other fine granular material, or concrete, to produce a true bottom. Where the longitudinal gradient is steep, the material has to be sufficiently coarse to resist erosion by a permanent flow of ground water along the base of the trench.

14.4 Mechanical excavation of trenches. When mechanical excavating machines as described in Appendix A are used, the full depth and width of a trench is made in one cut from the surface to the bottom of the excavation, but the final trimming to levels is usually completed by hand. Temporary supports to the sides of the trench should be placed as quickly as possible as excavation proceeds. It may be advantageous to use a movable shoring device (**13.1.1**) for this temporary support. Operatives should not enter the trench until adequate protection has been provided. Ladders should be available at frequent intervals to allow safe access to the trench and easy egress in an emergency.

14.5 Hand excavation of trenches. When conditions make it impracticable to excavate mechanically, hand excavation becomes necessary. Such conditions include the following:

a) ground too steep for a machine, or working space restricted (as in narrow streets);

b) road and railway crossings where a machine would interfere with traffic;

c) sites where cables, mains, drains and other obstructions are known to exist;

d) paved surfaces or lawns where damage to the surface by a machine cannot be tolerated;

e) very bad ground which is incapable of supporting the weight of a machine;

f) when the job is small or where for any other reason it is more economical to use hand labour.

14.6 Methods of dewatering trenches. The various methods of removing water from excavations referred to in **12.2.3.3** are generally applicable to trenches (see CP 2004).

The method to be adopted, including the type and size of pumping equipment, depends upon the nature of the ground and the volume of water to be dealt with. It is important to ascertain whether the flow of water is intermittent or continuous and whether the rate is constant. If the work is near tidal water, observations should be made to determine whether the ground water level varies with the tide.

If there is a continuous flow of water, a sub-drain with sumps at intervals may be installed below the permanent work to keep the trench dry. These sub-drains should be so positioned and constructed as not to weaken the permanent work. Wherever possible excavation should proceed in an upgrade direction so that water drains away from the working face.

In excavating trenches in water-bearing soils, the well-point system of ground water lowering is particularly useful where the ground is of suitable grading or structure.

Ground water should not be allowed to accumulate in the bottom of the trench, and pumping facilities should be provided to deal with any excess ground water or temporary flood water. If water is allowed to accumulate in the trenches, there is a risk of displacement of completed work, including damage to timbering and lifting of pipes. The disposal of water from trench excavations may require measures to prevent pollution of watercourses. Approval from the statutory authority is required before discharging water into a sewer or watercourse.

14.7 Installation of permanent work or materials in trenches. Small drains or pipes and small quantities of concrete are usually lowered and placed by hand. Heavier units can be placed with the help of tripods or travelling gantries fitted with hand-operated chain blocks and tackle; with very long pipes "soldiering" arrangements are made in the timbering for the temporary removal of struts (see **13.1.7**) to allow the pipes to be lowered and placed in position (see Figure 31). Alternatively, the timbering system can be designed to permit the insertion of long pipes without the need to remove members.

Heavy items may be placed with mobile power-driven cranes travelling alongside the trench. In some instances the excavator can be used for installing the permanent work in the trenches provided that the machine is designed to be capable of safe operation while undertaking this task.

14.8 Backfilling and reinstatement of surface. Backfilling and compacting around any pipes or conduits should be done by hand, using selected materials, to a height at least sufficient to prevent damage by subsequent compacting operations (see also **13.2**).

The remainder of the backfilling is then completed in layers which are levelled by hand and thoroughly compacted by mechanical equipment. The maximum thickness of layer should suit the nature of the backfill material and the type of compaction plant, having regard to the acceptable settlement of the reinstated ground surface.

Bulldozers may be used to push spoil back into a trench where subsequent settlement is acceptable; in such cases the fill is left high to allow for consolidation. Care is needed to avoid damage to permanent work and collapse of the trench sides.

In maintainable highways temporary reinstatement of surfaces for a period is often adopted to allow the final settlement of the backfilling before the permanent reinstatement of the surface is carried out. To minimize settlement the highway authority may require, particularly at important road crossings, a material that can be compacted to a dense, relatively incompressible mass for use as backfilling instead of the excavated material.

14.9 Special cases

14.9.1 ***Narrow trenches***

14.9.1.1 *General.* Narrow trenches are used for cables, small pipes, trench fill foundations and land drains. They are generally shallow, are not entered by workmen, and require no support to the sides if the cables or pipes are laid immediately after the cutting of the trench. They may be excavated by hand using special narrow spades, by mechanical trencher, or by plough-type trencher.

14.9.1.2 *Hand-dug trenches for land drainage work.* These trenches are dug with tapered sides, and special tools are used for each spit. The first cut is made with an ordinary spade or graft and the second with a long tapered spade. Lastly, a half-round scoop is dragged horizontally to form a shaped bottom to the trench to fit the particular size of pipe. If the pipe is to be surrounded by granular material, a practice to be preferred, this last operation may be omitted. The pipes are placed in position by means of a pipe hook (wooden shaft with metal hook at end).

14.9.1.3 *Trenchers for narrow trenches.* There are two main types of trencher for narrow trenches.

a) *Mechanical trenchers.* The machines described in **A.1.4** and **A.1.5** are suitable for excavating narrow trenches. Trailing pipe layers in the form of open chutes may be used for laying agricultural pipes.

Backacters (**A.1.1**) may be equipped with special narrow bucket equipment, the spoil being ejected from the buckets mechanically.

b) *Plough-type trenchers.* These vary widely in type and weight and are drawn by a self-anchoring tractor-mounted winch, by a heavy diesel-engined winch or by a crawler tractor (direct haulage) as appropriate. Some machines aim at excavating the full depth in one cut, others require a number of runs.

14.9.2 *Trenches for docks, quays and similar retaining walls.* Figure 32 shows a method of trenching in sloping ground either partly or wholly submerged, such as might be found on the bank of a river or in tidal waters. The steel sheeting is driven and the trench excavated, timbered and diagonally braced to prevent distortion under varying pressures. A land anchorage may be required to resist the earth pressures from the land side when the level of water falls. Alternatively, fill can be placed over the area of the permanent work and brought up to form a working platform above high water level. Trenches are then excavated through the fill to the required depth in the underlying natural ground. Support to the sides of the trenches is given by means of a bentonite slurry. The concrete walls of the permanent structure are then cast in place through the slurry, or are inserted in the form of precast concrete units to the required configuration.

15 Pits and shafts

15.1 General. Pits and shafts are usually constructed on the principles described in clause **13**. Methods of excavation are governed by the confined space and by obstructions caused by timbering. These reduce the effectiveness of mechanical plant and may necessitate the adoption of specially designed equipment. Hand excavation followed by loading the spoil into buckets or trays may be more economical than machine excavation in small pits and shafts. The use of drilling equipment for sinking shafts is described in **15.4**.

15.2 Methods of support of excavations. The normal methods of support described in **13.1** are used. The choice of poling boards, runners, horizontal sheeting or sheet piling depends on the depth of excavation, the soil or rock conditions, and the purpose for which the pit or shaft is being dug. The top of the pit or shaft should be made large enough to allow for the combined set-in of all the waling frames where runners, or combined poling boards and runners, are used. Guard rails, handrails or other safety measures are necessary around the pit or shaft at ground level.

The chief difference between timbering of pits and shafts and that of trenches is that in the former "end" timbering is present in each successive frame, so that the walings which are "caught" at their ends by the abutting walings act as struts [Figure 33(a)]. Where intermediate struts are necessary to support the walings on the long side of rectangular pits or shafts, they can conveniently be provided as diagonal members restrained from movement by liners. This arrangement of struts gives a clear space in the centre of the pit or shaft [Figure 33(b)]. Proprietary support systems incorporating hinged struts and walings with hydraulic jacking arrangements can also be used in pits and shafts.

Propping and lacing of pits and shafts follow trench practice except that in wide and deep pits intermediate props and diagonal lacings are sometimes needed.

In loose water-bearing ground, "runs" of soil from behind the sheeting may relieve the load on the walings and struts causing them to collapse. To prevent this happening the laced-up frames should be slung from long bulk timbers or steel beams placed across the top of the hole on to an adequate bearing. The "runs" of soil should be stopped by stuffing the gaps in the timbering with hay, straw, cement bags, or plastic sheeting.

15.3 Drainage. The various methods of removing water from excavations referred to in **12.2.3** are generally applicable to pits and shafts. In water-bearing ground the most favourable working conditions are achieved by maintaining one or more temporary pumping sumps about 0.5 m below the general level of the excavation at all stages. On reaching final excavation level the sump may have to be timbered or provided with a perforated steel lining surrounded by a graded gravel or stone filter in order to allow for long-continued pumping without loss of ground or clogging of the pump intake. It may be convenient to construct the sump outside the periphery of the main excavation.

The use of well-points or bored wells around the excavation for a pit or shaft has the advantage of drawing water away from the excavation thus giving drier working conditions. These methods of lowering ground water are described in CP 2004.

15.4 Striking of supports and backfilling. These operations generally follow large excavation practice as described in **13.1.9** and **13.2**. Care should be taken to avoid unbalanced loading either of the permanent structure or of the supports.

15.5 Alternative methods of sinking

15.5.1 *General*. Medium and deep pits and shafts can often be sunk economically by one or a combination of the methods described in **15.5.2** to **15.5.5**.

15.5.2 *Well sinking*. The permanent lining is constructed above ground level in concrete or brickwork on a reinforced concrete shoe (or curb) provided with a cutting edge. The soil from within the lining and below the shoe is removed by grabbing, air-lift pump or hand excavation while the permanent lining is allowed to sink in a controlled manner. The lining is extended above ground level as necessary to keep pace with the rate of sinking and to provide dead load to overcome skin friction on the outside surface of the lining from the surrounding soil. It may be necessary to increase the load on the lining by kentledge in the form of concrete blocks.

Alternatively, or additionally, the skin friction can be reduced by injecting a bentonite slurry into the annular space between the lining and the soil.

15.5.3 *Underpinning with segmental lining*. A concrete collar is constructed in-situ at ground level and a ring of precast concrete segments with bolted radial and circumferential joints is cast into the collar. Excavation then takes place within the ring by grabbing or hand methods. After the soil has been removed for the depth of a segment over the whole shaft area, a ring of segments is assembled and bolted to the ring above. Excavation then continues ring by ring to the base of the shaft. At intervals cement grout is injected behind the segments to fill all voids and to prevent the lining from sinking under its own weight and under dragdown forces from the loosened soil surrounding the shaft. This method can only be used in soil which can stand unsupported over the depth of the segment.

15.5.4 *Cast-in-place concrete lining*. This method is similar to that described in **15.5.3** except that concrete cast-in-place inside formwork is used instead of a precast concrete segmental lining. This method again requires the ground to stand unsupported while the formwork is assembled and the concrete placed. A formwork depth of 1.5 m is convenient for stable ground conditions.

In soft, loose or water-bearing ground the concrete can be cast-in-place in a ring trench excavated with support by a bentonite slurry to form a circular diaphragm wall. The techniques used for this form of construction are described in [60].

The concrete lining to the pit or shaft can alternatively be cast-in-place as a ring of touching (contiguous) or interlocking (secant) bored and cast-in-place piles as described in CP 2004. In the case of the diaphragm wall or the bored pile wall the excavation is taken out subsequent to completion of the lining. The cast-in-place diaphragm wall can be designed to be self-supporting without the need for walings or bulkheads. However, support in the form of ring walings may be needed in deep pits or shafts lined with bored and cast-in-place piles. The walings may consist of reinforced concrete sections cast-in-place or steel rings assembled inside the lining and packed off the piles with dry concrete well rammed into place.

15.5.5 *Steel lining installed by drilling methods.* A temporary or permanent steel lining can be installed using a casing oscillator machine with moving clamps which impart a semi-rotary and downward movement to the lining as the soil is removed from within the lining by grabbing. Alternatively the pit or shaft can be drilled by mechanical auger or grab, the permanent or temporary casing being installed as the drilling proceeds. In stable ground the pit or shaft can be drilled to its full depth without support, after which a steel lining can be lowered to the base of the hole if operatives are required to enter it.

Weak ground can be supported by a bentonite slurry which is pumped out after installing the liner. The permanent work can then be constructed within the steel liner which is either left in place or extracted by means of the casing oscillator or by jacking.

If the liner is extracted care has to be taken to avoid damage to the permanent work.

15.6 Use of ground treatment processes. The sinking of deep pits or shafts in soft, loose or water-bearing ground may, in suitable circumstances, be facilitated by one or more of the special ground treatment processes. These include the injection of cement or chemicals to stabilize the soil and support by compressed air or ground freezing. These methods are described in CP 2004.

16 Excavation in rock

16.1 General. Methods of breaking, loosening and excavating rock or other hard materials are chosen according to the conditions prevailing, the hardness of the rock, the volume of excavation and the equipment available. Possible methods are described in **16.2** and **16.3**.

Drilling and blasting is usually the most effective and economical method of excavating a hard, massively bedded rock formation. Reference should be made to BS 5607. Blasting may not be desirable where:

a) the site is enclosed or built up to an extent that protective measures against damage to buildings, services or other property through blast, vibration, flying material, and other hazards are not practicable;

b) the site is adjacent to thoroughfares where blasting would cause undue inconvenience to the public or stoppage of traffic;

c) the site adjoins buildings such as hospitals and schools where annoyance or health hazard would be caused to the occupants;

d) blast or vibrations might cause instability of slopes;

e) damage might be caused to excavation supports.

16.2 Rock excavation by mechanical or hand methods. Weak rocks or well-jointed and thinly bedded hard rocks in large excavation areas can be loosened by ripping mechanisms attached to tractors. In confined areas such as trenches or pits the ripping action can be effected by teeth on the bucket of a backactor excavator, by a toothed grab, or by a toothed rotary bucket auger.

Rock can be loosened by means of mechanical impact devices such as air-, hydraulic-, or diesel-operated hammers with chisel points, mounted on the bucket arm of an excavator or suspended from a crane.

Hand methods of loosening rock formations include the use of pneumatic breakers, picks, and wedges. When using mechanical hammers or handheld pneumatic breakers it is desirable to excavate in the form of shallow benches. The levels of the benches may be governed by the bedding of the rock and the mechanical equipment available.

16.3 Rock excavation by drilling and blasting. The techniques of rock excavation by drilling and blasting in, trenches pits and shafts are different from those used in quarrying. In the latter case the spacing of drill holes and their loading with explosives is mainly governed by the need to produce rock suitable for crushing and screening with the minimum of secondary blasting. In the case of excavations for, trenches, pits and shafts, the main requirement is to produce an effective blast with the minimum of explosive and the minimum of disturbance to the rock surrounding the completed excavation. In both cases care should be taken to cause the minimum noise and vibration and to avoid flying rock.

The most effective blast is achieved by drilling near vertical shot holes in rows behind and angled towards a free face, so that the force of the explosion is directed transversely towards the face, rather than upwards which would result in excessive flying rock. For general excavation work a face up to 6 m high gives economical employment of drilling equipment and explosives. Maximum effectiveness is achieved by using delay detonators in each row with the delay increasing with increasing distance from the face, thus keeping the burden or distance between the charge and the face to the optimum value for each row of holes (Figure 34). The use of delayed charges also minimizes the noise and vibration.

Overbreak around the sides of an excavation can be minimized and, in favourable rock conditions, awkward angles or re-entrants can be excavated by using one of several techniques known collectively as "controlled blasting", and including techniques known as pre-splitting, smooth blasting and cushion blasting. These techniques involve the drilling of closely-spaced holes along the excavation line, and spacing the charge over the length of each hole in a planned layout. Either all holes, or only certain of the holes, are charged, some methods using stemmed, others spaced charges. The charges in the line of holes are either detonated before drilling and charging the holes within the main body of the excavation, or are detonated at the same time as the charges within the main body of the excavation but using a system of delayed detonators.

When excavating in pits and shafts the centre holes ("cutholes") are fired first to blow the rock upwards. Then the concentric rings of holes are fired using delay detonators to blow the rock towards the crater formed by the cut hole charges. Finally the trimming hole charges around the periphery of the excavation are fired (Figure 35), unless pre-splitting has been used perviously to maintain the shaft profile.

In large pit excavations it is advantageous to keep the centre of the excavation at a lower level than the surrounding area, thus forming circular benches from which the rock is blasted towards the centre for removal by grab or shovel.

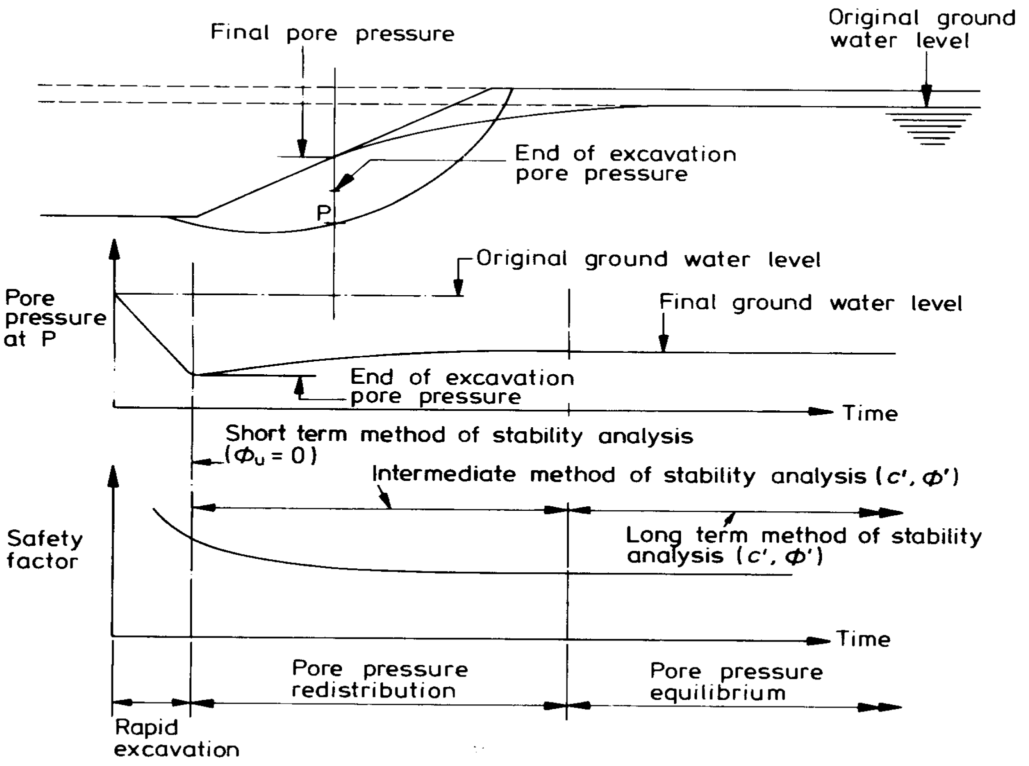
Special techniques for loosening rock by means of devices employing liquid carbon dioxide or by hydraulic bursters are available. Thermic lances can be used to drill close-spaced holes without noise or vibration, but care has to be taken to ventilate the atmosphere in confined spaces. The rock is then split along the row of holes into suitable sized masses for lifting out of the excavation.

For the most effective use of explosives with proper safeguards to the public and protection of property and the minimum of annoyance, expert advice should be sought. Such a service can be provided by the manufacturers of explosives.

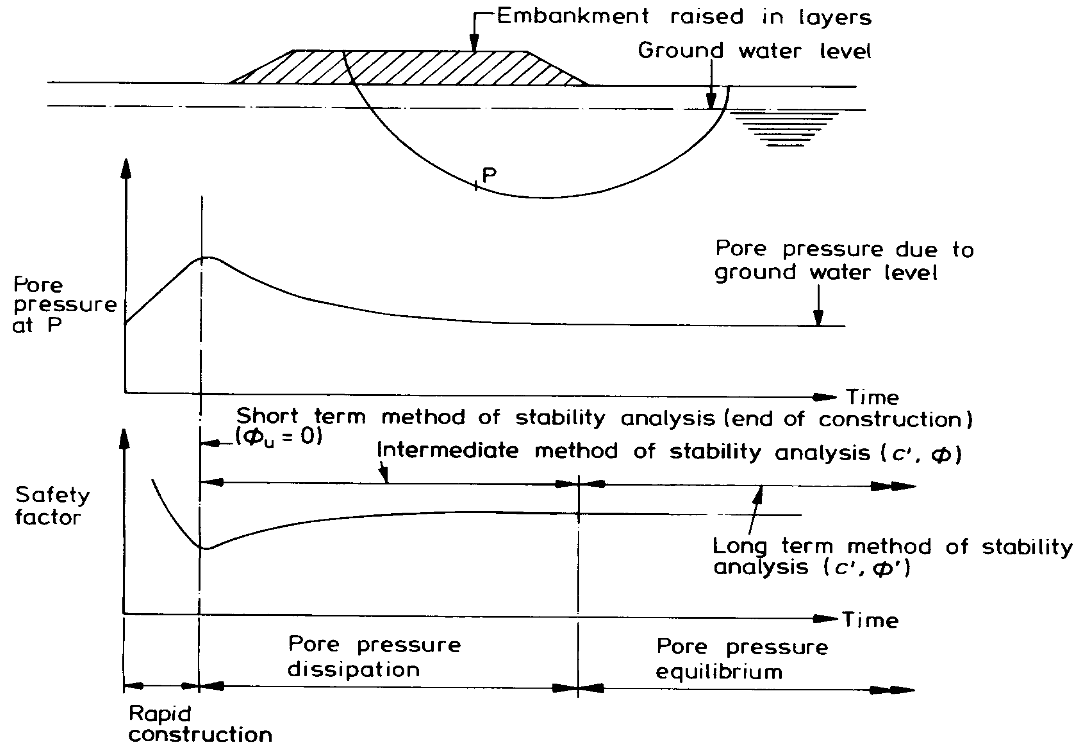
The storage and handling of explosives, the loading of shot holes, and the connecting and firing of the charges is to be undertaken with full regard to the safety of the operatives and the public and in compliance with the legislation governing the use of explosives.

In confined surroundings the ground surface over the charges should be covered with mats fabricated from sleepers, heavy wire netting, rope coils, or chains to prevent damage by flying rock. Adequate warning arrangements should be provided for all persons working in or passing through the danger zone, with control of vehicular traffic as necessary.

16.4 Removal of loosened rock. The rock loosened by ripping, by mechanical or hand-operated breakers or by explosives is removed by normal earth-moving equipment. Loosened rock in trenches is removed by backacter excavator, and in pits and shafts by grabbing. Loosened rock in large excavations is removed by front-end loading shovel or face shovel with suitable means of protecting operatives against material falling from the face. Alternatively it can be removed by dragline excavator standing on the bench above the cut.



(a) Stability of cuttings



(b) Stability of embankments

Figure 1 — Short and long term stability of cutting and embankment slopes

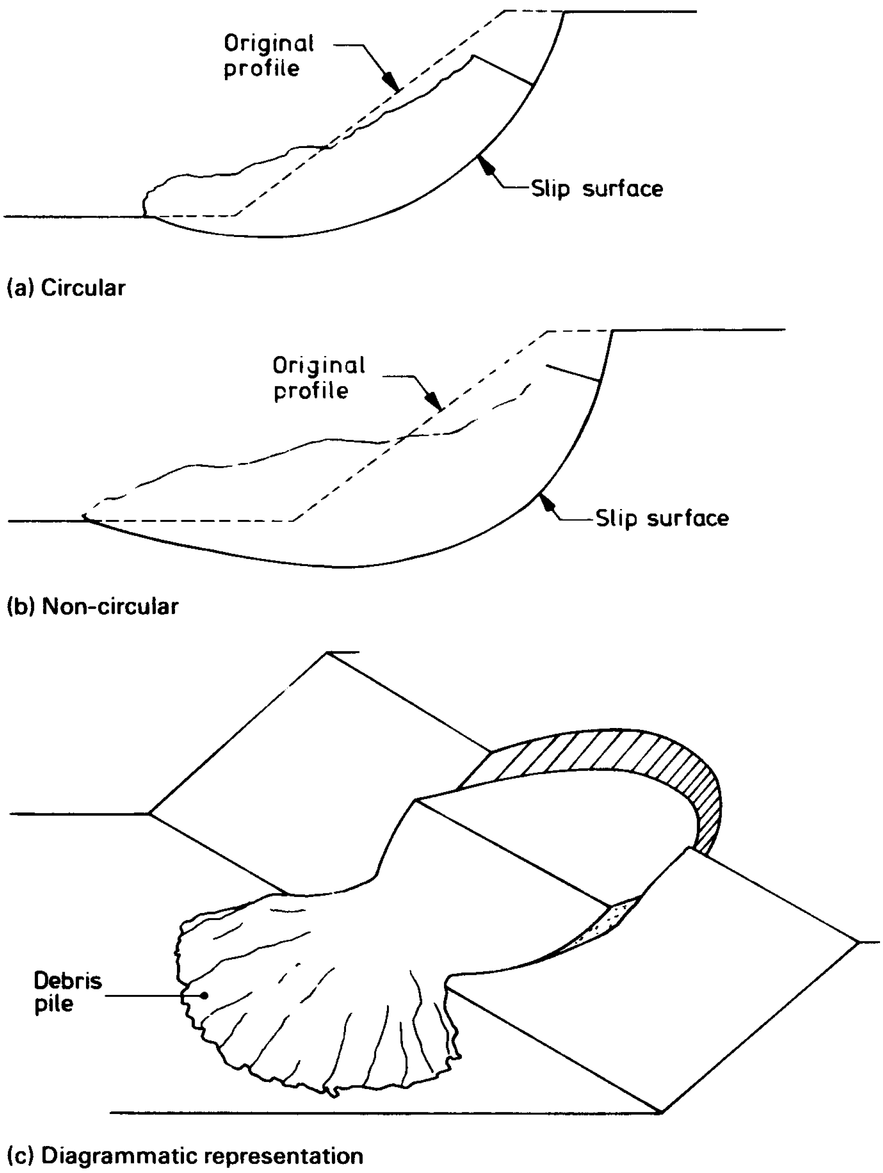


Figure 2 — Types of rotational slide

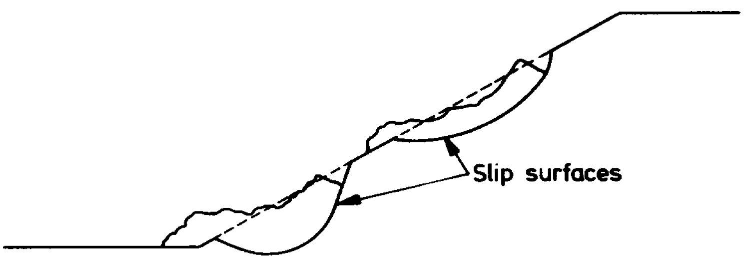


Figure 3 — Successive rotational slide

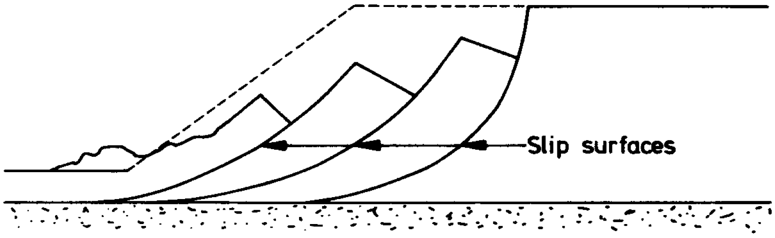


Figure 4 — Multiple retrogressive slide

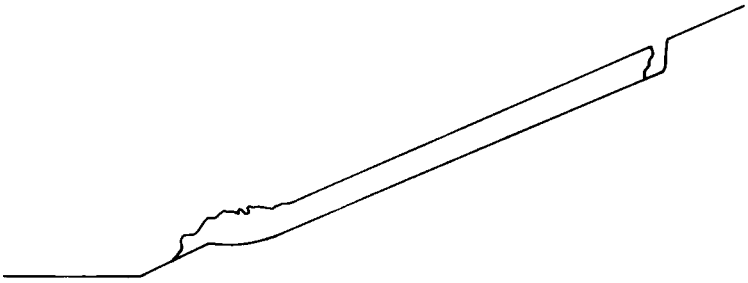


Figure 5 — Slab slide

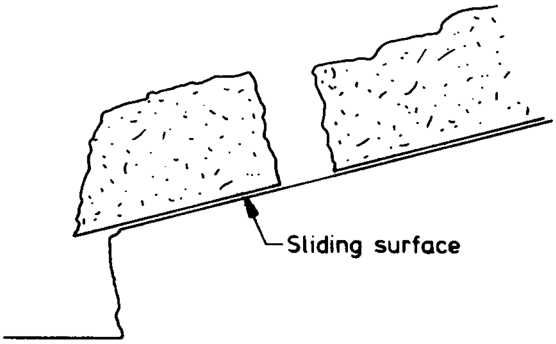


Figure 6 — Block slide

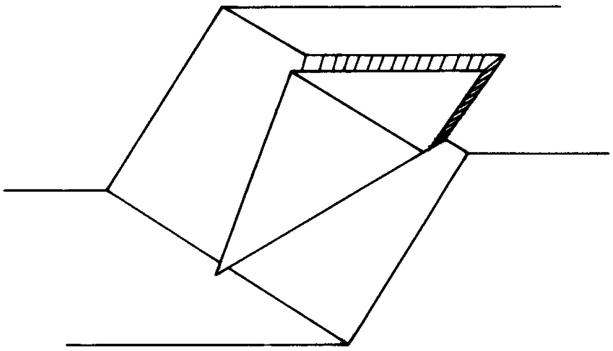


Figure 7 — Wedge failure

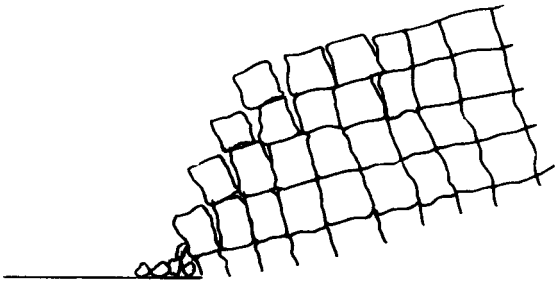


Figure 8 — Toppling failure

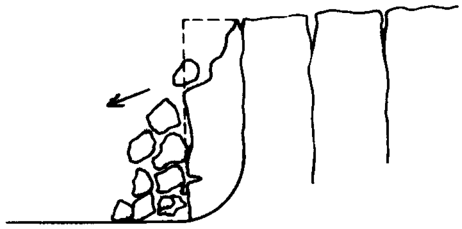


Figure 9 — Fall

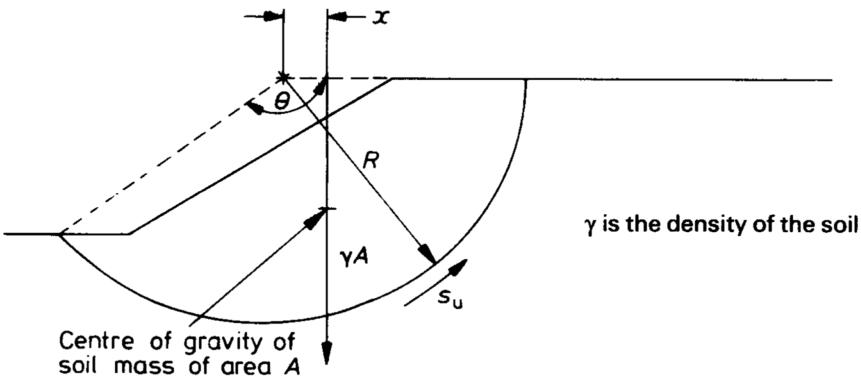


Figure 10 — Slip circle analysis (undrained shear strength s_u constant with depth)

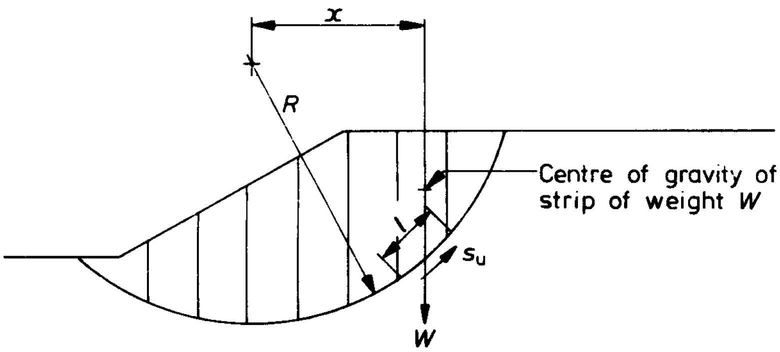
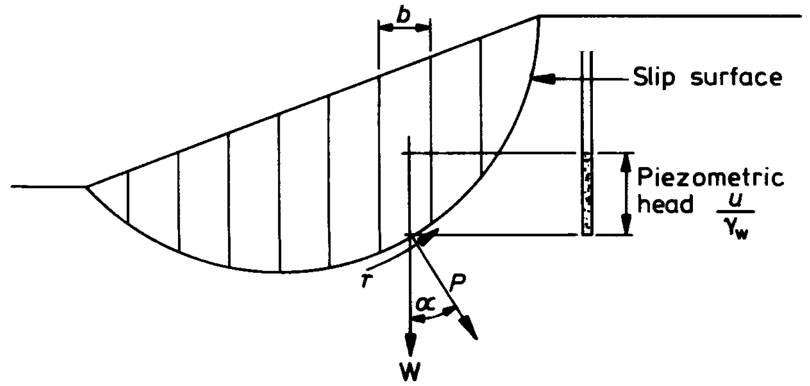


Figure 11 — Slip circle analysis (undrained shear strength s_u varies laterally and with depth)



NOTE An arrow head has been added to the bottom end of the force *P* line.

Figure 12 — Slip circle analysis (effective stress method)

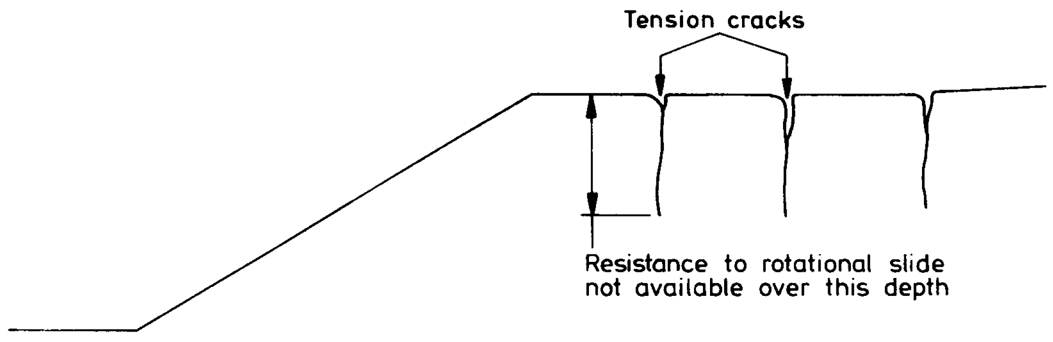


Figure 13 — Effect of tension cracks on slope stability

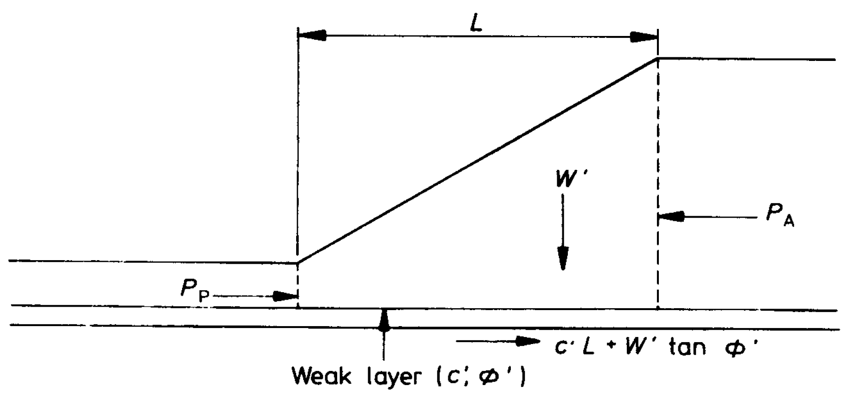
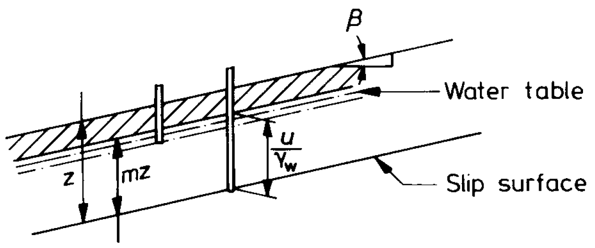


Figure 14 — Non-circular slide on weak soil layer



For limiting equilibrium

$$\gamma z \sin\beta \cos\beta = c' + (\gamma - m\gamma_w)\, z \cos^2\beta \tan\phi'$$

If $c' = 0$:

$$\tan\beta = \frac{(\gamma - m\gamma_w)}{\gamma} \tan\phi'$$

Figure 15 — Planar slide on infinite slope- stability analysis

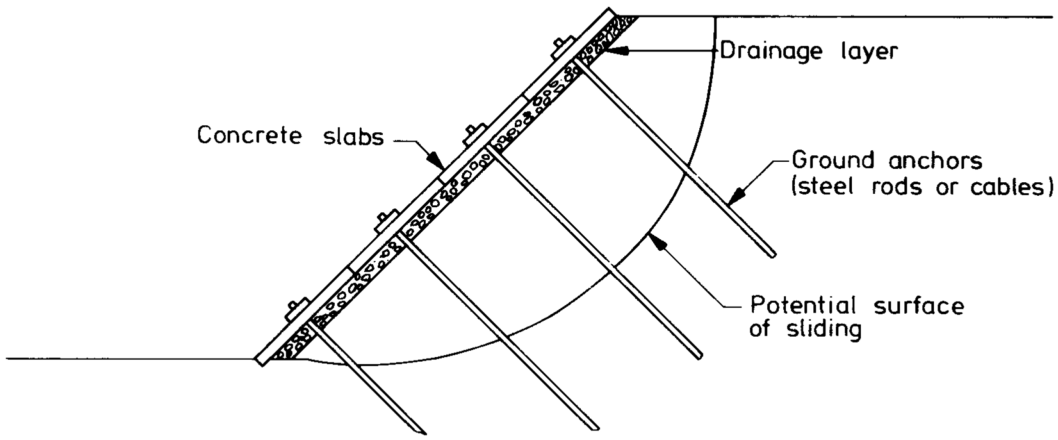


Figure 16 — Steep soil slope retained by ground anchors (see **6.5.5.3**)

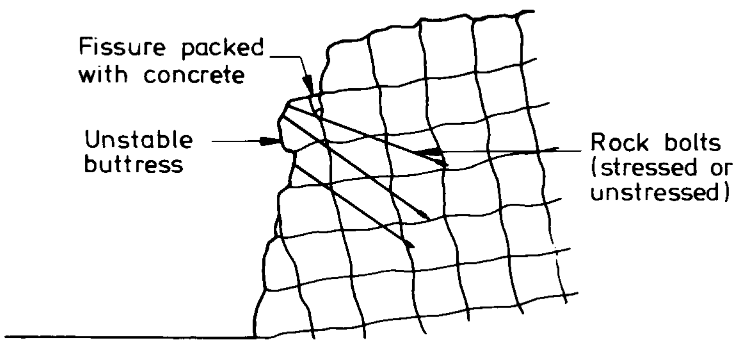


Figure 17 — Unstable rock slope stabilized by rock bolts (see **6.5.5.4**)

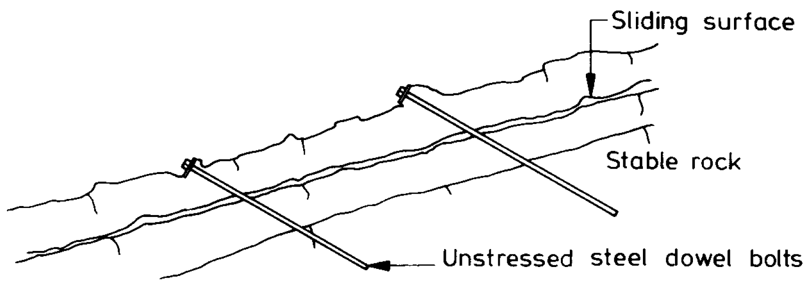


Figure 18 — Slab slide stabilized by dowel bolts (see **6.5.5.4**)

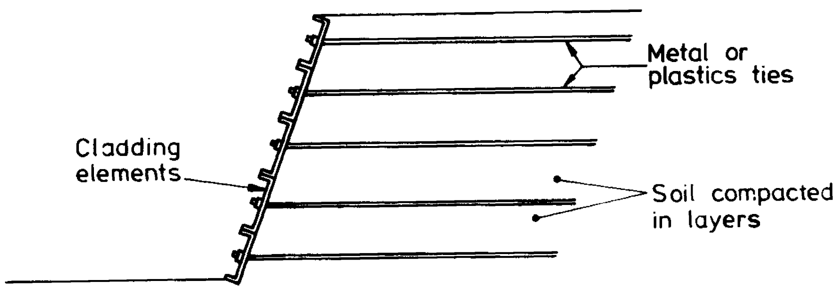


Figure 19 — Reinforced earth (see **6.5.5.2**)

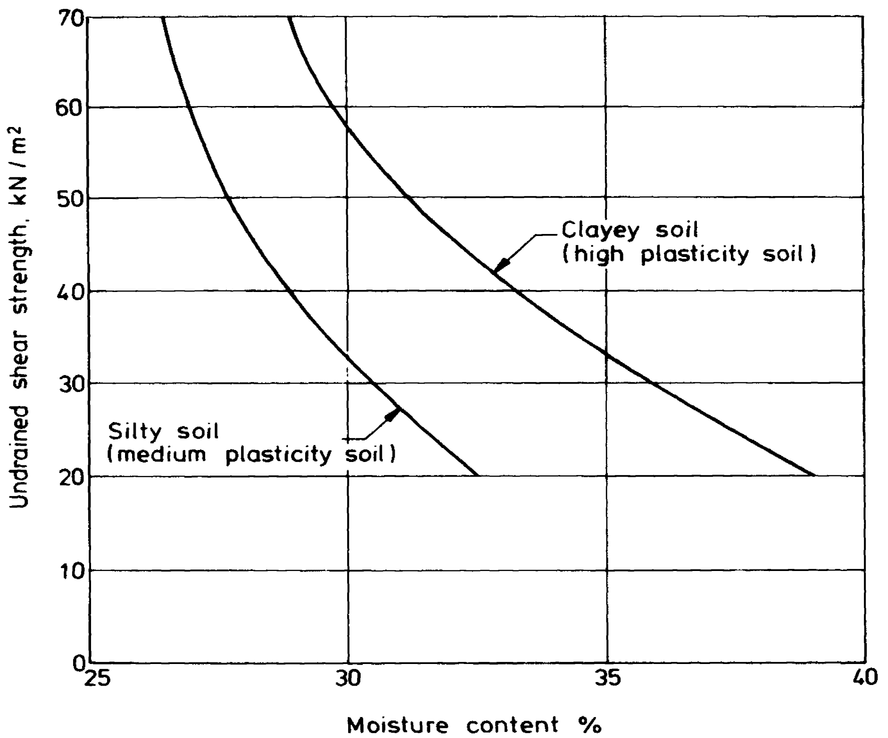


Figure 20 — Variation of undrained shear strength with moisture content for typical soils compacted using the 2.5 kg rammer method (BS 1377) at moisture contents above the optimum value (see **7.6.2.2**)

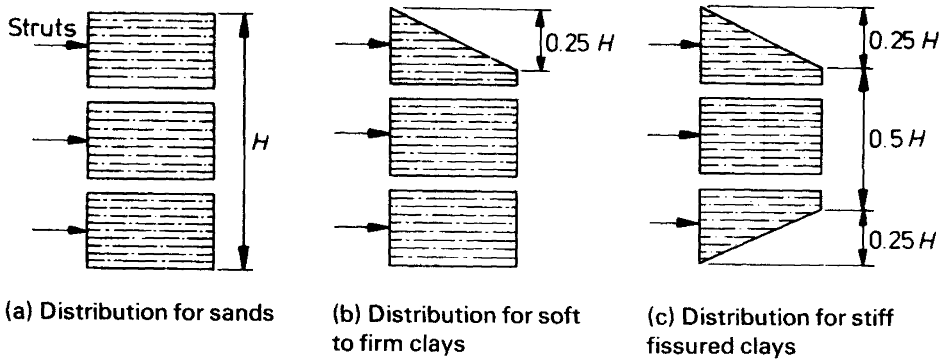


Figure 21 — Envelope of strut loads on braced excavations

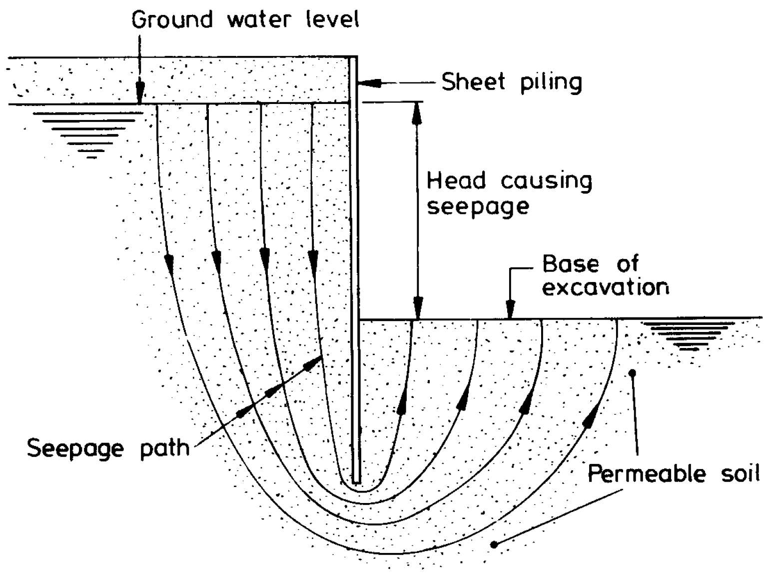


Figure 22 — Water-bearing permeable soils: seepage and uplift

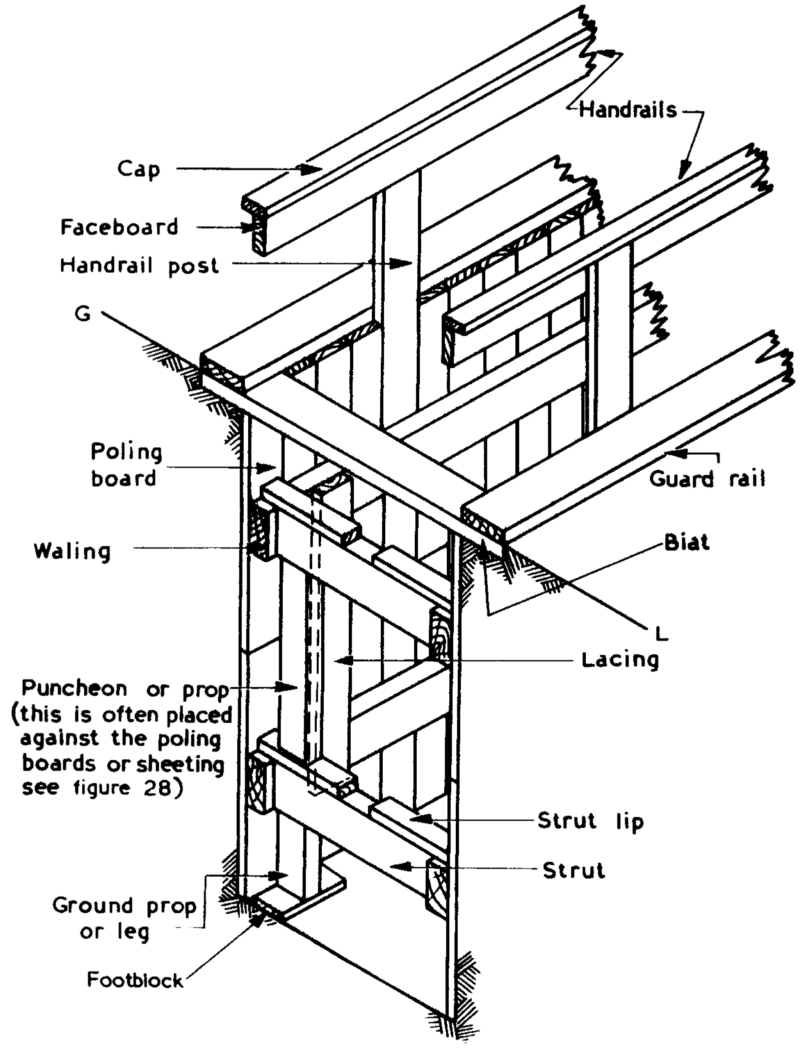


Figure 23 — Typical single or centre waling poling frame (see **13.1**)

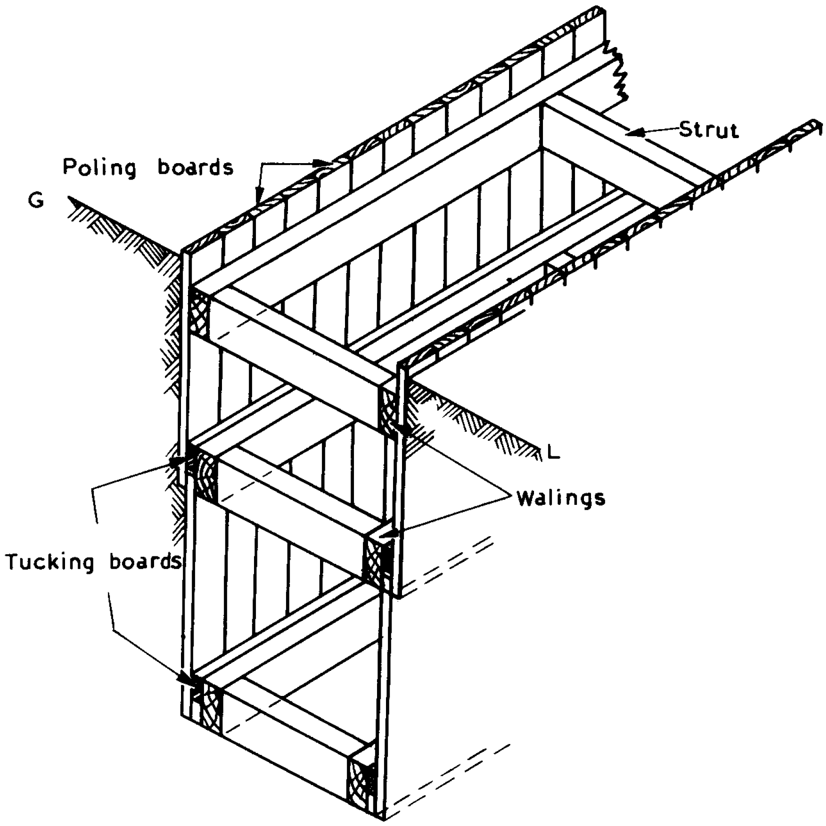


Figure 24 — Close poling with tucking frames (see **13.1**)

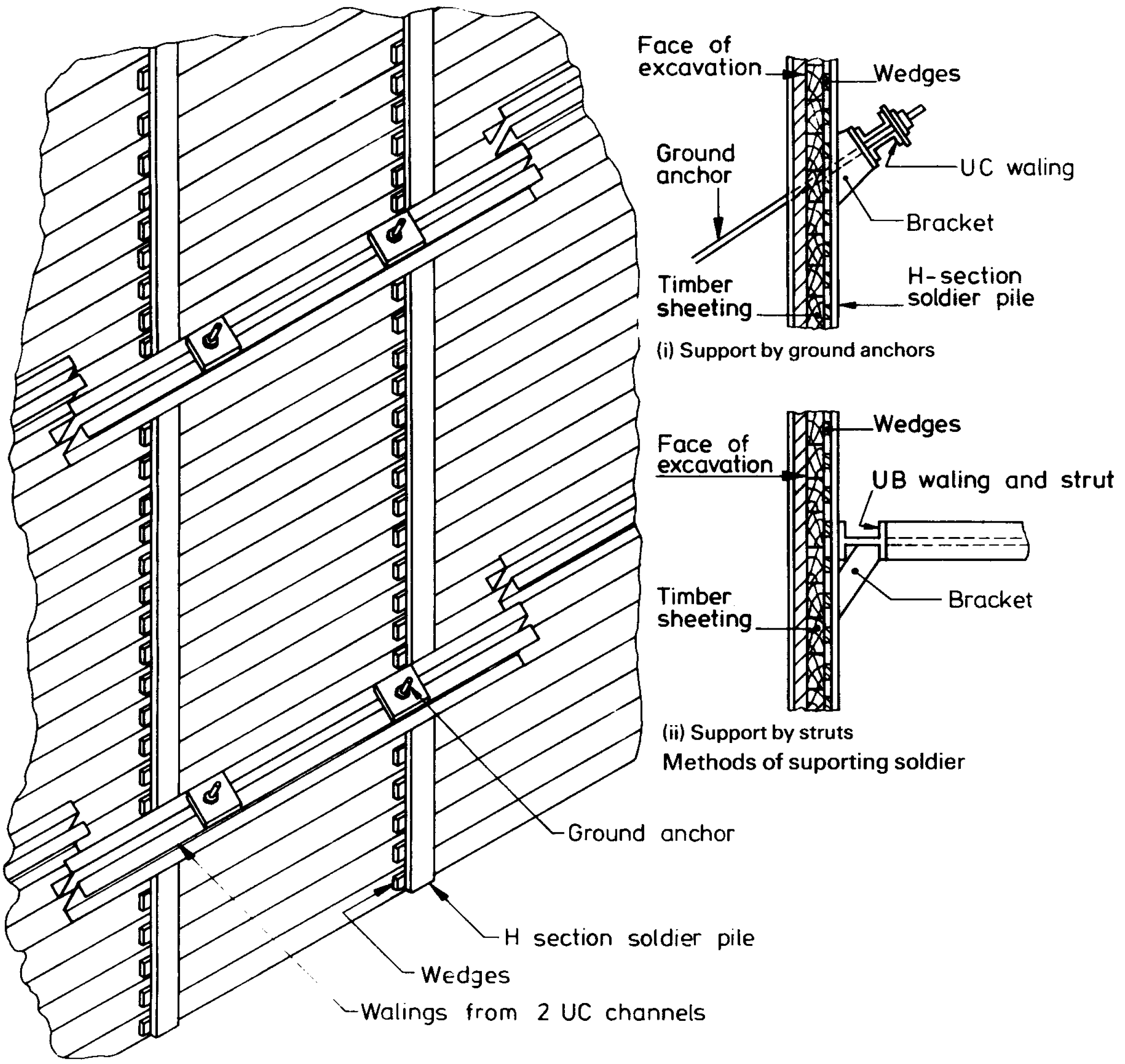


Figure 25 — Support of deep excavation by horizontal sheeting and soldier piles

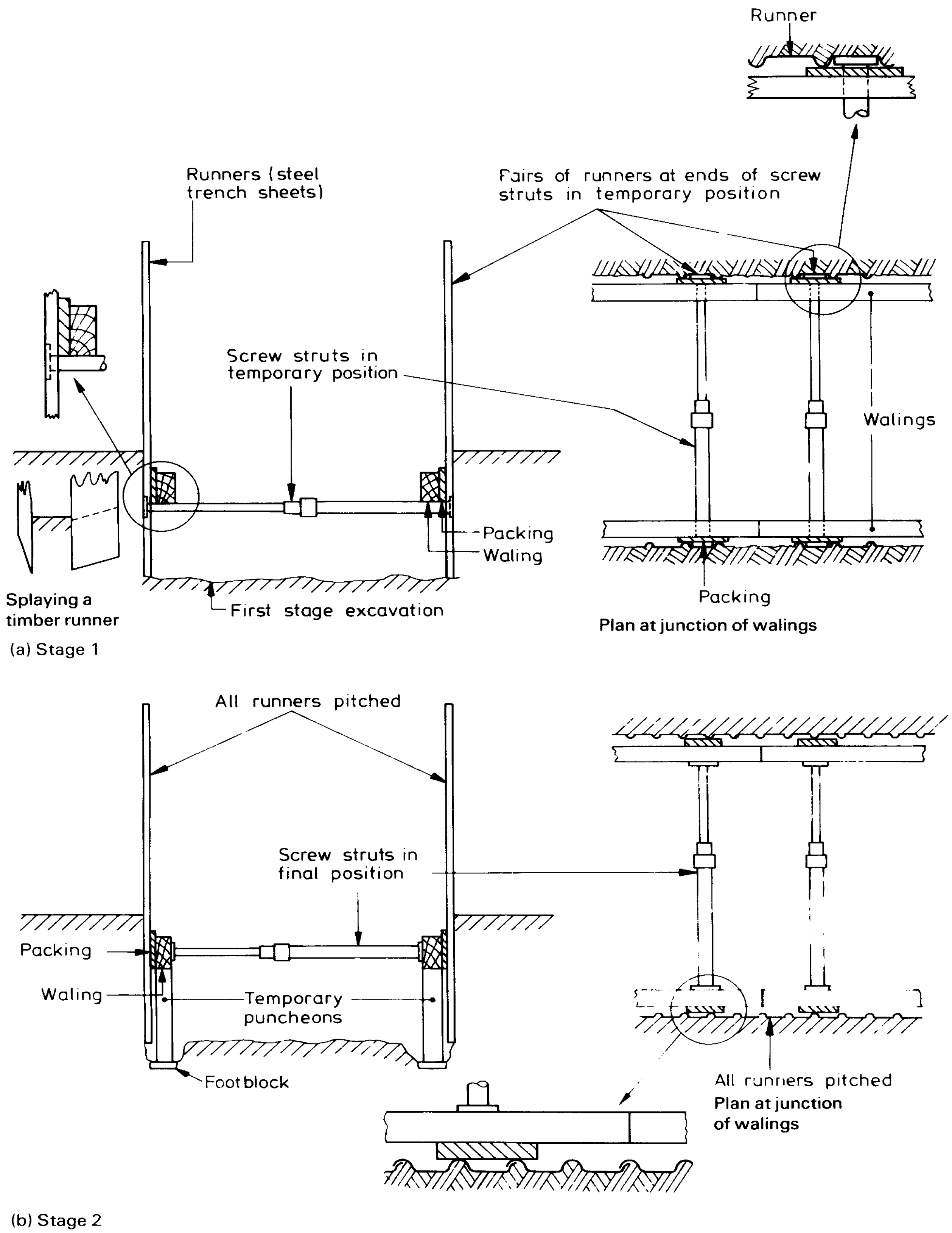


Figure 26 — Trench excavation using steel trench sheets as runners *(continued)*

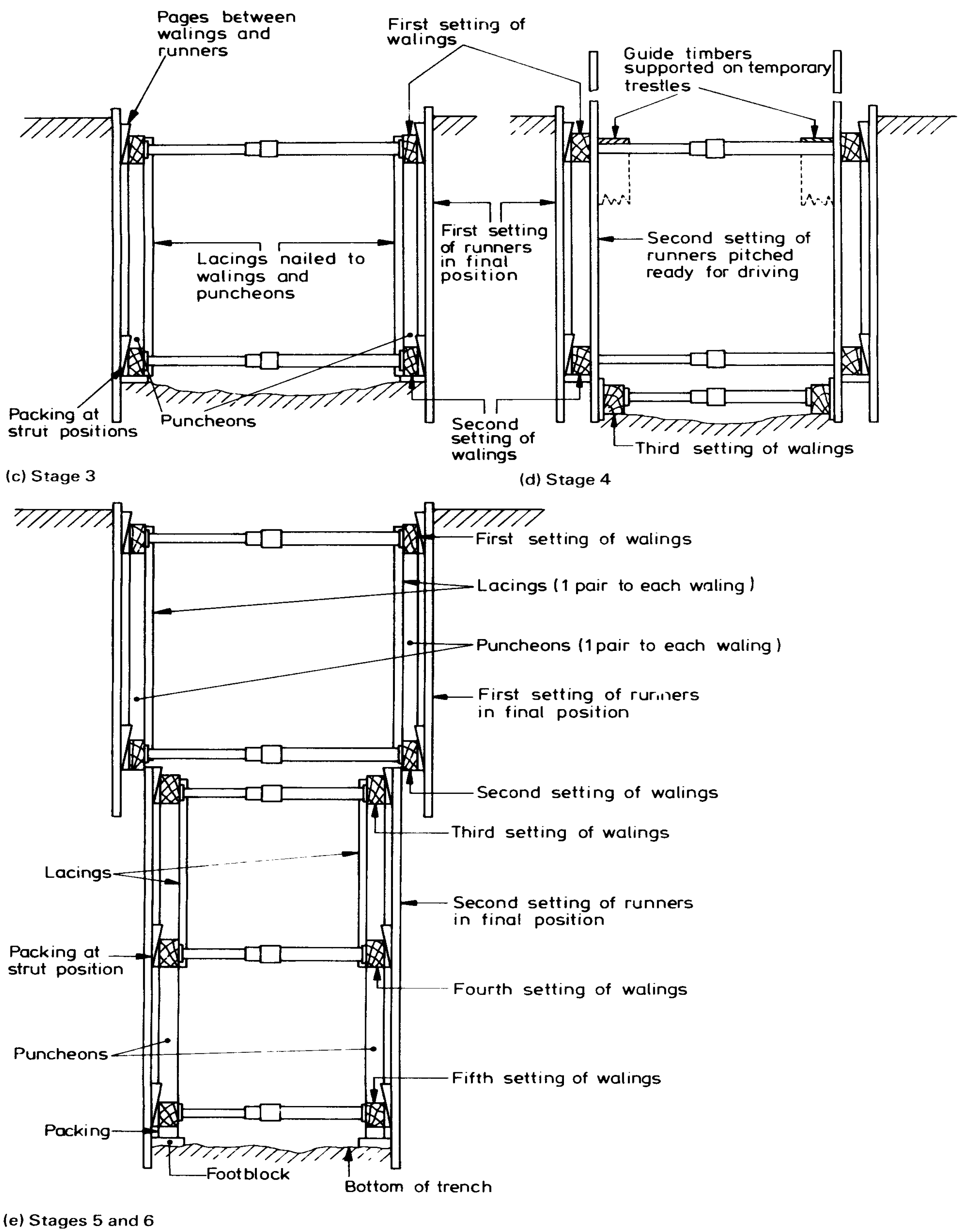


Figure 26 — Trench excavation using steel trench sheets as runners *(concluded)*

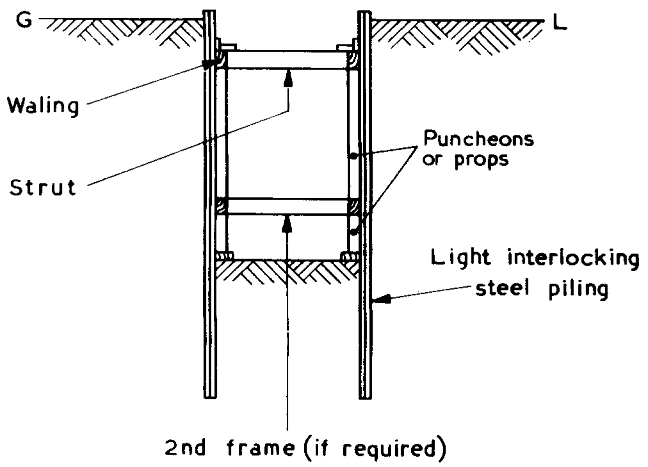


Figure 27 — Typical medium-depth trench with steel sheet piling in bad ground (see **13.1**)

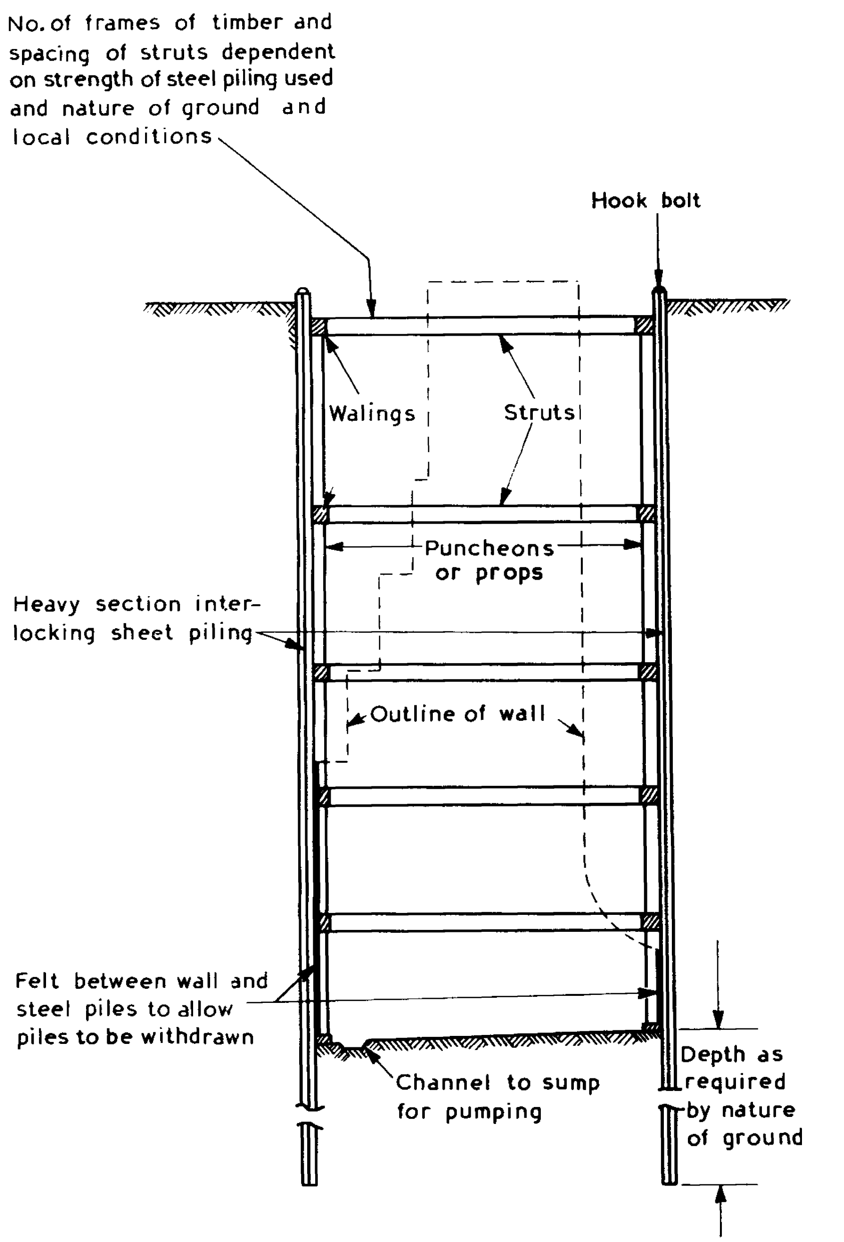


Figure 28 — Wide and deep trench using steel sheet piling (see **13.1**)

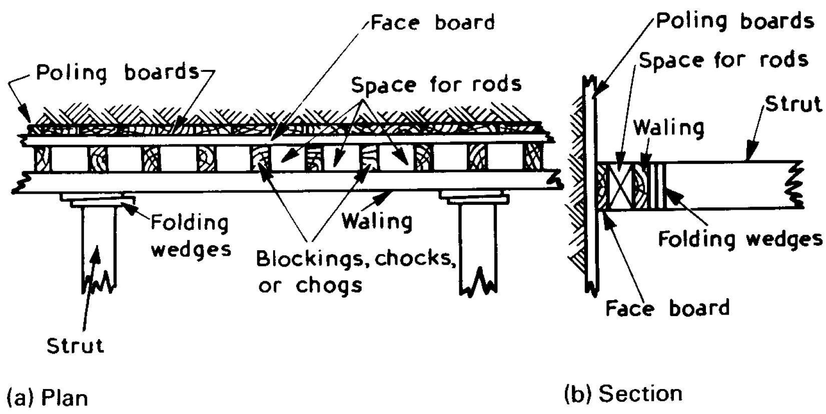


Figure 29 — Blockings, chocks or chogs

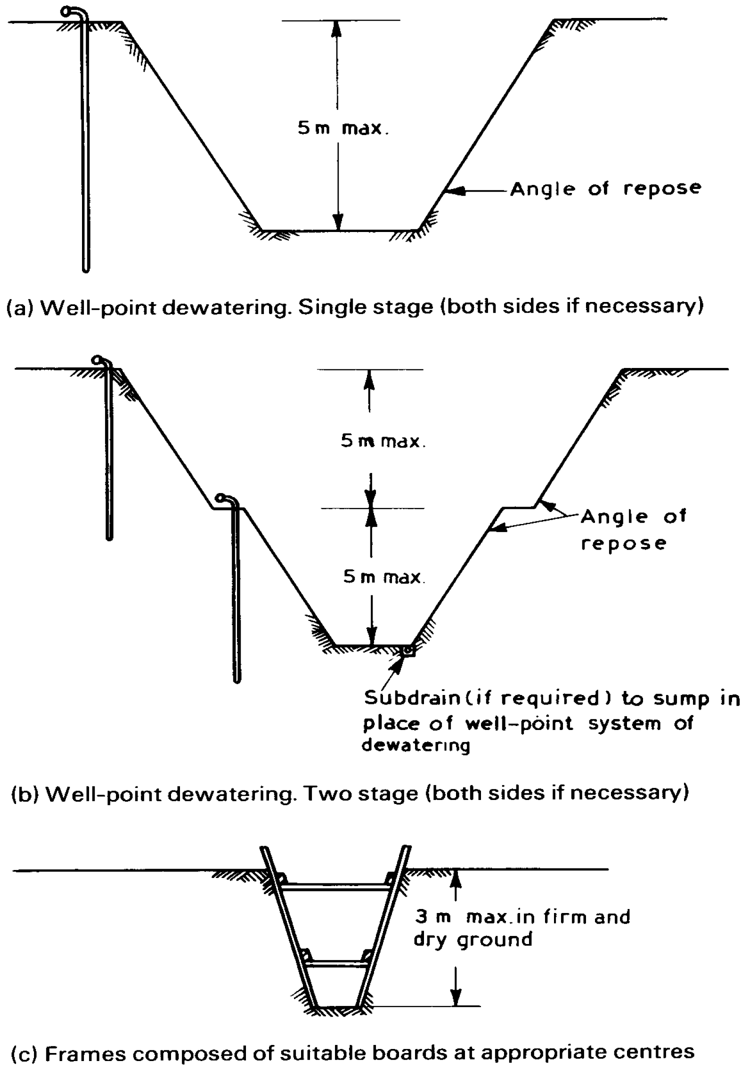


Figure 30 — Trenches with sloping sides (see **14.3.2**)

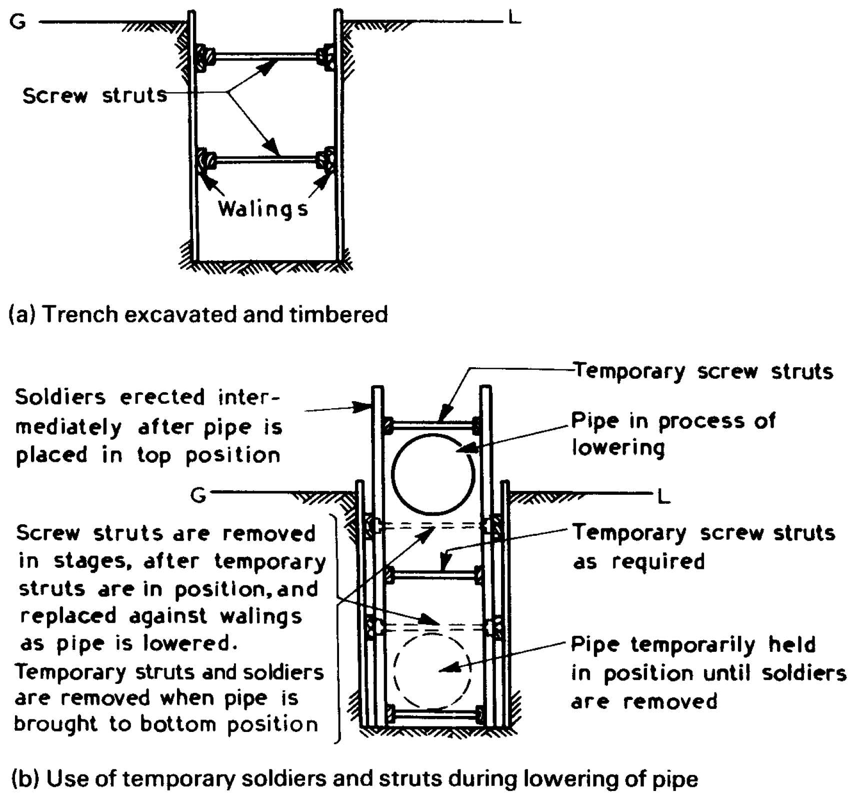


Figure 31 — Method of lowering long steel pipes by use of temporary strutting (see **14.7**)

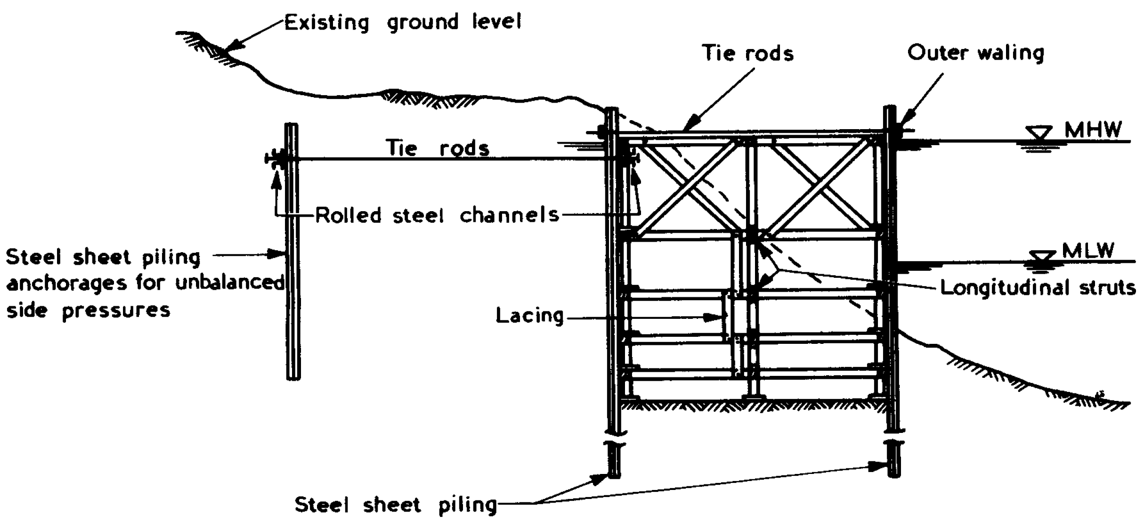


Figure 32 — Trench timbering for quay walls, etc. (see **14.9.2**)

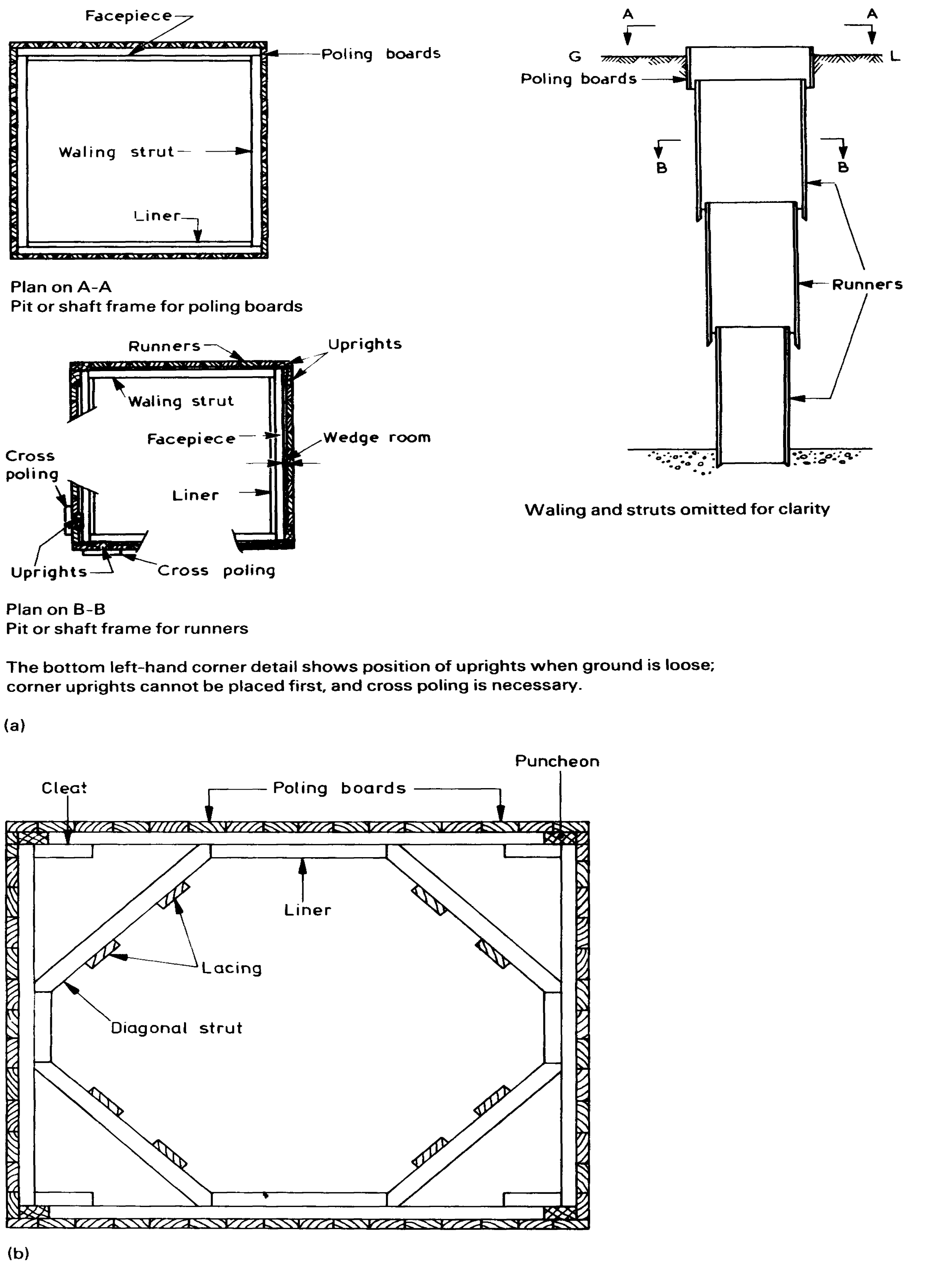


Figure 33 — Method of excavating isolated pits and shafts (see 15.2)

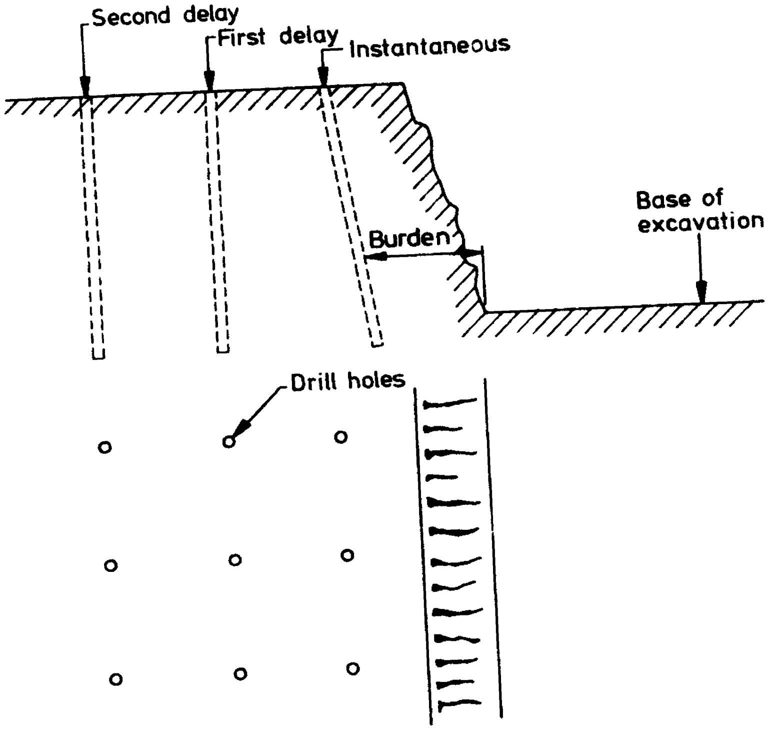


Figure 34 — Rock excavation by drilling and blasting: general excavation

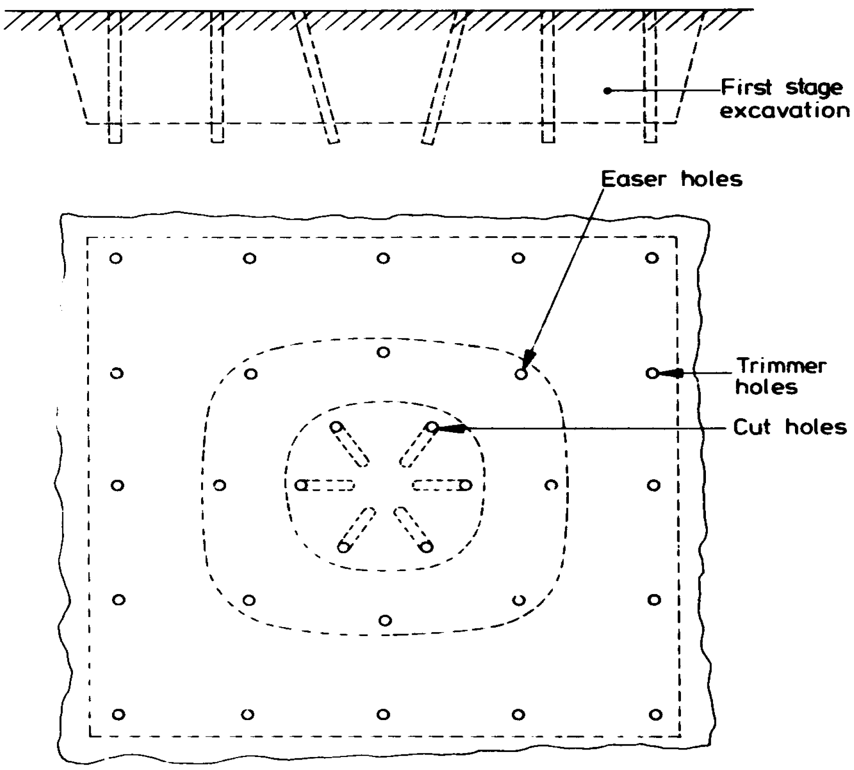


Figure 35 — Rock excavation by drilling and blasting: pits and shafts

Appendix A Descriptions of construction plant for earthworks

A.1 Excavation plant

A.1.1 *General purpose excavators*. General purpose excavators consist of a power unit, usually mounted on tracts, operating a variety of digging equipment located on jibs or hydraulically controlled arms. The principle types and functions of digging equipment are as follows.

a) *Dragline equipment* permits the machine to cast a bucket controlled by ropes and to excavate at a considerable distance below or in front of the prime mover. It can cast the excavated material to one side or load it into transport.

b) *Backacter or hoe equipment* consists of a bucket on the end of the jointed arm that digs back towards and mainly below the machine. It is usually employed to load into transport, and when used in a trench deposits material to one side.

c) *A face shovel* is a bucket fitted to a jointed arm in the reverse direction to the backacter. It digs into the face in front of and above the machine.

d) *A skimmer* has a bucket that travels horizontally along an arm to trim off the surface of the level on which the machine is standing. Backacters and face shovels are also used for this purpose.

e) *A grab* can be fitted to a basic machine having a jib and is used for mucking out from within timbered or sheeted excavations or cofferdams, or for dredging purposes. It can also be fitted to a basic machine having an hydraulically powered arm, and is then itself power-actuated. In this case the length of reach and the depth to which the grab can operate are very limited by comparison with grabs operated by jibs. In highly specialized operations such as pile excavations or the excavation for diaphragm walls, powered digging buckets are mounted on a special equipment mast fixed both to the jib and to a table projecting forward from the base of the machine.

The effectiveness of these different types of equipment depends on the stiffness of the soils being excavated. The more positive hydraulically operated machines are generally more efficient. Hydraulically operated equipment generally requires to stand closer to the excavation than equipment operated by rope. The most effective method of excavation is to use a face shovel digging forward and loading direct into transport standing alternately on each side of the machine. This requires that the excavation at all times should allow both the excavating plant and the earth-moving transport to operate readily over it without damage.

Dragline equipment is suitable for a wide range of materials but does not dig hard material as readily as the hydraulically powered face shovels or backacters. It is able to operate in a wide range of bucket sizes though the cost of the machine increases as the bucket size increases. The dragline is normally used in situations where its length of reach is of paramount importance, and in situations where excavation is required at the same level or at a level below that at which the machine and the earth-transporting vehicle can operate.

The backacter or trench hoe can be fitted with a wide range of buckets. This type of equipment is commonly used for trench excavation and in situations where excavation requires to be removed from the top or where the earth-moving transport is only able to operate at top ground level.

The use of the universal excavator fitted with these different attachments enables high utilization of the basic machine as it is adaptable to a wide variety of conditions.

A.1.2 *Front end loaders*. Front end loaders consist of purpose-built shovels mounted on track or rubber-tyred operating power units. Some front end loaders are basically bulldozers in which the bulldozing blade has been replaced by a bucket. Considerable use is made of rubber-tyred loaders in quarries where material is excavated from a face and transported a short distance by a loader to lorries or directly into reception hoppers leading by conveyor to the crushing or screening plant.

On many civil engineering projects the front end loader is widely used for cleaning access roads, moving materials from stock piles into trenches, and feeding materials between different parts of the site, all in addition to acting as a piece of excavating plant.

A.1.3 *Tractors and scrapers*. Tractors and scrapers can both excavate and transport material. Rubber-tyred scrapers can travel quickly over suitable surfaces and are economical over a considerable range of haul distances. However, when the conditions at the point of loading are very soft or slippery the scrapers may require pushing by a powerful track-laying tractor to load quickly and effectively. Some scrapers are fitted with powered elevating equipment to assist loading. When the conditions both for loading and for traversing are too soft for rubber-tyred plant, then track-laying tractors and scrapers have to be used. Because they are slow they are unlikely to be used for hauls of more than 300 m.

Most major earth-moving contracts involve the use of rubber-tyred scrapers, the sizes of which usually vary between 10 m^3 and 25 m^3 struck. In general, twin-engined machines driving all four wheels maintain traction far better on softer surfaces and steep gradients. The economical haul limit for these machines is usually no more than 2 000 m.

Tractors and scrapers can usually unload under their own power and spread the material to a required thickness and in the right position. This can reduce the amount of additional spreading equipment required. In exceptional cases they may require pushing assistance to unload.

A.1.4 *Continuous trenching machines*. These machines comprise an arm fitted with continuous chains to which are attached buckets of a width suitable for the trench required. These machines are very effective in consistent ground conditions. They are unable to cater for obstructions which may be met and coarse gravel or cobbles can affect their output. In the right ground conditions their speed of operation justifies their use.

Continuous trenching machines can be used for trenches ranging from those required for narrow vertical drains to wide trenches for pipelines up to 1 m in diameter.

A.1.5 *Bucket wheel excavators*. Bucket wheel excavators are a particular form of front loading machine. Digging buckets are attached to a revolving head at the end of an arm that points forwards from the base machine. They load onto a conveyor belt stretching down the arm and across the top of the machine, feeding a secondary conveyor belt having reception facilities at the back of the machine. This in turn may feed further conveyor belts or may load directly into earth-moving transport.

Bucket wheel excavators are designed for continuous digging performance and to be effective have to be of robust construction. The size and design of the machine should have particular relevance to the type of ground to be excavated. These excavators may be obtained in a variety of sizes and may be diesel or electric powered.

A.1.6 *Specialist excavating plant*. Other specialist plant includes dredgers and cutter suction equipment using air, water or bentonite lift, and water cutting/slusher devices. The latter employ high pressure water jets to wash down the face to be excavated to collecting equipment which removes the spoil usually by pipeline or conveyor belt.

A.2 Compaction plant

A.2.1 *Smooth-wheeled rollers*. Smooth-wheeled rollers are suitable for most types of fill. They are inappropriate for fill where a mixing-kneading action is desired and where smooth interfaces between layers should be avoided. However, they are useful for "sealing" the fill surface, i.e. giving a smooth surface to promote rainfall run-off, prior to temporary cessation of embanking. They can be self-propelled or towed, frequently by a track-laying tractor.

Three-wheeled road rollers may offer the choice of standard transmission with clutches for general work, or with torque converter or hydrostatic transmission to provide smooth, shockless drive. Roll diameter, load and rolling resistance are important factors for satisfactory performance. The weight distribution can be altered by adding sand or water ballast to the rolls or by attaching ballast weights to the main frames. Some rollers have combined steering on front and rear rolls to facilitate rolling right up to the inside and outside of curves. A constant roll overlap is maintained and there is less chance of scuffing the surface.

Tandem rollers usually have torque converter transmission, and there is a choice of machines with either a steerable rear roll or centre point steering. Centre point steering, although providing better tracking, is more expensive.

A.2.2 *Pneumatic-tyred rollers*. Pneumatic-tyred rollers are capable of compacting a wide range of soils from clays to granular fills. They are designed to give a kneading action to the material, the back wheels being out of line with those at the front to cover the work completely.

Most pneumatic-tyred rollers are self-propelled with an odd number of tyres ranging from 7 to 19 on two axles. Large tyres and wheel loads give greater effective depth of compaction, but since they are further apart some of the kneading action contributing to compaction is lost. Large tyres offer less rolling resistance and more contact area. The special tyres have to have a wide, flat tread since round-shaped tyres would squeeze material sideways. The tyre pressure has to be adjusted to suit the material and the provision of an air compressor is essential. In some machines, tyre pressure can be varied whilst the machine is operating.

Some rollers are provided with oscillating axles to prevent the bridging of low spots and depressions with the individual wheels following ground surface irregularities. These are sometimes referred to as "wobbly wheel" rollers.

Pneumatic-tyred rollers can be drawn by tractor or bulldozer or can be self-propelled. Self-propelled machines can either have a steerable axle or centre point steering. Roller weights can also be varied by attaching kentledge and wheel loads of up to 13 tonnes can be obtained. Those heavier wheel loads enable the compaction of thicker layers which can frequently reduce the number of passes required.

A.2.3 *Grid rollers*. A cylindrical roller the surface of which consists of a network of welded steel bars arranged in a rectilinear pattern is termed a grid roller. It is best suited to material requiring high contact pressures but little kneading action, for example, rockfill and most coarse grained granular soils.

A.2.4 *Sheepsfoot rollers*. These rollers comprise a hollow steel cylinder, to the rolling surface of which are attached rows of steel feet projecting from the surface.

The performance of the roller is a function of the number, arrangement and shape of the steel feet. These feet may be shaped in the form of tapers, cylinders or clubs and designed so that the compacted material is not lifted out as the feet withdraw. Sheepsfoot rollers are most commonly towed but where the use of specialist plant can be justified they may be self-propelled and fitted with a dozer blade.

During the first few passes the feet penetrate the loosely spread layer of fill and the roller weight may be carried directly by the soil beneath the drum or by the feet on the next lower layer. During subsequent passes the feet compact the soil in the lower part of the top layer so that the roller walks out of the layer, which progressively becomes denser and stronger and able to support the high pressures from the feet. The rollers are best suited to dry fine-grained soils especially where it is desirable to break up lumps of stiff clay and weak rock. The top of each last compacted layer is left in a loose or rough state, which facilitates good bonding between successive layers.

A.2.5 *Tamping rollers*. These are similar to sheepsfoot rollers but with a higher ratio of foot area to cylinder area, the foot area generally exceeding 15 % of the swept area of the drum. They are suited to the wetter fine-grained material, such as cohesive soil, at or above the plastic limit.

A.2.6 ***Vibratory rollers***

A.2.6.1 *General*. These are smooth-wheeled machines fitted with an engine-driven vibrating unit. Vibrating rollers are available with weights ranging from 150 kg to 15 000 kg and operate at pulse rates in the range of 1 100 to 2 000 pulses/min. These rollers are particularly effective for the compaction of coarse-grained materials such as sands, gravels and rockfills. The weight and vibration characteristics should be selected for their effectiveness on the soil and the layer thickness being treated, so that full compactive penetration is achieved. Vibratory rollers have a tendency to dig themselves in when operated on too thick a layer.

The principal types of vibratory roller are described in **A.2.6.2** to **A.2.6.8**.

A.2.6.2 *Single roll pedestrian-controlled rollers*. The principle of vibration is an eccentric shaft in the middle of the roll which is supported on flexible mountings.

These rollers feature simplicity of control, a clear side to allow for compaction against kerbs and a choice of petrol or diesel engines. Some machines have a high centre of gravity which renders control difficult on uneven terrain.

A.2.6.3 *Double roll pedestrian-controlled rollers (vibration in each roll)*. The advantages of a double roll over a single roll are easier control with no weight on the operator and the ability to achieve the same compaction in half the number of passes. It is common to have each vibration roll either supported independently in its own flexible mountings or rigidly connected to the frame. The latter method often utilizes vertically directed vibration produced by synchronized eccentric weights in the rolls.

A.2.6.4 *Tandem vibrating rollers*. These machines usually have a vibrating smooth rear roll and a non-vibrating deadweight smooth front steering roll. The operator rides on the machine, which has at least one projection-free side. Some large machines can operate at a high frequency and small amplitude for surface compaction, and at a low vibration frequency and a large amplitude for depth compaction. These machines can be used with or without vibration, so widening their field of application.

A.2.6.5 *Double vibrating rollers*. Double vibrating rollers feature all-roll drive and vibration. Some machines have centre point steering, hydrostatic transmission and a vibration system to produce vertically directed vibrations. On many of these machines the vibration system in each roll can be operated independently of the other if desired. Other double vibrating rollers have four rolls, mounted in a chassis in tandem pairs. In both variations the operator is seated on the machine.

A.2.6.6 *Towed vibrating rollers*. These rollers should have a large roll diameter to reduce rolling resistance and hence the power of the towing tractor. The tractor unit has to be as heavy as, or heavier than, the roller. The vibrating roller should be flexibly suspended to prevent the transmission of harmful vibrations to the roller frame. The roller frame has to be well balanced to allow easy coupling to the towing tractor. Control of the rear engine can be accomplished either mechanically or by means of an electrical control system with a push button control box in the towing tractor.

A.2.6.7 *Self-propelled vibrating rollers*. The large self-propelled vibrating rollers have a vibrating front roll, centre point articulated steering, and a choice of rear driving wheels. Depending on the application, the machines can be equipped with traction tread tyres, smooth tyres or smooth steel rolls. These machines are highly manoeuvrable and have good stability due to a low centre of gravity. They can be obtained with a facility for varying the frequency of vibration.

A.2.6.8 *Vibrating plate compactors and tampers*. Vibrating plate compactors consist of a steel base plate, with the edges turned upwards, on which is mounted either a fixed eccentric or a pivoted double-geared eccentric which is synchronously balanced. The pivoted eccentric mechanism allows backward as well as forward travel. The weights of these compactors range from 200 kg to 2 000 kg. It is important that the compactors should be easy to control and have a low centre of gravity.

Vibrating plate tampers have a similar steel base plate on which is attached a system of springs. These springs are activated by a hollow piston within the main engine housing. Inclining the main housing in the forward direction is sufficient to impart some horizontal motion to the tamper as it is vibrated.

A.2.7 *Power rammers*. A petrol-driven piston connects directly to the foot of the rammer. When the piston is fired, the energy is expended against the ground, which causes the rammer to jump, so there is compaction on the firing stroke and a comparable compaction when the machine lands. Good balance allows these machines to jump on the spot or, when given a slight tilt, to "walk" in any direction.

The weights of these rammers are of the order of 100 kg to 800 kg. They are most commonly used to compact small areas with granular soils, particularly in confined areas. They are susceptible to bogging if the soils which they are compacting are too wet or soft.

Appendix B List of references

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BS 5930, *Code of practice for site investigations*[6].
BS ...., *Code of practice for safety in tunnelling*[7].
BS ...., *Ground anchors*[7].

[4] Under revision.
[5] Referred to in the foreword only.
[6] Formerly known as CP 2001.
[7] In preparation.

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