

Code of practice for Earthworks

UDC 624.13

NO COPYING WITHOUT BSI PERMISSION EXCEPT AS PERMITTED BY COPYRIGHT LAW



Document Sharing Hub

Code drafting committee CSB/3 earthworks

Chairman	Mr M J Tomlinson
British Railways Board	Mr D J Ayres
County Surveyors' Society	Mr A Onions
Department of the Environment (PSA)	Mr R E Boshier
	Mr K O Pook
Department of the Environment (Building Research Establishment)	Dr A D M Penman
Department of Transport (Highways)	Miss S B Crutchlow
Federation of Civil Engineering Contractors	Mr O P Edge
Geological Society of London	Mr G Walton
Health and Safety Executive	Mr T R Mills
Institution of Civil Engineers	Mr J N Holmes
Institution of Highway Engineers	Lt Col F R Oliver, OBE
Institution of Municipal Engineers	Mr A W Bullett
Institution of Structural Engineers	Mr S Thorburn
	Mr B O Corbett
	Mr P Mason
	Mr N J Ruffle
Institution of Water Engineers and Scientists	Mr A J Southard
Royal Institute of British Architects	Mr J Dawson
Trades Union Congress	Mr R E Bloomfield
Co-opted Members	Dr J A Charles
	Mr G I Cottis
	Mr R M W Cotton
	Mr B Cox
	Mr C Garrett
	Mr A T Hutchinson, OBE
	Prof J N Hutchinson
	Mr D C Lindsay
	Mr E T Purcell
	Mr D W Quinion
	Mr H J Stevens
	Mr J S Tanner
	Mr J C Waddington
	Mr H V Wells
	Mr W T Sharp

This British Standard, having been prepared under the direction of the Civil Engineering and Building Structures Standards Committee, was published under the authority of the Executive Board and comes into effect on 30 June 1981

Secretary

© BSI 12-1999

First published (as CP 2003)
April 1959
First revision June 1981

The following BSI references relate to the work on this standard:
Committee reference CSB/3
Draft for comment 77/13223 DC

ISBN 0 580 11835 5

Amendments issued since publication

Amd. No.	Date of issue	Comments
4251	April 1983	Indicated by a sideline in the margin

Contents

	Page
Code drafting committee	Inside front cover
Foreword	viii
Section 1. General	
1 Scope	1
2 References	1
3 Definitions	1
3.1 General	1
3.2 Terms used in section 2	1
3.3 Terms used in section 3	2
3.4 Terms used in timbering	3
4 Safety procedures	6
4.1 General	6
4.2 Statutory obligations	7
4.3 Precautions against incidental hazards	7
Section 2. Cuttings and embankments, grading and levelling	
5 Site conditions and investigations	8
5.1 General	8
5.1.1 Environmental considerations	8
5.1.2 Preliminary assessment of the project	8
5.1.3 Ground exploration	9
5.1.4 Description and classification of soils and rocks	11
5.1.5 Field and laboratory testing	11
5.1.6 Report on investigations	11
5.1.7 Further investigation during construction	11
5.2 Economic and environmental considerations	11
5.2.1 General	11
5.2.2 Cut and fill for roads and railways	12
5.2.3 Cut and fill for airfields and other extensive paved areas	12
5.2.4 Dredging and hydraulic fill	12
5.2.5 Sand dunes	12
5.3 Risks of failure and acceptance of deformation	17
5.3.1 General	17
5.3.2 Effect on neighbouring structures	17
6 Cuttings	17
6.1 General considerations	17
6.1.1 Environmental factors	17
6.1.2 Site geometry	17
6.1.3 Economic and safety considerations	17
6.1.4 A guide for preliminary design of cuttings in rocks	18
6.2 Factors governing the stability of cutting slopes	18
6.2.1 Materials: soils and rocks	18
6.2.2 Selection of parameters of soil and rock masses for assessment of slope stability	22
6.2.3 Structure and fabric of soils and rocks	23
6.2.4 Water: surface and sub-surface	24
6.2.5 Time effects	24
6.2.6 Other factors influencing stability	25
6.3 Modes of failure of cuttings and natural slopes	26

	Page
6.3.1 General	26
6.3.2 Rotational slides	26
6.3.3 Compound slides	26
6.3.4 Translational slides	26
6.3.5 Toppling	27
6.3.6 Falls	27
6.3.7 Internal erosion	27
6.4 Methods of analysis of stability of slopes	27
6.4.1 General	27
6.4.2 Limit-equilibrium methods	27
6.4.3 Stress analysis	29
6.4.4 Physical models	30
6.5 Design	30
6.5.1 Assessment of safety	30
6.5.2 Slope profile	31
6.5.3 Influence of construction procedure on slope stability	31
6.5.4 Drainage	32
6.5.5 Mechanical methods of support	34
6.5.6 Improvements to soil parameters	36
6.6 Monitoring of slopes	36
6.6.1 General	36
6.6.2 Water pressure	36
6.6.3 Monitoring surface and sub-surface movements	36
6.6.4 Earth pressure measurements	37
6.6.5 Seismic observations	37
7 Embankments and general filling	37
7.1 General factors affecting the design of embankments	37
7.2 Strength and deformation characteristics of foundations and fill materials	37
7.2.1 Materials	37
7.2.2 Suitability and testing	38
7.3 Design of embankments	39
7.3.1 General	39
7.3.2 Stability	39
7.3.3 Deformations	39
7.4 Drainage	40
7.4.1 General	40
7.4.2 Pre-earthworks drainage	40
7.4.3 Temporary drainage during construction	40
7.4.4 Drainage measures to improve and maintain the stability of embankment or general filling	40
7.4.5 Permanent drainage	41
7.4.6 Construction considerations	41
7.5 Special site conditions affecting embankment or general fill design	41
7.5.1 Fill deposited in water	41
7.5.2 Embankments on soft ground	41
7.5.3 Embankments on sloping ground	42

	Page
7.5.4	Embankments over quarried ground 42
7.5.5	Embankments on existing filled ground 42
7.5.6	Embankments over mine workings and other underground voids 43
7.6	Suitability of materials for fill 43
7.6.1	Unsuitable materials 43
7.6.2	Suitable materials 43
7.6.3	Materials suitable with the adoption of special construction measures 43
8	Excavation and filling 44
8.1	Planning 44
8.1.1	General considerations 44
8.1.2	Planning of work 46
8.2	Preparation of site 46
8.2.1	General 46
8.2.2	Site clearance 46
8.2.3	Stripping topsoil 46
8.2.4	Treatment of ditches and watercourses 46
8.2.5	Underground cavities 47
8.2.6	Treatment of subsoils 47
8.3	Excavation 47
8.3.1	General 47
8.3.2	In cuttings or borrow pit areas 47
8.3.3	Below water 47
8.3.4	In rock 48
8.4	Earth-moving 48
8.5	Deposition and spreading 48
8.6	Control 49
8.6.1	Preliminary trials 49
8.6.2	Methods of monitoring deformations and stresses 49
8.6.3	Earthworks adjacent to structures 50
8.6.4	Shallow cut and fill 50
8.6.5	Earthworks adjacent to properties 50
8.7	Inclement weather during construction 50
8.7.1	Wet weather 50
8.7.2	Freezing conditions 51
8.8	Haul roads 51
8.9	Advanced earthworks 51
8.10	Stage construction 51
8.11	The disposal of spoil 51
9	Compaction 52
9.1	General 52
9.2	Test for compaction of earthworks 53
9.3	Methods of compaction 53
9.4	Soil compaction characteristics 53
9.4.1	General 53
9.4.2	Rockfill 53
9.4.3	Granular soils 53
9.4.4	Cohesive fine soils 54

	Page
9.4.5 Special fills-waste materials	54
9.5 Control of compaction in the field	54
9.6 Compaction adjacent to structures	54
9.6.1 Culverts	54
9.6.2 Abutments and retaining walls	54
9.6.3 Obstructions	54
10 The operation of construction plant for earthworks	55
10.1 Classification of plant	55
10.2 Particular factors affecting earth-moving plant	55
10.3 Preparation for excavation	55
10.4 Excavation and earth-moving plant	59
10.5 Compaction equipment	60
10.6 Specialist plant and equipment	60
10.7 Safety measures	60
11 Maintenance and protection of slopes	61
11.1 General	61
11.2 Maintenance of slopes	61
11.2.1 Factors leading to instability after construction	61
11.2.2 Inspection	61
11.2.3 Check and maintenance of drainage systems	61
11.2.4 Check and maintenance of structures	61
11.2.5 Check and maintenance of anchor systems	62
11.2.6 Control of development	62
11.3 Sources of erosion	62
11.3.1 General	62
11.3.2 Water	62
11.3.3 Frost	63
11.3.4 Wind	63
11.3.5 Other sources	63
11.4 Methods of protection	63
11.4.1 General	63
11.4.2 Vegetation	63
11.4.3 Drainage	64
11.4.4 Other forms of protection	64
11.5 Remedial works after slope failure	64
11.5.1 General	64
11.5.2 Immediate measures	65
11.5.3 Long term measures	66
Section 3. Trenches, pits and shafts	
12 Design considerations	67
12.1 Site investigation	67
12.2 Ground conditions	67
12.2.1 General	67
12.2.2 Influence of ground conditions on construction methods	67
12.2.3 Sources and control of ground water	68
12.3 The design of stable slopes and supports to excavations	69
12.3.1 General	69
12.3.2 Magnitude and distribution of lateral soil pressures	70

	Page
12.3.3	Stability of base of excavation 70
12.3.4	Movements at base of excavation 70
12.4	Practical considerations 71
12.4.1	Methods of excavation and types of support 71
12.4.2	Existing buildings, buried structures and services 71
12.4.3	Working space 72
12.4.4	Disposal of spoil 72
13	Construction procedure 72
13.1	Temporary support of excavations 72
13.1.1	General 72
13.1.2	Effect of width 73
13.1.3	Support by poling boards or steel trench sheets 74
13.1.4	Support by horizontal sheeting 75
13.1.5	Support by runners 75
13.1.6	Sheet piling of excavations 76
13.1.7	Adjustments to supports to enable the permanent work to be constructed 76
13.1.8	Maintenance of supports 77
13.1.9	Striking of supports 77
13.2	Backfilling 78
13.3	Support of excavation by diaphragm walls or cast-in-place bored piles 78
14	Trenches 79
14.1	Construction methods 79
14.2	Statutory obligations 79
14.3	Excavation procedure 80
14.3.1	Methods available 80
14.3.2	Trenches with sloping sides 80
14.3.3	Trenches with vertical sides 80
14.3.4	Bottoming of trenches 80
14.4	Mechanical excavation of trenches 81
14.5	Hand excavation of trenches 81
14.6	Methods of dewatering trenches 81
14.7	Installation of permanent work or materials in trenches 81
14.8	Backfilling and reinstatement of surface 81
14.9	Special cases 82
14.9.1	Narrow trenches 82
14.9.2	Trenches for docks, quays and similar retaining walls 82
15	Pits and shafts 82
15.1	General 82
15.2	Methods of support of excavations 82
15.3	Drainage 83
15.4	Striking of supports and backfilling 83
15.5	Alternative methods of sinking 83
15.5.1	General 83
15.5.2	Well sinking 83
15.5.3	Underpinning with segmental lining 83
15.5.4	Cast-in-place concrete lining 83

	Page
15.5.5 Steel lining installed by drilling methods	84
15.6 Use of ground treatment processes	84
16 Excavation in rock	84
16.1 General	84
16.2 Rock excavation by mechanical or hand methods	84
16.3 Rock excavation by drilling and blasting	84
16.4 Removal of loosened rock	85
Appendix A Descriptions of construction plant for earthworks	105
Appendix B List of references	108
Figure 1 — Short and long term stability of cutting and embankment slopes	86
Figure 2 — Types of rotational slide	87
Figure 3 — Successive rotational slide	87
Figure 4 — Multiple retrogressive slide	88
Figure 5 — Slab slide	88
Figure 6 — Block slide	88
Figure 7 — Wedge failure	88
Figure 8 — Toppling failure	88
Figure 9 — Fall	89
Figure 10 — Slip circle analysis (undrained shear strength s_u constant with depth)	89
Figure 11 — Slip circle analysis (undrained shear strength s_u varies laterally and with depth)	89
Figure 12 — Slip circle analysis (effective stress method)	90
Figure 13 — Effect of tension cracks on slope stability	90
Figure 14 — Non-circular slide on weak soil layer	90
Figure 15 — Planar slide on infinite slope — stability analysis	91
Figure 16 — Steep soil slope retained by ground anchors	91
Figure 17 — Unstable rock slope stabilized by rock bolts	91
Figure 18 — Slab slide stabilized by dowel bolts	92
Figure 19 — Reinforced earth	92
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	92
Figure 21 — Envelope of strut loads on braced excavations	93
Figure 22 — Water-bearing permeable soils: seepage and uplift	93
Figure 23 — Typical single or centre waling poling frame	94
Figure 24 — Close poling with tucking frames	95
Figure 25 — Support of deep excavation by horizontal sheeting and soldier piles	96
Figure 26 — Trench excavation using steel trench sheets as runners	97
Figure 27 — Typical medium-depth trench with steel sheet piling in bad ground	99
Figure 28 — Wide and deep trench using steel sheet piling	100
Figure 29 — Blockings, chocks or chogs	101
Figure 30 — Trenches with sloping sides	101
Figure 31 — Method of lowering long steel pipes by use of temporary strutting	102
Figure 32 — Trench timbering for quay walls, etc.	102

	Page
Figure 33 — Method of excavating isolated pits and shafts	103
Figure 34 — Rock excavation by drilling and blasting: general excavation	104
Figure 35 — Rock excavation by drilling and blasting: pits and shafts	104
Table 1 — British soil classification system for engineering purposes and field identification	13
Table 2 — Field characteristics of soils and other materials used in earthworks	15
Table 3 — Design of slopes in rock cuttings and embankments	20
Table 4 — Typical compaction characteristics for natural soils, rocks and artificial materials used in earthwork construction	56
Publications referred to	Inside back cover

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 in 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.

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 5930¹⁾, 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 use 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 local

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	G-M	GWM GPM	5-15	Well/poorly graded, silty (clayey) GRAVEL	GC/GF	
		Clayey GRAVEL	G-F G-C	GWC GPC				
		Very silty GRAVEL	GF GM	GM	15-35	Very silty GRAVEL Sub-divide like GC Very clayey GRAVEL (clay of low/ intermediate/high/very high plasticity)	GF GF	
		Very clayey GRAVEL	GC	GCL GCI GCH GCV				
	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	S-F S-M	SWM SPM	5-15	Well/poorly graded silty (clayey) SAND	SC/ SF	
		Clayey SAND	S-C	SWC SPC				
		Very silty SAND	SF SM	SM				
Very clayey SAND	SC	SCL SCI SCH SCV						
^a Gravel and sand may be qualified as Sandy GRAVEL and Gravelly SAND where appropriate.								

^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						Field identification		
		Description	Group symbol	Sub-group symbol	Liquid limit %	Sub-group name	Casagrande group symbol			
FINE SOILS more than 35 % finer than 0.06 mm	Gravelly or sandy SILTS and CLAYS 35 %–65 % fines	Gravelly SILT	MG	MG	35 35–50 50–70 70–90	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 CIG CHG CVG		Gravelly CLAY of low plasticity Gravelly CLAY of intermediate plasticity Gravelly CLAY of high plasticity 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	35 35–50 50–70 70–90	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 CI CH CV	CLAY of low plasticity CLAY of intermediate plasticity CLAY of high plasticity CLAY of very high plasticity		CL CI CH —				
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	
Coarse soils and other materials	Boulders and cobbles	Boulder gravels	—	—	Good	None to very slight	Almost none	Good to excellent	— mg/m ³	— mg/m ³	— %
	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	
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	mg/m ³ 1.70–1.90	mg/m ³ 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

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

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 cleavage 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:

$$s = c' + (\sigma_n - u) \tan \phi' \quad (2)$$

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 heterogeneity 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 retrogressive 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.