

BS 6031:2009

Incorporating Corrigendum No. 1



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Code of practice for earthworks

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Summary of pages

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Foreword

Publishing information

This British Standard is published by BSI and came into effect on 31 December 2009. It was prepared by Technical Committee B/526, *Geotechnics*. A list of organizations represented on this committee can be obtained on request to its secretary.

Information about this document

The start and finish of text introduced or altered by Corrigendum No. 1 is indicated in the text by tags C1 C1.

Supersession

This British Standard supersedes BS 6031:1981, which is obsolescent.

NOTE It was considered important to make the information on timber support and other largely historic advice available through the previous edition, which is still available from BSI.

Relationship with other publications

The standard has been completely re-written to bring it into line with both UK earthworks practice and the framework that is created by the Eurocodes. The aim was to reduce the size of the document and wherever possible include cross references to other existing documents.

This revision of BS 6031 reflects the widespread UK practice of using the Specification for Highway Works (SHW) 600 series [1] for the construction of earthworks. Within this standard, the SHW has been set as the default approach for earthworks specification that applies unless the designer details an alternative form of specification/earthworks management system.

Cross references are included within this standard to various other documents, to link into the existing information sources available. However, it remains the responsibility of the designer of the earthworks for a project to assess whether a reference is relevant to the particular project.

Use of this document

As a code of practice, this standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Any user claiming compliance with this standard is expected to be able to justify any course of action that deviates from its recommendations.

It has been assumed in the preparation of this standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people, for whose use it has been produced.

Presentational conventions

The provisions in this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

Section 1: General

Introduction

The structure for this document reflects the actual processes that might be followed on a typical project to deliver the earthworks. In practice earthworks design is an iterative process where design decisions are often taken by various parties (employer, consultant, main contractor, sub-contractors and construction validation team). To reflect this situation BS 6031:2009 includes some subjects in more than one clause.

This revision of BS 6031 reflects the widespread UK practice of using the Specification for Highway Works (SHW) 600 series [1] for the construction of earthworks. Within this standard, the SHW has been set as the default approach for earthworks specification that applies unless the designer details an alternative form of specification/earthworks management system. Guidance on the use of the SHW can be found in *Notes for Guidance to the Specification for Highway Works* [2].

Earthworks are commonly associated with transport infrastructure, but there are many other important applications:

- platforms for industrial, commercial and residential buildings;
- water engineering, flood defence and coastal protection works;
- other civil engineering projects.

BS 6031 is intended to be an all-encompassing code of practice; the document has been developed to enable it to cover all earthworks projects, with the exception of dams. In this regard it is relevant to note the following.

- a) Embankment dams are constructed either to retain water or for waste impoundments and, while some aspects of the design, construction and maintenance of such embankments are similar to those pertaining to infrastructure embankments, those features which relate specifically to their function as dams are not within the scope of this standard. Note that since 1930 reservoir safety in Great Britain has been regulated by Act of Parliament. *A guide to the Reservoirs Act 1975*¹⁾ [3] describes the application of current legislation and *An engineering guide to the safety of embankment dams in the United Kingdom* [4] provides some relevant information on earthworks.
- b) Substantial earthworks can take place for the purpose of providing a suitable landform for building development. Typically this can involve:
 - 1) backfilling old pits and quarries with engineered fill;
 - 2) cut and fill operations on natural slopes to provide terraces for building.

In the former situation the major hazard to be guarded against is long-term settlement of the fill occurring subsequent to building development; in the latter situation slope instability can also be a significant hazard. While most of the technical background

¹⁾ Likely to be revised in 2010.

of highway earthworks (as captured within the SHW [1]) is also relevant to this type of application, two significant differences have to be recognized where structures are built on fill:

- i) settlement criteria can be much stricter than those normally acceptable for general earthworks; the designer has to consider whether the SHW [1] criteria are sufficient for the project; and
- ii) the engineering environment in which the earthworks are carried out does influence the approach that is applicable to earthworks; this is particularly relevant on comparatively small scale projects where the designer might need to modify the approach to earthworks.

This standard has been drafted to include sufficient flexibility to allow for these scenarios.

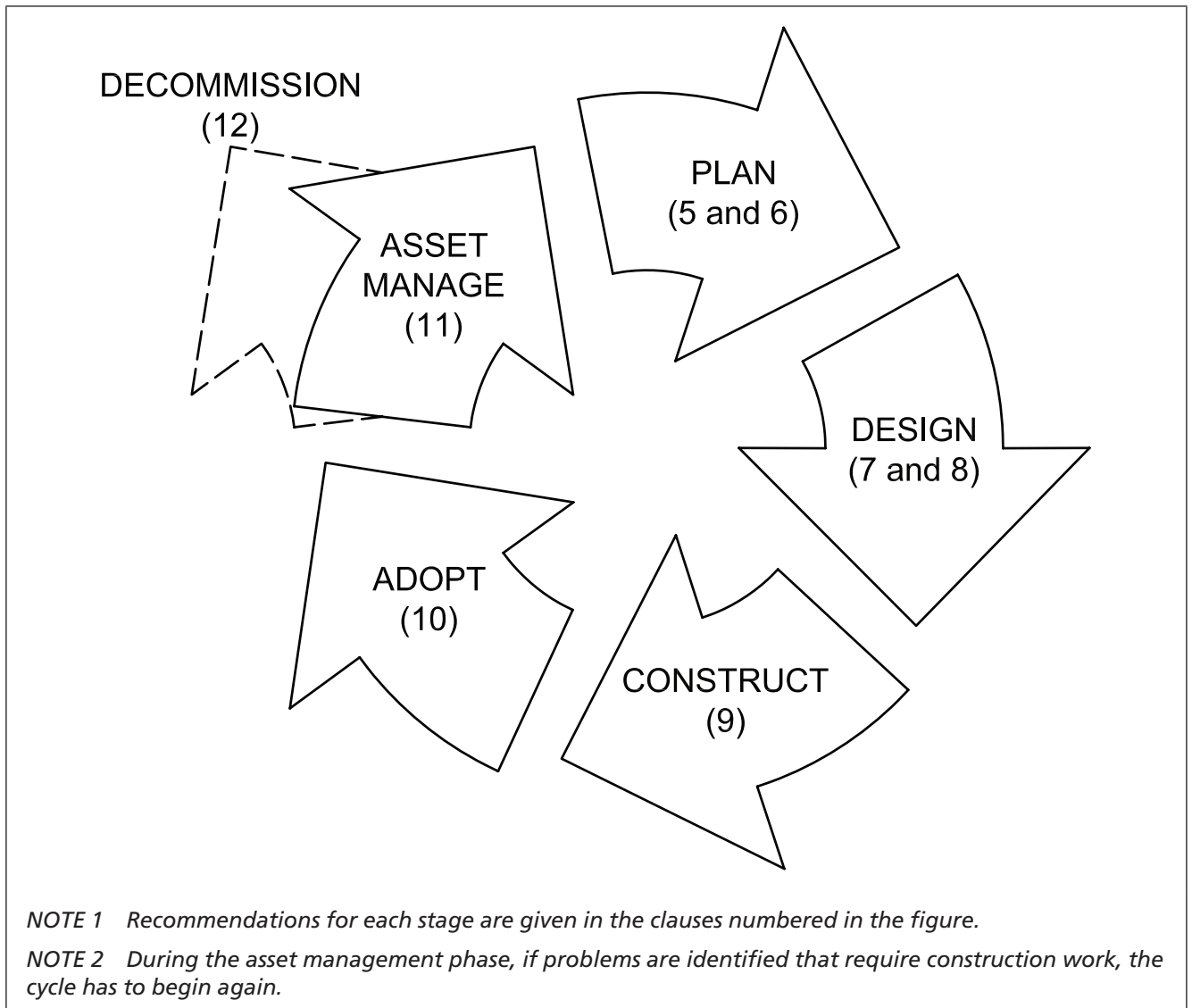
This standard is aligned with Eurocode BS EN 1997-1:2004. Advice on using BS EN 1997-1:2004 is provided in Thomas Telford *Designers' Guide to EN 1997-1 Eurocode 7: Geotechnical Design – General Rules* [5], local government guide *A designer's simple guide to BS EN 1997* [6], and CIRIA C641, *Eurocode 7: implications for UK practice* [7]. These publications help highlight differences between BS EN 1997-1:2004 and previous traditional practice.

The foreword to BS EN 1997-1:2004 states that "BS 6031 is to be withdrawn", which is an error; it has been agreed since the publication of BS EN 1997-1:2004 that BS 6031 will remain as part of the system of earthworks standards in UK.

The style adopted within BS 6031:2009 is to cross reference BS EN 1997-1:2004 (not repeat it), summarize the aspects of BS EN 1997-1:2004 that form the overall framework for undertaking an earthworks project, include an interpretation of certain key points that are relevant/important to earthworks (e.g. selection of partial factors to be used at design stage) and add additional information that is relevant to earthworks (i.e. add some commentary to the "dry rules" set out in BS EN 1997-1:2004). The overall aim is for BS 6031 to be non-conflicting complementary information (NCCI) to BS EN 1997-1:2004.

This edition of BS 6031 is set out in clauses to reflect the overall earthworks process: where earthworks are planned, designed, constructed, adopted/approved following construction, and then the earthworks moves into an asset management process. This cycle is only broken when the earthworks reach the end of their useful life and are decommissioned. This cycle is illustrated in Figure 1. This standard recognizes that construction of new earthworks and the remediation or repair of existing earthworks are activities that have similarities and significant differences. Wherever possible, clauses cover both construction and remediation activities, which need to be taken into consideration as appropriate.

Figure 1 Flow diagram of lifecycle of an earthworks project



1 Scope

This standard gives recommendations and guidance for unreinforced earthworks forming part of general civil engineering construction, with the exception of dams. This standard also gives recommendations and guidance for temporary excavations such as trenches and pits.

NOTE Reinforced earthworks are covered in BS 8006-1 and BS 8006-2.

This document applies to earthworks classified as Geotechnical Category 1, 2 and 3 structures as defined in BS EN 1997-1:2004.

2 Normative references

Standards publications

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

BS 812-109, *Testing aggregates – Methods for determination of moisture content*

BS 1377 (all parts), *Methods of test for soils for civil engineering purposes*²⁾

BS 1924-1, *Stabilized materials for civil engineering purposes – Part 1: General requirements, sampling, sample preparation and tests on materials before stabilization*

BS 1924-2, *Stabilized materials for civil engineering purposes – Part 2: Methods of test for cement-stabilized and lime-stabilized materials*

BS 5607, *Code of practice for the safe use of explosives in the construction industry*

BS 5930:1999+A1:2007, *Code of practice for site investigations*³⁾

BS 6164, *Code of practice for safety in tunnelling in the construction industry*

BS EN 474 (all parts), *Earth-moving machinery – Safety*

BS EN 500-4, *Mobile road construction machinery – Safety – Part 4: Specific requirements for compaction machines*

BS EN 791, *Drill rigs – Safety*

BS EN 1990:2002+A1:2005, *Eurocode 0 – Basis of structural design*

BS EN 1997-1:2004, *Eurocode 7: Geotechnical design – Part 1: General rules*⁴⁾

BS EN 1997-2:2007, *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing*⁵⁾

BS EN 1998-5, *Eurocode 8: Design of structures for earthquake resistance – Part 5: Foundations, retaining structures and geotechnical aspects*

BS EN 12111, *Tunnelling machines – Road headers, continuous miners and impact rippers – Safety requirements*

BS EN 13331-1, *Trench lining systems – Part 1: Product specifications*

BS EN ISO 14688-1:2002, *Geotechnical investigation and testing – Identification and classification of soils – Part 1: Identification & description*

BS EN ISO 14688-2:2004, *Geotechnical investigation and testing – Identification and classification of soils – Part 2: Classification principles*

BS EN ISO 14689-1, *Geotechnical investigation and testing – Identification and classification of rock – Part 1: Identification and description*

NA to BS EN 1990:2002+A1:2005, *UK National Annex for Eurocode – Basis of structural design*

²⁾ Under revision to act as NCCI to BS EN 1997-2:2007.

³⁾ Under revision with the intention that BS 5930 remains as the “first stop” for site investigations but also to act as NCCI to BS EN 1997-2:2007.

⁴⁾ Any general reference to BS EN 1997-1:2004 is also to be taken as a reference to NA to BS EN 1997-1:2004.

⁵⁾ Any general reference to BS EN 1997-2:2007 is also to be taken as a reference to NA to BS EN 1997-2:2007.

NA to BS EN 1997-1:2004, *UK National Annex to Eurocode 7 – Geotechnical design – Part 1: General rules*

NA to BS EN 1997-2:2007, *UK National Annex to Eurocode 7 – Geotechnical design – Part 2: Ground investigation and testing*

Other normative references

- [1] *Manual of Contract Documents for Highway Works, Volume 1 – Specification for Highway Works, Series 600 – Earthworks*. Highways Agency. (www.standardsforhighways.co.uk)

3 Terms and definitions

3.1 General

For the purposes of this British Standard, the definitions given in Table 1a), Table 1b), Table 1c), 3.2 and 3.3 apply, together with those given in BS 1377-1, BS EN 1997-1:2004 and BS EN 1997-2:2007.

Within the Eurocode family of documents, BS EN ISO 14688-1:2002 and BS EN ISO 14688-2:2004 establish the basic principles for the identification and classification of soils for engineering purposes. Identification and description of a soil is initially undertaken on the basis of visual techniques in accordance with BS EN ISO 14688-1:2002. BS EN 1997-2:2007, Table 2.1 states that the main soil groups are according to BS EN ISO 14688-1:2002 which defines soils as being:

- Made Ground;
- organic soil;
- volcanic soil;
- fine soils [see Table 1a)];
- coarse soil [see Table 1a)]; or
- very coarse soil [see Table 1a)].

BS EN ISO 14688-1:2002, Figure 1 is a “flow chart for the identification and description of soils” for general (preliminary) characterization of soils after the very coarse fraction (cobbles and boulders) has been screened out. A general test is used to classify the soil type on the basis of “does the soil stick together when wet?”

- Yes (i.e. particles stick together) then = fine soil.
- No (i.e. particles don’t stick together) then = coarse soil.

Basic soils are defined as soils with uniform grading that consist of particles of only one size range, the definition of particle names and particle size ranges are set out at Table 1 of BS EN ISO 14688-1:2002, which are illustrated in Table 1a).

In reality almost all uniformly graded soils have some proportion of other particle sizes. The Eurocode system does not set an upper bound on the proportion of secondary fraction required before a soil is defined as a composite soil; this is likely to depend on the soil type. However, in earthworks less than 10% of secondary fraction might be considered as a general guide below which the soil could be considered as a basic soil (rather than a composite soil). The most likely exception would be sand where a fines content of less than 10% can significantly influence the engineering behaviour in earthworks and in that case would be considered a composite soil.

BS EN ISO 14688-2:2004 provides the principles for a more detailed classification of a soil where laboratory test data are available to allow grading, plasticity and organic content to be taken into account more fully. With regard to grading, BS EN ISO 14688-2:2004, Table 2 sets criteria for the designation of coarse fractions based on the shape of the grading curve (both uniformity coefficient and coefficient of curvature C_c). In earthworks particular consideration should be given to the uniformity coefficient (C_u , previously designated U_c , defined as particle size at which 60% of the material is finer/particle size at which 10% is finer). Soils that have a uniformity coefficient of less than 6 (and $C_c < 1$) are distinctly uniform in grading and described as "evenly-graded". The phrase "uniformly graded" is a general descriptive term in BS EN ISO 14688-2:2004 (unlike "evenly-graded"), but, the SHW [1] system sets a uniformity coefficient upper limit of 10 for classification of uniformly graded granular material which remains a valid consideration for earthworks.

The grouping of soils by particle size for testing purposes [see Table 1b)] is significantly different to that used for soil particle size classification [see Table 1a)]. BS 1377-1 divides soil groups based on the soil sizes that are suitable for different forms of test.

Most soils consist of a principal and secondary fractions and are defined as "composite soil", the identification of which is described at BS EN ISO 14688-1:2002, 4.3. A composite fine soil is one where the fines fraction determines the engineering properties of the soil (this can include relatively coarse soil where the fine matrix is sufficient to result in the soil being matrix dominated); where the fines fraction is insufficient to determine the engineering properties the soil is a composite coarse soil. To fully classify the soil consideration is given to other factors including plasticity of the fines fraction (test carried out on $< 425 \mu\text{m}$ material). However, the principles for classification of soils set out at BS EN ISO 14688-2:2004 is useful in assessing likely earthworks behaviour.

In the field of earthworks the assessment of whether the fines fraction (i.e. passing a $63 \mu\text{m}$ sieve) is sufficient to determine the engineering properties of the soil will differ between two major cases:

- where a soil is considered as an engineered fill, where all fills with $> 15\%$ fines are classified as cohesive; and
- soil assessed for geotechnical design (e.g. slope stability, settlement or bearing capacity) where generally fine (cohesive) soils are likely to include $> 35\%$ fines.

Therefore, in accordance with industry practice, the soils described in this standard as "coarse/granular", "intermediate" or "fine/cohesive" contain different percentages of fines depending on the context in which the descriptions are used [see Table 1c)]. Table 1c) also provides a simplified summary of the BS EN ISO 14688-1:2002 approach to illustrate that both approaches fit within that framework.

In composite soils the assessment of the "dominant soil fraction" which will determine the engineering properties of the soil requires consideration of a variety of soil characteristics (particularly plasticity, grading and soil fabric). Within the intermediate zones illustrated in Table 1c) experience shows that seemingly similar soils can behave differently with relatively small differences in these soil characteristics. These variations are very notable in glacial till as described in CIRIA C504 [8].

Note that Table 1a) and Table 1b) are presented based on particle sizes, while Table 1c) shows percentage passing the $63 \mu\text{m}$ sieve.

Table 1a) Soil classification (after BS EN ISO 14688-1:2002)

| Particle Sizes mm | 0.002 | 0.0063 | 0.02 | 0.063 | 0.2 | 0.63 | 2.0 | 6.3 | 20 | 37.5 | 63 | 200 | 630 | >630 |
|-------------------|-----------|--------|--------|-------------|------|--------|--------|------------------|--------|---------|----|---------------|-----|------|
| Soil fraction | Fine soil | | | Coarse soil | | | | Very coarse soil | | | | | | |
| Sub-fraction | Clay | Silt | | Sand | | Gravel | | Cobble | | Boulder | | Large Boulder | | |
| | | Fine | Medium | Coarse | Fine | Medium | Coarse | Fine | Medium | Coarse | | | | |

Table 1b) Grouping of soils for testing purposes (after BS 1377-1)

| | | | | |
|------------|---------------------------------|-----------------------------------|------------------------------------|---|
| Soil Group | Fine-grained soil ^{A)} | Medium-grained soil ^{B)} | Coarse-grained soils ^{C)} | Soil too coarse to be covered by BS 1377 ^{D)} (Zone X Material) |
|------------|---------------------------------|-----------------------------------|------------------------------------|---|

A) Soils containing not more than 10% retained on a 2 mm test sieve.

B) Soils containing more than 10% retained on a 2 mm test sieve but not more than 10% retained on a 20 mm test sieve.

C) Soils containing more than 10% retained on a 20 mm test sieve but not more than 10% retained on a 37.5 mm test sieve.

D) Soils with more than 10% of material retained on a 37.5 mm test sieve are not covered by the laboratory tests detailed in BS 1377-1: except for particle size analysis, and the moisture content and plasticity tests of the finer fraction if present. However, for earthworks control purposes the designer may still choose to utilize the test methods on a modified sample in order to generate test data to consider potential performance (see 7.6.4).

Table 1c) Comparison of soil definitions in different earthworks circumstances

| UK standard approach to earthworks material classification by grading (after SHW ^{A),B)} | % passing a 63 µm sieve | | | | | | | | | | | | | | | | | | | | |
|--|-------------------------|---|--|----|----|----|-----------------------|----|----|----|-----------|----|----|----|----|----|----|----|----|----|-----|
| | 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 | 45 | 50 | 55 | 60 | 65 | 70 | 75 | 80 | 85 | 90 | 95 | 100 |
| UK traditional approach to classification for geotechnical design (after BS 5930:1999+A1 ^{D)}) | granular fill | | intermediate fill ^{C)} , classified as cohesive fill | | | | cohesive fill | | | | | | | | | | | | | | |
| BS EN 1997-1:2004 geotechnical design approach, (after BS EN ISO 14688-1:2002 ^{E)}) | coarse/granular | | intermediate zone ^{C)} | | | | cohesive/fine grained | | | | | | | | | | | | | | |
| | coarse soil | | composite coarse soil | | | | composite fine soil | | | | fine soil | | | | | | | | | | |
| | | | BS EN 1997-1:2004 approach does not set any fixed boundary but generally > 10% of the secondary fraction is likely to be needed in most soil types to constitute a composite soil. | | | | | | | | | | | | | | | | | | |

A) SHW [1] sets the granular/intermediate divide at 15% in recognition of pore water pressure in dynamic action of compaction.

B) The terms "granular" and "cohesive" are included here with regard to behaviour, the soil description terms in accordance with BS 5930:1999+A1 are "coarse" and "fine".

C) The designer has to use judgement of how a soil will behave within the intermediate zone, which is not considered in BS 5930:1999+A1.

D) Most fills in the UK that are in the intermediate range are classified as class 2C. Alternatively, the designer can create a new site-specific class e.g. "class 2F, clayey sand".

E) The BS EN 1997-2:2007 approach for identification and description of soils is set out within BS EN ISO 14688-1:2002, 4.3, by this system many soils are classified as composite soils and the distinction between soil terms can be summarized as follows:

- "composite fine soil" is a soil where the fines content is sufficient to determine the engineering properties;
- "composite coarse soil" is a soil where the fines content is not sufficient to determine the engineering properties (BS EN ISO 14688-1:2002 should be referred to for the full determination procedures).

3.2 Terms used in Section 2

3.2.1 berm

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

3.2.2 colluvial deposits

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

3.2.3 earthwork

work of excavating, or the raising or sloping of ground

NOTE 1 From BS 6100-1:2004.

NOTE 2 The work can be a stand-alone activity or part of a larger project.

3.2.4 earthworks

- 1) structures formed by the excavating, raising or sloping of ground, e.g. embankments, cuttings or remediated natural slopes
- 2) civil engineering process that includes extraction, loading, transport, transformation/improvement, placement and compaction of natural materials (soils, rocks), and/or secondary or recycled materials, in order to obtain stable and durable cuttings, embankments or engineered fills

3.2.5 landslip landslide

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

3.2.6 piezometer

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

3.2.7 scree

accumulated rock debris at the foot of a cliff

NOTE See colluvial deposits.

3.2.8 slope face angle

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°)

3.2.9 solifluction

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

3.2.10 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

3.2.11 subsidence

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

3.3 Terms used in Section 3

3.3.1 trench

excavation whose length greatly exceeds its width

3.3.2 shallow trench

trench up to 1.5 m in depth

3.3.3 medium trench

trench between 1.5 m and 6.0 m in depth

3.3.4 deep trench

trench exceeding 6.0 m in depth

3.3.5 narrow trenches

class of excavation too narrow to allow the entry of personnel

3.3.6 pit

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, including trial pits excavated for site investigation purposes

3.3.7 shallow pit

pit up to 1.5 m in depth

3.3.8 medium pit

pit between 1.5 m and 6.0 m in depth

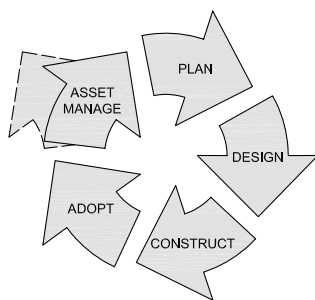
3.3.9 deep pit

pit exceeding 6.0 m in depth

3.3.10 shaft

excavation, which may be either vertical or inclined, constructed to give access to underground works

NOTE Shallow, medium and deep shafts are defined in the same way as shallow, medium and deep pits.



4 The control of risk

4.1 Competence

For earthworks to be successfully designed, constructed and maintained, it is important that the personnel undertaking each task should be competent. When possible this should be managed by the implementation of a system to demonstrate the competence of the staff involved.

In some industries there are specific requirements for competence management systems which should be adhered to, e.g. *Railway safety principles and guidance* [9].

4.2 Risk management

4.2.1 Projects

The management of risk should be a key aspect of projects involving earthworks as in all construction activity. The areas of risk to be managed should include:

- programme, quality and financial risks to ensure the successful delivery of the project;
- health and safety, along with environmental risks to satisfy statutory requirements.

NOTE 1 All projects involving earthworks come under the requirements of the Construction (Design and Management) Regulations 2007 [10] (CDM).

All those involved in design (and construction) activities associated with earthworks should consider the requirements of the project and seek to follow the "spirit of CDM" to ensure the approach taken is appropriate for the project.

NOTE 2 BS EN 1997 (both parts) [C1] was not drafted to follow the UK's CDM regulations, however it is consistent with the spirit of CDM. [C1]

NOTE 3 The creation and maintenance of a project risk register (PRR) is seen as good practice and is a requirement under CDM [10].

All risks identified by those involved in the design of earthworks should be fed into the PRR. It is important to realise that various parties have an input to the design of earthworks, and therefore as designers they should engage with development of the PRR.

Where a project includes geotechnical design, a geotechnical risk register (GRR) should be developed to support and enhance the PRR. The scale of the GRR depends on the complexity of the project. This approach should help the earthworks team manage out or minimize some of the geotechnical risks; all residual risks should then be fed into the PRR. On projects with multiple design teams, risks should be highlighted and their management co-ordinated in the PRR.

A GRR will tend to divide into two distinct sections:

- a) the "risks affecting investigation", which relate to the procurement and implementation of the site investigation (e.g. access, buried services, contamination, topography); and
- b) the "risk affecting works" which relate to prevailing geotechnical conditions that might affect the design and construction of the project.

For further details see Section 2, *Design and management of earthworks*.

4.2.2 Asset management

Maintenance of existing earthworks should follow the management approach of reducing risk levels to "as low as reasonably practicable" (ALARP), giving consideration to whether spending on remedial works will result in a cost benefit over a realistic maintenance period. Remedial works become a self-contained project requiring design and construction with concomitant geotechnical risks that should be approached as described in 4.2.1.

4.3 Geotechnical certification

The concept of staged approval of a scheme via a certification system is of benefit to the control of risk, and should be adopted on complex projects involving earthworks (BS EN 1997-1:2004, Geotechnical Category 2 and Geotechnical Category 3). The geotechnical certification scheme should be flexible enough to enable the level of detail required to be appropriate for the complexity and scale of the

project. When used, geotechnical certification can be a very effective system to record the identification and management of risks by the geotechnical design team.

BS EN 1997-1:2004 does not require certification but BS EN 1997-1:2004, Section 4 does require the designer to specify requirements for inspection and testing during construction, which for earthworks might be most easily achieved by a form of certification.

NOTE One system is that operated by the Highways Agency (see HD 22/08 [11]), which embraces the requirements of BS EN 1997-1:2004, Section 4.

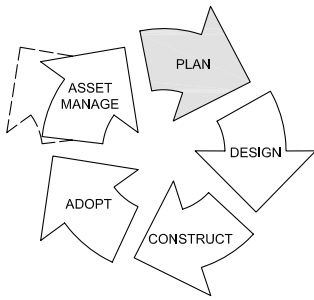
4.4 Geotechnical feedback report

Construction Design and Management Regulations [10] require the drafting of a health and safety file that contains full records of the works constructed. As-built records have to include details of the earthworks undertaken for a project to assist with future maintenance, design of additional works or decommissioning of the works.

For earthworks projects, the as-built records can be significantly enhanced by the inclusion of a geotechnical feedback report (GFR) that includes details of the earthworks undertaken. All parties involved have to be aware of the need to capture records of the works undertaken from an early stage of the project. This approach can bring significant benefits in the management of risk. When prepared, this document becomes one element of the health and safety file. The relationship between the GFR and other documentation is illustrated in 6.5.

BS EN 1997-1:2004 does not specifically include the concept of the GFR, however BS EN 1997-1:2004, Section 4 covers many of the topics which need to be addressed within this style of report. The GFR is considered an appropriate way of recording the data from earthworks projects to form a complementary record to the as-built drawings and meet the requirements of BS EN 1997 (both parts).

Section 2: Design and management of earthworks



5 Planning of earthworks

5.1 Introduction

Clause 5 is provided as a brief summary of some of the issues that should be considered when planning an earthworks project; it is not intended to be a complete list and is provided only as guidance. The person applying the code should remain cognisant of the current and applicable statutory requirements that apply to all civil engineering projects.

5.2 Sustainable development

Earthworks should be designed with sustainability in mind. For the purpose of this standard sustainable development is taken to mean: “an enduring, balanced approach to economic activity, environmental responsibility and social progress” (BS 8900:2006).

COMMENTARY ON 5.2

In future projects, evaluation of options will increasingly be required to consider construction cost and whole life cost, but also include an evaluation of the “carbon footprint” of various scheme options. The designer will be expected to calculate the “embodied carbon” present in various designs (this requires consideration of all aspects of an earthwork solution, e.g. import/export of fill materials; plant operations; use of carbon-based geosynthetic materials; soil modification or stabilization, etc.).

This process is intended to prioritize “carbon critical design”. To encourage designers to follow this approach, both the Environment Agency (EA) and Highways Agency websites provide “carbon calculators” (based on research undertaken by Bath University) that are suitable for undertaking a basic assessment of earthwork scheme options. It is likely that these calculators will be progressively enhanced as experience is developed in this field.

5.3 Scheme conception

5.3.1 Design strategy

The earthworks team should aim to provide an earthworks design that is feasible, functional, constructible and suitable for the proposed end use. Consideration should be given to land requirements, including all temporary works. The design should be developed to minimize environmental impact during the construction phase, in use and for future maintenance operations.

5.3.2 Consultations

5.3.2.1 General

The consultation process is a necessary part of any earthworks design and should be implemented at an early point following the project’s conception.

The earthworks team should identify their environmental, legal and planning obligations at the earliest opportunity to enable them to conceive and develop the project in the most effective way.

The nature of the consultation(s) will depend on the size of the project, and should involve dialogue with both regulatory and community bodies.

5.3.2.2 Regulators

It is important to develop a constructive dialogue with the regulators to ensure that they are aware of what is happening on the project and the reasons behind any design decisions that are made.

COMMENTARY ON 5.3.2.2

Legislation is in place which is enforced by the environmental regulators to protect both the natural environment and residents surrounding sites through the planning process and implementation of project specific environmental management schemes.

Regulators have a diverse range of responsibilities and enforcement powers. Their responsibilities are as follows:

| | |
|--|--|
| <i>Local Authority</i> | <i>Noise, air quality, traffic, the planning process and contaminated land</i> |
| <i>Environmental regulators</i> | <i>Discharges to land and controlled waters, waste, water abstraction control, nature conservation</i> |
| <i>Nature conservation organizations</i> | <i>Designated ecological, geological and geomorphological sites and protected species</i> |
| <i>Heritage bodies</i> | <i>Designated archaeological and heritage sites</i> |

5.3.2.3 Community relations

Developing effective communication and consultation with the local community is important to minimize the likelihood of nuisance (e.g. noise and vibration, dust, waste etc.) and should be instigated at the earliest opportunity; if members of the community are engaged and kept informed throughout a project they are less likely to complain.

Public consultation is particularly important on larger projects where prolonged periods of disturbance and/or significant changes to the environment or landscape are likely to occur.

Typical liaisons should include:

- community representatives e.g. town or parish councillors;
- occupants of sensitive buildings e.g. schools, nursing homes, hospitals, etc.;
- local residents and/or resident groups;
- national and local interest groups, e.g. environmental groups, trade associations, etc.

5.4 Environmental considerations

Environmental considerations should be assessed by the designer, who should judge where the balance of importance lies for a given scheme.

NOTE 1 Which aspect takes priority will change from scheme to scheme and with time as legislation and other factors change.

At many sites there are environmental restrictions on the works that will be permitted (SSSI, protected species, etc); at all sites the earthworks

should be planned to consider environmental considerations and thus reduce the environmental impact of the scheme.

NOTE 2 There is environmental and planning legislation to be complied with during the development of an earthworks design. This includes, but is not limited to Environmental Assessments, Environmental Statements and Habitat Surveys [12].

Affected parties such as Natural England should be consulted to ascertain environmental requirements during and post construction, including translocation of endangered species and acceptable environmental mitigation works.

The earthworks design team should be aware of the overall environmental objective to minimize environmental impact, which is often summarized by the slogan “rethink, reduce, reuse, recycle”, which reflects the priority of each option. These options in the case of earthworks may be applied in various ways including the following:

- rethink – e.g. consider whether any particular course of action is actually necessary;
- reduce – e.g. reduce the overall environmental impact, such as reducing the volume of material excavated or the requirement for aggregate;
- reuse – e.g. reuse all earthworks materials generated on a project whenever possible (within earthworks or landscaping), or use of stabilisation techniques to render materials suitable for reuse;
- recycle – e.g. recycling of materials between sites such as use of secondary aggregates.

Whenever management of waste materials forms part of an earthwork scheme, input should be obtained from an environmental specialist to address aspects such as waste classification and assessment for hazardous substances; this will necessitate appropriate testing as part of the site investigation (SI). In addition to the requirements of BS EN 1997-2:2007, the SI should consider the following issues that are of particular significance to earthworks.

- Existing ground and re-use of materials:
 - suitability as construction materials (classification), including “relationship testing” (see 6.1.3);
 - aggressivity of ground (see BRE SD1 [13]);
 - contamination testing sufficient to develop the conceptual model of the site and inform appropriate risk assessment.
- Modification of marginal materials for use as engineering fills.
- Enhancement of soils as growing media.

NOTE 3 This standard does not cover the treatment of contaminated materials.

Any materials that might require off-site disposal should be tested to ascertain their classification under relevant requirements, including:

- waste acceptance criteria;
- European waste classification; and
- suitability for re-use off-site (subject to acceptance by EA).

NOTE 4 The above list is a general guide and any assessment has to satisfy the requirements of current UK environmental and waste management legislation (and likely developments in the short term).

The project team should be aware of any changes to relevant legislation, and that industry best practice will change to accommodate regulatory revision. Consultation with relevant regulatory authorities is recommended at the earliest opportunity.

The tax imposed on disposal of waste and the importing of primary aggregate can change; up-to-date information should be obtained from HM Revenue and Customs.

Designers should be aware of the potentially significant commercial impact of designs requiring off-site disposal of waste. Treatment of waste can significantly increase the proportion of material classified as inert, thus reducing the potential cost of disposal. The following treatment methods may be cost-effective depending on the condition of the materials awaiting disposal.

- physical sorting/screening to separate inert and contaminated materials;
- pH adjustment to immobilize heavy metals;
- stabilizing with hydraulic binders;
- bioremediation;
- sparging or similar techniques to reduce volatiles;
- soil washing.

During the design of any investigation, testing should be specified to assist in the selection of a treatment process suitable for dealing with arisings from the site.

NOTE 5 Legislation [14] requires that all waste is pre-treated prior to disposal.

Environmental impacts and impact mitigation that should be considered include the following:

- habitat loss due to earthworks and opportunities for the creation/improvement of habitat provision;
- sustainable drainage systems (SUDS; a drainage option to enhance biodiversity, amongst other benefits, as described in 7.5.5);
- loose tipping (reduced compaction) of sub-soil, which should be practiced, where possible, to promote re-vegetation;
- geodiversity can be enhanced by retention of exposures in cutting, pre-existing spoils of mineralogical/palaeontological interest as an educational resource (some cuttings are designated as Regionally Important Geological and Geomorphological Sites [RIGS]);
- plan from an early stage to use local resources (e.g. use of locally available recycled and secondary aggregates) and to avoid unnecessary disposal of materials off site (e.g. inclusion of environmental bunding to avoid disposal of surplus material);

NOTE 6 Planning in this way greatly increases the potential for these options to be successful at detailed design stage.

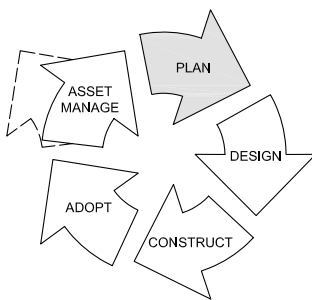
- the physical footprint of an earthwork can be modified by design to mitigate adverse impacts of land take; a narrow footprint achieved by using reinforced soil will reduce land take in areas where level ground is required for development; a wide footprint, with gentle batter slopes, may permit agricultural use of the slopes; and

- potential impact of the earthworks form on future air and noise quality in the immediate area.

NOTE 7 There are opportunities to apply innovation to achieve environmental sustainability objectives, e.g. use of tyre bales (see TRL report PPR080 [15]).

5.5 Environmental impact assessment (EIA)

The process of preparing the EIA for a scheme should be led by suitably experienced environmental specialists who are aware of both the nature of the proposed engineering works and the current legislation covering this area of work. Earthworks professionals should have sufficient understanding of the process to provide input to the EIA and use the completed assessment (see [12]).



6 Site conditions and investigations

6.1 Site investigation for earthworks

6.1.1 Scope

The site investigation [comprising desk study, geomorphological mapping, topographic survey and physical ground investigation (GI) as appropriate] should be planned and implemented to ensure that the site conditions are adequately understood. In particular it is essential that the GI provides adequate and sufficient information for design and construction, and for investigation of existing earthworks and slopes (BS EN 1997-1:2004, 2.1.8 and 2.1.9 have to be satisfied in this regard).

NOTE The degree of the investigation will depend on the complexity of the project. For small-scaled earthworks, such as simple re-grading or reshaping of the ground profile, a simple trial-pitting investigation with in-situ testing might be appropriate; for larger schemes a more detailed geotechnical investigation is likely to be needed.

The scope of the site investigation should be determined by the project's geotechnical engineer with consideration given to all parties that will be involved with the earthworks in future, particularly the earthworks contractor (input from whom at an early stage is to be encouraged).

The recommendations and guidance within this section apply to both new construction and the investigation of existing earthworks; the GI should be planned to suit the project.

6.1.2 Planning

NOTE BS EN 1997-2:2007 identifies the need to plan the investigation to provide sufficient information for the different stages of design; further guidance is given in BS 5930:1999+A1.

The geotechnical designer should be consulted and contribute to all stages of the investigation.

When planning a phase of ground investigation, it is important to consider the needs of all those who will use the data obtained, at a later phase of the scheme, e.g. if the project will go to design and build tender, what information will be required to evaluate the scheme?

If the observational method is the design approach, certain requirements for the GI should be addressed at an early stage (see BS EN 1997-1:2004, CIRIA R185 [16]). However, the GI and design process for earthworks projects should be responsive to ground conditions encountered, so there should always be an element of observation and response. Therefore, the GI may be planned accordingly from an early stage, which can be advantageous, (e.g. position of instrumentation to ensure future use through the project).

The investigation of failed earthworks should be given special consideration when planning investigations. Of paramount importance are the on-going risks posed by the failed earthwork and the economic implications for the earthworks owner. The investigation should be designed to identify the failure mechanism and provide sufficient information to design an engineered solution to the problem.

Existing earthworks that will be subject to modification e.g. embankment widening should be given special consideration when designing a GI to take account of the potential for differential settlement, increased porewater pressures and reduced slope stability.

6.1.3 Testing

The testing regime, both in situ and within the laboratory, should form an integral part of any GI to enable site characterization and material classification for both design and construction purposes.

The choice of testing method to control the works should maximize the volume of usable material and minimize disruption to the works. The use of laboratory relationship testing (MCV : mc : dry density : strength) in advance of the works is advisable; this approach will often enable control of earthworks by MCV which minimizes disruption of the works.

Relationship testing to identify an acceptable range of moisture contents for the material to be used in earthworks should be undertaken under Category 2 and Category 3 projects. An adequate amount of soil should be recovered as bulk samples to enable a range of testing to be undertaken at a set of moisture contents, allowing an assessment to be made of the acceptable range of moisture content of the material for use in earthworks (see HA44/91 [17] and HA70/94 [18] for details). An assessment may then be made for the appropriate treatment of marginal material falling outside of this acceptable moisture content range, or one of the other required material suitability criteria such as grading. Additional testing may be required as part of this assessment of marginal materials.

Table 2 provides a summary of the earthworks testing that is most commonly used to follow the SHW [1], and appropriate tests from Table 2 should be selected for the investigation and design stage.

COMMENTARY ON 6.1.3

The nature of soils tests undertaken for earthworks are detailed within BS EN 1997-2:2007 and the various supporting documents. Experience has shown that more meaningful results can be obtained for earthworks by modifying tests to take account of local soils and conditions. This subclause is provided to identify the tests most commonly recommended to control earthworks (especially when following the SHW [1]) and to comment on some of the main issues that ought to be considered both during GI and design of the earthworks.

Further details on some of the issues are provided within HA 44/91 [17] and HA 70/94 [18], which provide information on the selection and assessment of appropriate testing for the control of earthworks and the design assumptions that form part of the SHW [1]. This topic is summarized in 7.6.4. The information on earthworks control shown in Table 2 is only provided to illustrate how the test might be taken into the construction stage.

6.1.4 Geotechnical reporting

Site investigation and design should be undertaken as a phased process in order to ensure that the ground conditions of the site are adequately understood for the works to be constructed (see *Site Investigation in Construction* [20] for guidance⁶⁾). The geotechnical reporting should be dependent on the complexity of the scheme, as identified in the subsequent sections, but in most cases this will also follow a phased process.

All phases of the geotechnical reporting should feed back into the geotechnical risk register (see Clause 4) as appropriate to the scale of the project.

NOTE The reporting approach required to satisfy BS EN 1997 (both parts) includes the preparation of reports at various phases in the ground investigation, design and construction process. These are set out in 6.3, 6.4 and Figure 2 in the context of earthworks.

6.1.5 Soil and rock descriptions and classification

6.1.5.1 Description

Soils should be described in accordance with BS EN ISO 14688-1:2002 and BS EN ISO 14688-2:2004.

Rocks should be described in accordance with BS EN ISO 14689-1.

6.1.5.2 Classification

Earthworks materials should be classified in accordance with Table 6/1 of SHW [1].

The classification of the materials involved during excavation, transportation and deposition can vary, hence soil/fill may be classified at any of the following stages:

- in situ – classification in undisturbed condition prior to excavation;
- on excavation – disturbed material after excavation; and
- on deposition – classification following placing and prior to compaction.

The option for classifying soil/fill should be selected which is most appropriate for the particular project logistics and materials to be worked with, and should maximize the potential to win suitable fill from the site.

Classification should be based on both descriptive determination and standard use of materials, such as SHW [1] Table 6/1.

⁶⁾ Currently under review.

Table 2 Indicative earthworks tests by test type and material type

| Test type | Material type | Applicability | | | Uses | | Comments |
|-------------------------------------|---------------|---------------|----------------------|--------------------------|--|--|----------|
| | | Design | Construction control | Stabilization and re-use | | | |
| Natural moisture content | F,M,C,R | ✓ | ✓ | ✓ | Classification and compaction control | Used for comparison with laboratory moisture content values determined as part of relationship testing suites | |
| Particle size distribution | F,M,C,R | ✓ | ✓ | ✓ | Classification | Used for determining fill material grouping and assisting compaction plant selection | |
| Atterberg limits | F,M,C | ✓ | ✗ | ✓ | Classification | Used to derive behavioural characteristics and preliminary engineering properties in cohesive fill, e.g. A-line plot in BS 5930:1999+A1, Figure 18 | |
| Particle density | F,M,C,R | ✓ | ✓ | ✓ | Classification | Used for determining loadings, bulking and compaction control (air voids determination) of fill materials | |
| 2.5 kg rammer compaction | F,M | ✓ | ✗ ^{A)} | ✓ | | | |
| 4.5 kg rammer compaction | F,M | ✓ | ✗ ^{A)} | ✓ | Determination of dry density/moisture relationship | Used to specify moisture content limits for use of material as fill (may be used in conjunction with CBR Test) | |
| Vibrating hammer compaction | M,C | ✓ | ✗ ^{A)} | ✓ | | A) Additional tests are commonly undertaken during construction to validate the relationship. | |
| Moisture condition value (MCV) | F,M | ✓ | ✓ | ✗ | Classification and compaction control | Used for assessing fill material suitability for specification/design and in-situ monitoring of sources of fill | |
| California bearing ratio (CBR) | F,M | ✓ | ✓ | ✓ | Formation strength determination | Used to determine pavement construction thicknesses and assist in-situ compaction control during construction | |
| Undrained shear strength parameters | F,M | ✓ | ✗ | ✓ | Design of earthworks subject to undrained loading conditions | Temporary slope and foundation design for construction purposes; assessment of plant trafficability; permanent works constructed with cohesive soils, subject to rapid loading | |
| Drained shear strength parameters | F,M | ✓ | ✗ | ✗ | Design of earthworks subject to drained loading conditions | Slope and foundation design for long-term temporary or permanent works; temporary works constructed with granular soils | |
| Los Angeles abrasion test | M,C | ✓ | ✗ | ✗ | Design of permanent works | Used for selected fill materials | |
| Plate load test | F,M,C | ✓ | ✓ | ✓ | Design and compaction control | Used for assessment of settlement characteristics and bearing capacity at formation level and of compacted fills | |
| Dynamic cone penetrometer | F,M,C | ✓ | ✓ | ✓ | Design and compaction control | Used for designing foundations (bearing capacity), formations and in-situ compaction monitoring | |

Table 2 Indicative earthworks tests by test type and material type (continued)

| Test type | Material type | Applicability | | | Uses | Comments |
|-----------------------------|---------------|---------------|----------------------|--------------------------|---|--|
| | | Design | Construction control | Stabilization and re-use | | |
| MEXE probe | F,M,C | ✓ | ✓ | ✓ | Design and compaction control | Used for designing foundations (bearing capacity), formations and in-situ compaction monitoring |
| Dynamic plate load test | F,M,C | ✓ | ✓ | ✓ | Design and compaction control | Used for designing foundations, formations and in-situ compaction monitoring. The form of test adopted for pavement foundation design is the lightweight falling weight deflectometer (LWD). |
| Clegg impact soil tester | F,M | ✓ | ✓ | ✓ | Formation strength determination | Used to determine pavement construction thicknesses and assist in-situ compaction control during construction |
| Field density test | F,M | ✓ | ✓ | ✓ | Design and compaction control | Used to determine in-situ densities and assist in-compaction control during construction |
| pH, SO ₄ , Cl | F,M,C,R | ✓ | ✓ | ✓ | Design and confirmation during construction | Determination of aggressive ground conditions for cementitious products and buried metallic structures and elements (see BRE SD1 [13]) |
| Redox potential/resistivity | F,M,C,R | ✓ | ✓ | ✓ | Design and confirmation during construction | Determination of aggressive ground conditions for buried metallic structures and elements |
| Chemical analysis | F,M,C,R | ✓ | ✓ | ✓ | Design risk assessments and safe construction | Determination of toxic elements/compounds for environmental and health and safety control during construction |
| Waste acceptance criteria | F,M,C,R | ✓ | ✓ | ✓ | Design and confirmation during construction | Determination of chemical characteristics for disposal offsite to landfill |

Key

F = soils not containing more than 10% retained on a 2 mm test sieve

M = soils containing more than 10% retained on a 2 mm test sieve but not containing more than 10% retained on a 20 mm test sieve

C = soils containing more than 10% retained on a 20 mm test sieve but not containing more than 10% retained on a 37.5 mm test sieve

R = zone "X" material and rock fill

Based upon BS 1377 (all parts) and *Manual of Laboratory Soil Testing* [19].

6.2 Site characterization and investigation

NOTE 1 See BS EN 1997-2:2007, 2.1 and 2.2.

The stages of a geotechnical investigation should include a desk study (sometimes referred to as a preliminary sources study), a preliminary investigation, to characterize the site in general terms, and wherever required, subsequent phases of design investigation to provide detailed information for specific elements of the design. These phases are briefly described within the following subclauses.

NOTE 2 Under certain circumstances, it is impracticable to use a phased approach to a geotechnical site investigation beyond the desk study. The ground investigation phases have to be combined. This is particularly true for investigations in a railway environment or major trunk road where opportunities for access are limited.

6.3 Desk study

6.3.1 General

A desk study is an essential prerequisite for planning a ground investigation and informing subsequent design. Some general guidance on desk studies that should be consulted is provided in BS 5930:1999+A1 and BS 1997-2:2007, 2.1.1.

Feedback (through an iterative process) should be made from the desk study to the overall conception and design of the project, where warranted by its complexity. Preliminary information may be fed back into the project risk register at an early stage to aid conceptual design and preliminary planning application(s).

The desk study for earthworks projects should include an evaluation of both internal and external geotechnical influences on the project. This may include both current and historical land use, ground stability, an examination of existing earthworks, surface and groundwater conditions, contamination, etc. The desk study should include an adequate site-based inspection of the site and adjacent land by the geotechnical engineer; the use of geomorphological mapping techniques for this activity can prove advantageous.

The desk study should provide a design for the preliminary investigation based on the information available at that point in time.

The desk study should enable an initial input to geotechnical and project risk register(s) (see Figure 2).

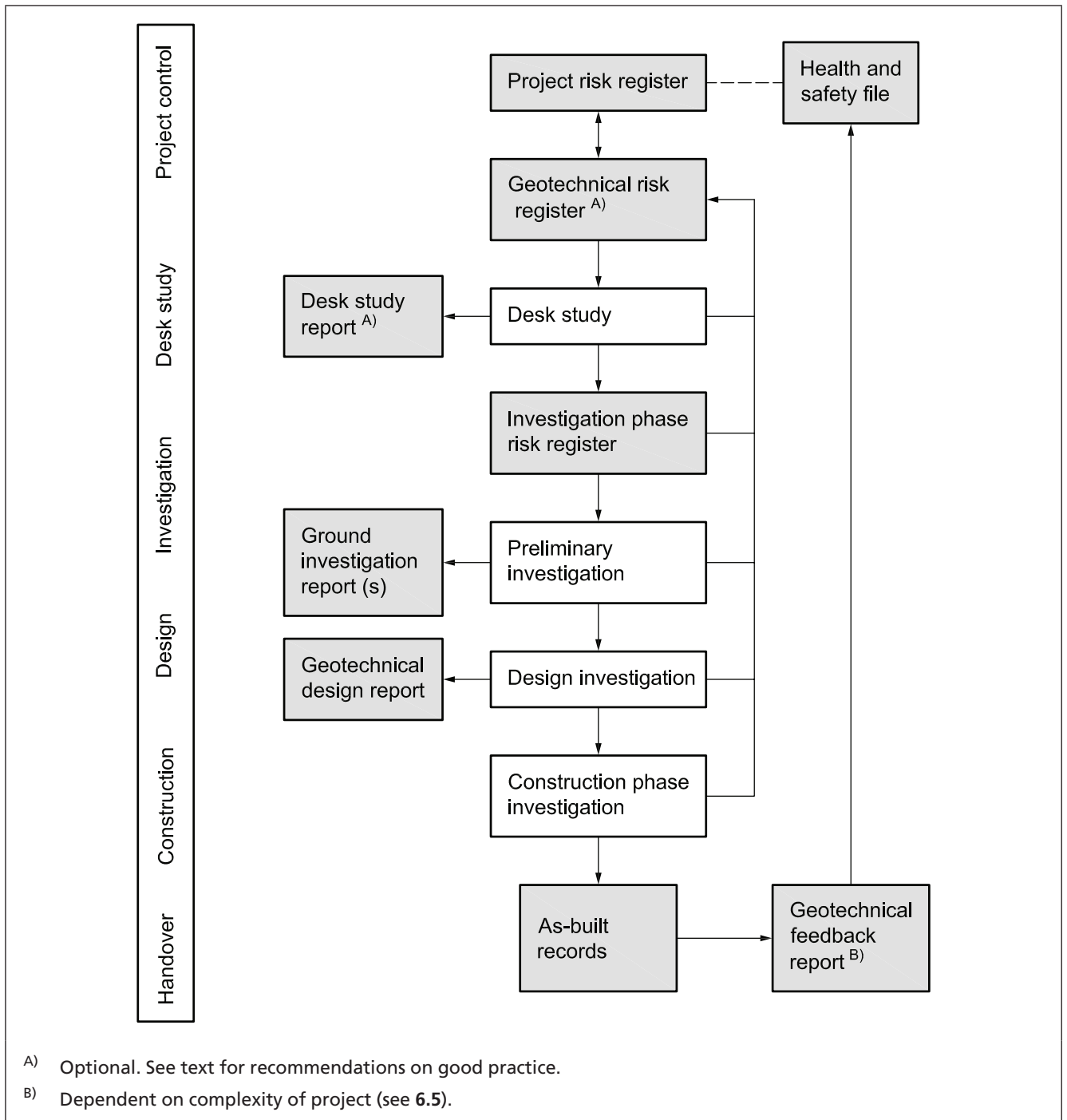
Where appropriate an initial review may be undertaken based on the desk study information obtained of the various engineering options available for the scheme; when undertaken this generally forms the first phase of the project planning process.

6.3.2 Extent of desk study

6.3.2.1 General

The scope and extent of a desk study should be commensurate with the proposed works. The desk study for widening of an existing highway embankment may be limited to a walk-over survey and review of existing as-built drawings, whereas the desk study for 5 km of new flood defence embankment through an industrial area may require enquiries of multiple sources.

Figure 2 The iterative feedback processes of a site investigation



6.3.2.2 Existing earthworks

A desk study for projects involving existing earthworks should seek to make maximum use of construction, inspection and maintenance records available for the earthwork, and give consideration to common performance problems with that particular type of earthwork (e.g. likely method of construction given age of earthwork).

A thorough search of drainage plans and records is essential when dealing with existing earthworks. Any maintenance records of infrastructure supported by the earthworks should be included in the desk study as these might highlight areas with potential underlying problems.

6.3.2.3 New-build earthworks

In addition to the recommendations of 6.3.1, attention should be paid to areas where land drains are present, as these might require interception. Topographical low points should be identified as these might be underlain by relatively soft, recent or weathered soils. The presence and extent of alluvium should be gleaned from geological maps and memoirs; this is particularly important when studying flood plains of large, mature river systems.

6.3.2.4 Reporting of desk study

COMMENTARY ON 6.3.2.4

In the UK, the desk study report (DSR) may be presented as a preliminary sources study report (PSSR) in accordance with HD 22/08 [11].

The presentation of desk study findings should be dependent on the scale of the project. Desk study information should be collated in a coherent form and be made available for all interested parties, including designers of ground investigations and designers of the earthworks. This may be achieved by the preparation of a formal DSR. However, if a DSR is not prepared, the information should be collated within the project file and made available to the project team; one option is to include the data within the ground investigation report (GIR).

NOTE The potential benefit of compiling the desk study information and interpretation into a formal DSR depends on the size of the project and complexity of the ground conditions. The DSR can be an important document in itself and may be used for preliminary planning applications, design, etc.

6.4 Ground investigation

6.4.1 Preliminary investigation

COMMENTARY ON 6.4.1

See BS EN 1997-2:2007, 2.3.

The preliminary investigation is undertaken once the general scope of the project has been identified to provide an overall understanding of the ground conditions at the site and thereby minimize the geotechnical risk associated with the project (see Clause 4). The outputs from the preliminary investigation are likely to include some or all of the following:

- *ground investigation report;*
- *feedback to geotechnical risk register;*
- *feedback to project risk register (option analysis);*
- *proposals for design investigation.*

6.4.1.1 Purpose and extent of preliminary investigations

The specific parameters that should be evaluated depend on the nature of the works; with different priorities being applied to new build, modification and repairs. Four broad categories of earthworks related assessment should be considered.

- a) Shear strength and stiffness (consolidation) characteristics of in-situ soils that will be subject to changes of in-situ stress; e.g. soils below formation level in cut zones and fill zones, soils that will form batter slopes in areas of cut. This may include existing earthworks that are to be incorporated into new works; e.g. raising of an existing embankment.

- b) Characteristics of fill materials after compaction. This may include classification of fills in terms of SHW Table 6/1 and establishing relationships between moisture content and other compaction control related parameters. It may include estimation of shear strength and stiffness parameters.
- c) Assessment of ground for excavation. This may cover estimation of volumes of suitable, or potentially suitable, fill material available from identified borrow areas or other sources. The assessment may also include rippability of material to be excavated to form cuttings.
- d) Assessment of the hydrological and/or hydrogeological conditions that may influence the design and construction of the earthworks.

A preliminary investigation for existing slopes and earthworks should include:

- a check for existence of shear zones;
- gathering data to enable back analysis of existing slopes for comparison with laboratory parameters;
- highlighting potential access difficulties for remedial works; and
- consideration of the possible need for specialist plant.

6.4.1.2 Reporting of preliminary investigations

Preliminary investigations should be presented within the GIR. The degree of detail in the GIR should be consistent with the objective of the particular phase of ground investigation.

As required by BS EN 1997-2:2007, 6.1, the GIR should include both factual information and evaluation.

COMMENTARY ON 6.4.1.2

BS EN 1997-2:2007, 6.1 states that "the GIR shall form a part of the Geotechnical Design Report". It goes on to state that:

"(2) The Ground Investigation Report shall consist of the following:

– a presentation of all appropriate geotechnical information including geological features and relevant data; [commonly referred to as a "factual report" in the UK]

– a geotechnical evaluation of the information, stating the assumptions made in the interpretation of the test results. [historically included within an "interpretative report" in the UK]."

"(3) The information may be presented as one or as separate parts."

6.4.2 Design investigation

COMMENTARY ON 6.4.2

Design investigation is undertaken to clarify ground conditions at selected locations or to clarify particular issues to enable the design (there can be as many phases of design investigation as are required).

See BS EN 1997-2:2007, 2.4.

6.4.2.1 General

The design investigation should concentrate on specific aspects of the geotechnical design that mitigate the risks identified in the geotechnical/project risk register(s), where appropriate.

6.4.2.2 Purpose and extent of design investigation

The design investigation for an earthworks project should include classification testing, soil relationship testing and marginal materials assessment (see 6.1.3). The design investigation should elaborate on and develop the classification and relationship testing undertaken as part of the preliminary investigation.

At this stage the designer may give consideration to the likely approach to the earthworks and arrange for appropriate specialist testing. Sufficient numbers of large bulk samples, particularly from trial pits, should be specified in the design investigation to enable extensive laboratory trials to be carried out. These may include strength and stiffness testing and undertaking stabilization trials (modification of marginal materials at varying additions of binder). These trials consume large volumes of material but it should always be considered that reducing the number and size of bulk samples can be a false economy; this is especially the case where factors beyond the control of the project team preclude further GI.

Where appropriate, the design investigation should include the installation of monitoring instrumentation such as piezometers and inclinometers. Further guidance is given in Dunnicliff [21].

The design investigation should be designed to meet and, if needs be, enhance/expand on the requirements of BS EN 1997-2:2007.

6.4.2.3 Reporting of design investigation

A geotechnical design report (GDR) should conform to BS EN 1997-1:2004, 2.8. HD 22/08 [11] has been drafted in compliance with BS EN 1997 (both parts), with a particular emphasis on the requirements of a major earthworks project.

The following items might be of particular relevance to an earthworks design and should be considered for inclusion in the GDR as appropriate.

- a) Draft specification of fill materials. This may be in the form of draft numbered appendices in accordance with the SHW [1]. In particular, the designer should be looking to set upper and lower limits for identified parameters in Table 6/1.
- b) Formalized charts for relationship testing of fills.
- c) The proposed scheme for monitoring and measuring compliance with the specification. This may be in the form of a draft Appendix 1/5 in accordance with the SHW [1].
- d) Deformation limits and associated monitoring requirements if an observational design approach is being followed. The designer should make clear any assumptions made in the design. In particular, the GDR should make clear any assumed sequence of works as this may affect the routing of haul roads or selection of plant.

6.4.3 Further investigation during construction

It should be appreciated that no ground investigation, however carefully done, ever examines more than a very small proportion of the ground. It is essential that the soil conditions revealed during progress of the excavations are checked to see that they correspond with those forming the basis for earthworks design as interpreted from

the ground investigation; it might be necessary to undertake further investigation to determine the extent of anomalous conditions.

NOTE Guidance is given in BS 5930:1999+A1.

6.5 Geotechnical feedback

The iterative feedback process that should be followed, including the production of a geotechnical feedback report (GFR) if appropriate, is illustrated in Figure 2.

COMMENTARY ON 6.5

BS EN 1997-2:2007 does not include a requirement for a GFR to be produced to record the construction works phase of a project, however BS EN 1997-2:2007, Annex B, under "Execution of works", does advise a "report from inspection, supervision and monitoring", and this would at least in part be satisfied by the production of a GFR. As stated at Clause 4, the GFR is a document of value for capturing geotechnical data from an earthworks project (this would also be a valuable part of the CDM health and safety file).

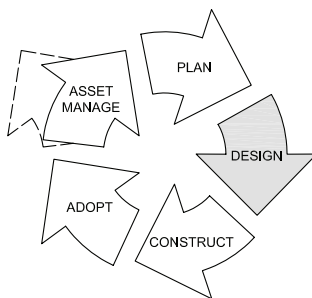
A GFR is usually prepared on completion of construction.

Generally two broad types of information are contained in a GFR:

- a) geotechnical design changes during construction; and
- b) results of monitoring and testing conducted during construction.

It forms an ideal vehicle that may be used for reporting the results of compliance testing (e.g. in-situ density measurement, plate load tests), distribution of fill classes within an earthwork, and data from monitoring instruments (e.g. piezometers, inclinometers, settlement gauges). The report should give a commentary on how the results of testing and monitoring have compared with the expected or specified range of values. This is particularly important where the observational method of design has been applied. The commentary should include a time-line that allows a chronological understanding of events, e.g. relating instrument readings to filling operations.

NOTE The benefits of preparing a geotechnical feedback report are both for maintenance of the site and to capture "lessons learned" for future projects: as-built drawings alone generally do not capture all the information relevant to earthworks that will be relevant to future maintenance of the earthwork; an appropriate range of information to be included within the geotechnical feedback report is detailed within HD22/08 [11].



7 Design of earthworks

7.1 General

7.1.1 Introduction

Clause 7 should be consulted for information relating to the design of earthworks that are intended to be self supporting (excavations requiring temporary support are covered in Clause 13).

Earthworks that incorporate some degree of reinforcement should satisfy the recommendations of BS 8006-1 or BS 8006-2, in addition to this standard, where relevant.

7.1.2 Concept of BS EN 1997-1:2004 geotechnical categories

Earthworks should be designed in accordance with the principles and application rules of BS EN 1997-1:2004, which is intended to be used in the United Kingdom as a general basis for the geotechnical aspects of the design of buildings and civil engineering works.

NOTE BS EN 1997-1:2004, Section 5 (fill, dewatering, ground improvement and reinforcement), Section 11 (overall stability) and Section 12 (embankments) are particularly relevant to the design of earthworks whilst Section 1, (general), Section 2 (basis of geotechnical design) and Section 3 (geotechnical data) provide guidance on the application of design rules and the selection of parameters used in the design.

BS EN 1997-1:2004, 2.1 introduces the concept of limit state design to earthworks, typically the ultimate limit state (ULS) and the serviceability limit state (SLS); earthworks should be designed such that the relevant limit states are not exceeded during their design life.

COMMENTARY ON 7.1.2

Instead of using global factors of safety that have been adopted previously in traditional earthworks design, BS EN 1997-1:2004 adopts the approach of applying partial factors to actions (loads) and the effects of actions, materials (soil parameters) and earth resistances.

To establish the geotechnical design requirements, BS EN 1997-1:2004 recommends the classification of geotechnical structures into three geotechnical categories according to the complexity of the structure, the ground conditions, the loading and the level of risk that is acceptable. The Geotechnical Categories are used to establish the extent of site investigation required and the amount of input to the design. The Geotechnical Category should be checked throughout the design and constructions process and the Category amended if necessary as information becomes available.

Geotechnical Category 1 – for small and relatively simple structures for which it is possible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations. Category 1 structures carry negligible risk.

Geotechnical Category 2 – encompasses conventional geotechnical structures with no exceptional risk or difficult ground or soil loading conditions, most earthworks will fall into this category.

Geotechnical Category 3 – includes very large or unusual structures which are not included in Geotechnical Categories 1 and 2. Examples of such Category 3 structures (which could be earthworks) are given as: structures involving abnormal risks or unusual or exceptionally difficult ground or loading conditions; structures in highly seismic areas; structures in areas of ground instability or persistent ground movements that require separate investigation or special measures.

BS EN 1997-1:2004 permits the use of three design approaches but the National Annex adopts Design Approach 1 (DA1). DA1 includes two combinations and the application of these combinations is discussed in 7.3.3.

7.1.3 Design input at GI stage

The scope and extent of the ground investigation should reflect the BS EN 1997-1:2004 Geotechnical Category of the project. For Category 2 or 3 projects the GI will normally be undertaken as a staged process (see 6.4); this process might require a certain amount of design to be

undertaken as the GI develops in order to confirm the scope of certain aspects of the GI.

7.1.4 Detailed design using GI data

The detailed design should be undertaken based on the GI data obtained; BS EN 1997-1:2004, 2.8 requires that a geotechnical design report is prepared (see 6.4.2) that identifies how the GI data has been interpreted into the design including justification for the design values adopted for soil and rock properties. The geotechnical design report should also identify ground condition issues that should be checked during construction.

7.1.5 Geotechnical certification

Consideration should be given to the adoption of a formal system of geotechnical certification (see 4.3), for recording formally the Geotechnical Category and for certifying the design process at specific stages; BS EN 1997-1:2004 Section 4 provides recommendations for the procedures to be adopted for supervision, monitoring and checking during the construction process.

7.2 Factors governing the stability of slopes

7.2.1 Introduction

As part of the design of earthwork slopes, the designer should assess aspects that can be considered by calculation (e.g. stability of slopes, potential for adverse settlement, scour), and also other potential modes of failure that might require engineering judgement and precedence to be used rather than relying entirely on calculation (e.g. erosion, influence of animals or vegetation). The site observation, modelling of ground conditions and the risk assessment process is an important part of the design of earthworks to help identify potential modes of failure.

The overall stability of slopes should be assessed based on the requirements given in BS EN 1997-1:2004, 11.5, and designers should consider whether it is appropriate to assess deformation of the ground as detailed in BS EN 1997-1:2004, 12.6. The applicable actions for different design situations should be considered (BS EN 1997-1:2004, 11.3) which for earthworks can include external influences such as earthquakes and pile driving.

7.2.2 Materials and ground conditions

For the purpose of making a preliminary assessment of stability conditions and for guidance in formulating a field or laboratory testing programme, consideration should be given to previous relevant experience and published information to obtain an indication of the behaviour of a particular type of soil when excavated to form slopes and platforms. 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 slopes and platforms.

The designer should review the nature of the materials that form or influence the stability of the slope and consider which attributes of the material could have a significant influence on the potential modes of failure:

- grading/permeability of the soil (coarse soil or fine soil, or intermediate soil that may show attributes of both) and groundwater conditions, these factors tend to dominate the likely form of instability within soil slopes in most general cases; further discussion on this topic is provided at 7.2.4;
- previous stress conditions are of particular importance to fine grained soils which can be in a normally consolidated or overconsolidated state;
- presence of geological structure within the soil or rock, such as bedding planes, laminations, fissures or other discontinuities;
- presence of zones of contrasting permeability;
- presence of historical slip surfaces where previous movement was sufficient to generate smooth (slickensided) surface with a "residual" shear strength in clay soils;
- weathering of fine grained soils or rocks leading to a reduction in strength of the material often resulting in zones of weakness (e.g. along fissures in soil or rock, and can lead to karst conditions in limestone), and the leaching of minerals under prolonged seepage or other weathering phenomena can lead to the development of sensitive soils prone to collapse on disturbance in some normally consolidated fine soils.
- influence of human activities in the form of mining.

Once the potential modes of failure have been identified, each should be assessed by a method suitable for the material type and reflecting the geotechnical category of the structure. Details of methods of analysis are provided within soils mechanics references such as Bromhead [22].

7.2.3 Actions

NOTE 1 Load cases for earthworks design usually comprise externally applied actions and the self-weight of the earth structure itself.

For externally applied actions, details should be obtained of static, transient and dynamic loads that might be applied to the earthworks. A minimum surcharge of 10 kN/m² should be applied to the surface at the top of embankments and cuttings where the external action might adversely affect the stability of the slope.

NOTE 2 This requirement is not in addition to any specific live loading of equal or greater magnitude that is included within the slope design model.

The minimum surcharge should be considered as a permanent load and appropriate partial factors should be applied to the action.

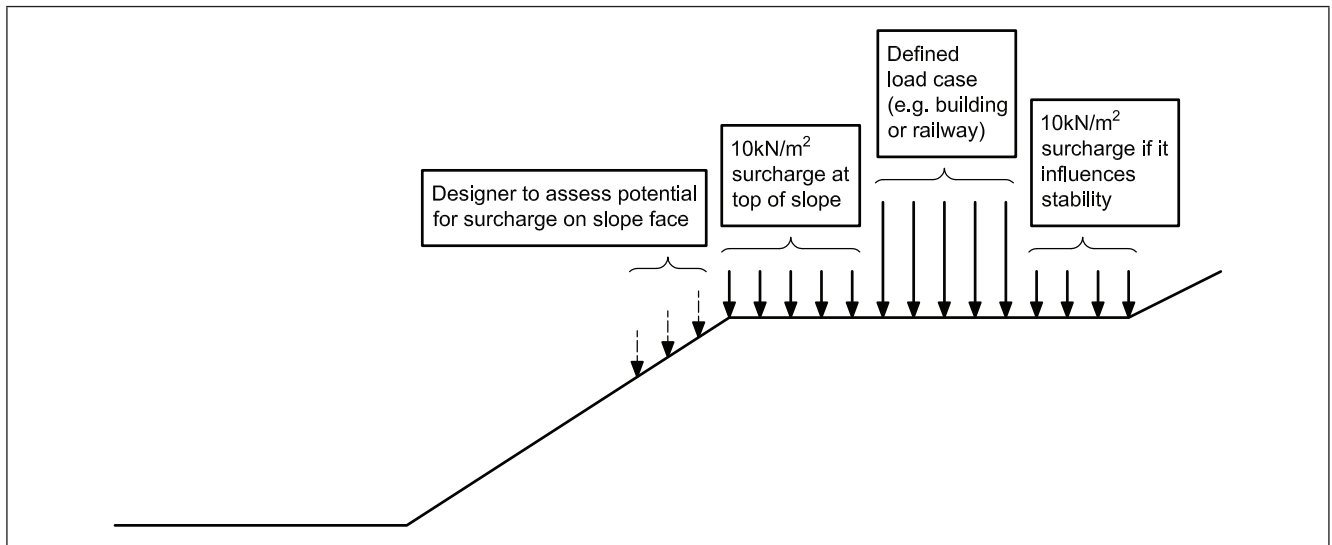
Additional surcharge loading should be applied to take account of actions resulting from loads imposed on the earthworks during construction and during the design life. The surcharges applied to the earthworks may be classified as:

- uniformly distributed load (UDL) consisting of a continuous load on the surface, this may be a defined load case (e.g. railway industry RL or RU loading), or a general surcharge to represent construction plant, stored materials (10 kN/m² minimum);

- concentrated loads (e.g. pad foundations);
- line loads (e.g. strip footings);
- dynamic loads (e.g. impact loads), these are generally modelled as a UDL. In some specific cases impact loads may be modelled as point loads.

The combination of these load cases can result in a combination of applicable surcharge being applied to the slope; the designer should identify the applicable surcharges to be modelled. This situation is illustrated in Figure 3 as a possible design scenario.

Figure 3 Example of possible surcharge combination on a slope



In the absence of more exact calculations, the nominal loads due to live load surcharge may be taken from Table 3.

Table 3 Nominal load due to live surcharge

| Standard load | Uniformly distributed load UDL kN/m ² | Typical applicable design cases |
|-------------------------|---|---|
| No specified load case | 10 | Earthworks slopes where maintenance equipment might present an adverse load case. |
| Typical highway loading | 10 | Common practice is to assume this value. Extreme cases agreed on a site-specific basis. |
| RL loading | 30 on area occupied by tracks | London Underground and other light rail systems |
| RU loading | 50 on area occupied by tracks (see Note 3) | "Rail universal" used for all standard UK railways |

NOTE 3 RU loading is included in Table 3 because this has formed the basis of design for many years. However, in future UK railways will change their standards to apply surcharge loads in accordance with Eurocodes. It remains important that the designer considers the likely distribution of load below the track and the dispersal of load with depth through the soil.

A clear understanding of the loads that will be applied to the earthworks should be obtained from the asset owner; otherwise, the

designer should identify to the asset owner the restrictions on surface loading of the earthworks.

The designer should give consideration to the duration of applied loads and the selection of appropriate soil parameters for the assessment of slope stability, i.e. whether drained or undrained conditions are appropriate for the duration of the load. The designer should consider that some surcharges might exist for short periods of time only, in which case, the ground can respond in an undrained manner during the entire period of the application of the surcharge.

The dead load may be assumed to be the load of the earthwork.

NOTE 4 BS EN 1997-1:2004 distinguishes between permanent loads and variable loads that are applied to the slope in so far as different partial factors are applied to each type of load. Permanent loads will, for example, comprise structures and buildings whilst variable loads will normally consist of traffic or rail loading or other transient loads. Loads imposed by structures and buildings will consist of both the dead load from the structure and live load applied to the structure; unless the live load forms a significant proportion of the total load from the structure the load applied to the earthworks by the structure may be assumed to be permanent. An example of where variable load from a structure might need to be applied separately to the dead load is in the case of a service reservoir located at the crest of a slope. In this case the loading from the water might be greater than the dead load of the structure and the load might fluctuate throughout the design life of the structure.

Care should be exercised when applying surcharge loads to the slope face since these loads can act either favourably or unfavourably (i.e. they might contribute to either destabilizing forces or to restoring forces) depending on their position on the slope.

7.2.4 Selection of parameters

7.2.4.1 General

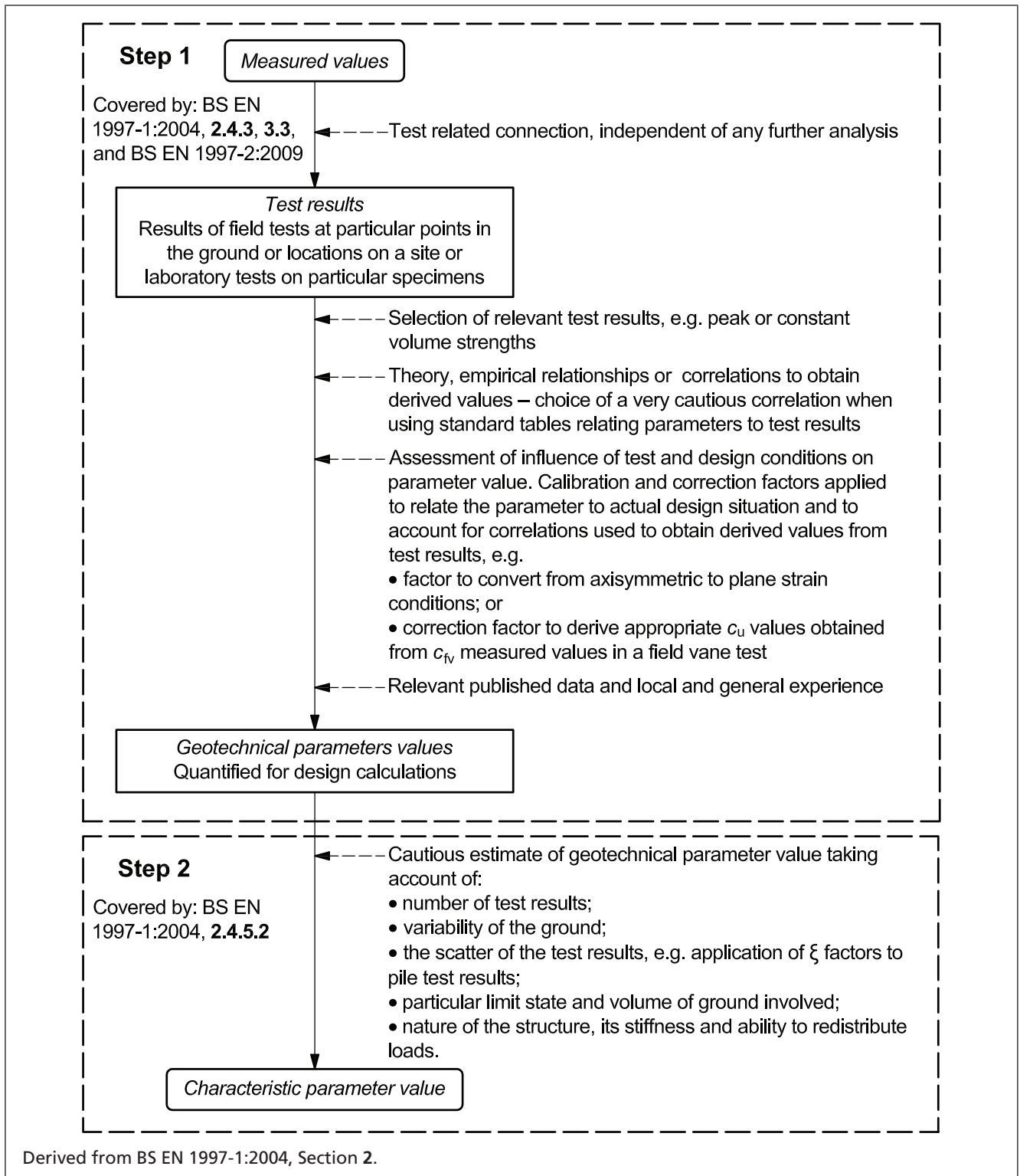
NOTE 1 BS EN 1997-1:2004, 2.4.3 requires ground properties to be obtained from test results or from other relevant data. Such data might be, for example, back calculations, empirical or theoretical correlations, field measurements and observations or published data.

The assessment of geotechnical parameters from tests should take account of the difference between the properties obtained from the tests and those that govern the behaviour of the mass of the ground forming the embankment or cutting.

NOTE 2 Potential influencing factors: for slope design the potential for strain-softening behaviour or brittleness can be of particular concern for cohesive soils as a significant loss of resistance can occur if the peak strength is exceeded locally (see 7.2.7 and Table A.1 regarding progressive failure).

Characteristic values for the geotechnical parameters should be selected in accordance with BS EN 1997-1:2004, 2.4.5. The process of selecting the characteristic values from the ground properties may be divided into two stages. First, establish the values of the appropriate ground properties and second, select the characteristic value as a cautious estimate of the value affecting the limit state under consideration taking into account all relevant information. This process is illustrated in Figure 4.

Figure 4 General procedure for determining characteristic values from measured values



When selecting the characteristic values, due account should be taken of the items listed in BS EN 1997-1:2004, 2.4.5.2, the following items are of particular relevance to slope design:

- geological, historical and other background data;
- the amount of measured data relating to the parameter value under consideration;

- the variability of the measured data and the degree of confidence in the data;
- the extent of the zone of ground governing the limit state under consideration;
- the ability of the ground to transfer load from weak to strong zones; and
- the consequences of failure at the limit state under consideration.

NOTE 3 Statistical methods may be used to select characteristic values for the ground properties and procedures for the application of statistical methods for this purpose are described in detail by Frank et al [5]. The use of statistical methods implies that sufficient data are available to permit a meaningful evaluation to be undertaken and, where statistical methods are used, BS EN 1997-1:2004, 2.4.5.2 recommends that the calculated probability of a worse value governing the occurrence of the limit state under consideration should be no greater than 5%.

Although the use of statistical methods is permitted by BS EN 1997-1:2004, the use of such methods should be adopted with caution unless a large population of data is available for the geotechnical parameter under consideration. Statistical methods should not be used as a substitute for reasoned judgement of the geotechnical parameter which takes account of all the relevant data related to the parameter.

The design values of geotechnical parameters should either:

- be derived from characteristic values by the application of a partial factor in accordance with BS EN 1997-1:2004, 2.4.6; or
- be assessed directly.

Partial factors are shown in NA to BS EN 1997-1:2004 and these factors indicate the minimum level of safety for conventional designs that should be used. An increased level of safety should be specified for unconventional designs or earthworks where the consequences of failure are especially onerous. If design values of geotechnical parameters are assessed directly, the partial factors given in the National Annex should be used as a guide to the required level of safety.

The selection of parameters for design of slopes should consider both the soil grading and, in fine grained soils, the nature of the fines content which is commonly defined by the soil plasticity. It should be remembered that, in UK practice, the definition of material described as "intermediate" differs when used for:

- a) selection of material parameters for slope stability design (where a soil with more than 35% fines is normally defined as "fine grained"); as opposed to
- b) classification of the material as a fill (where more than 15% fines is the change point within the SHW [1]), see Table 1c) and 7.6.2 for details.

COMMENTARY ON 7.2.4.1

Figure 5 illustrates two types of soil: one where the minimum conceivable value of soil strength is represented by the critical state parameters ϕ'_{cv} , c'_{cv} (where c'_{cv} will normally be zero) and the second in which very low residual strengths ϕ'_r , c'_r (where c'_r will also normally be zero) can develop at large displacements. These two types of soil may be categorized by plasticity index, I_p (see Figure 6). However, according to the data of Lupini [23], the distinction between turbulent shear and sliding shear for fine soils is not well-defined; there is a transitional zone. The distinction at $I_p = 25\%$ is an over-simplification but provides a useful rule-of-thumb. The parameter ϕ'_{cv} will generally lie in the range 30° to 35° for granular fills and in the range 20° to 25° for clay fills.

Figure 5 Variations of ϕ' with displacement

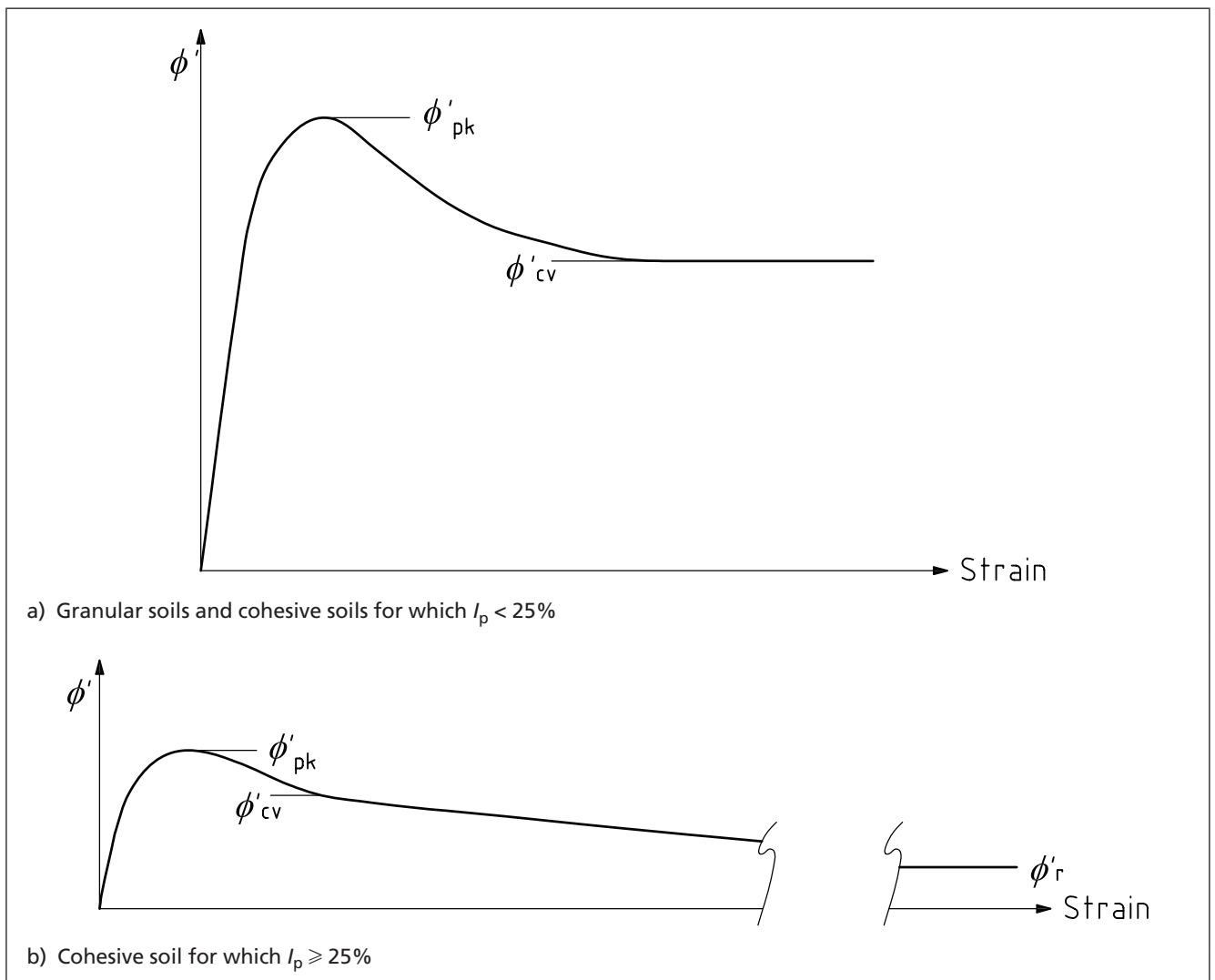
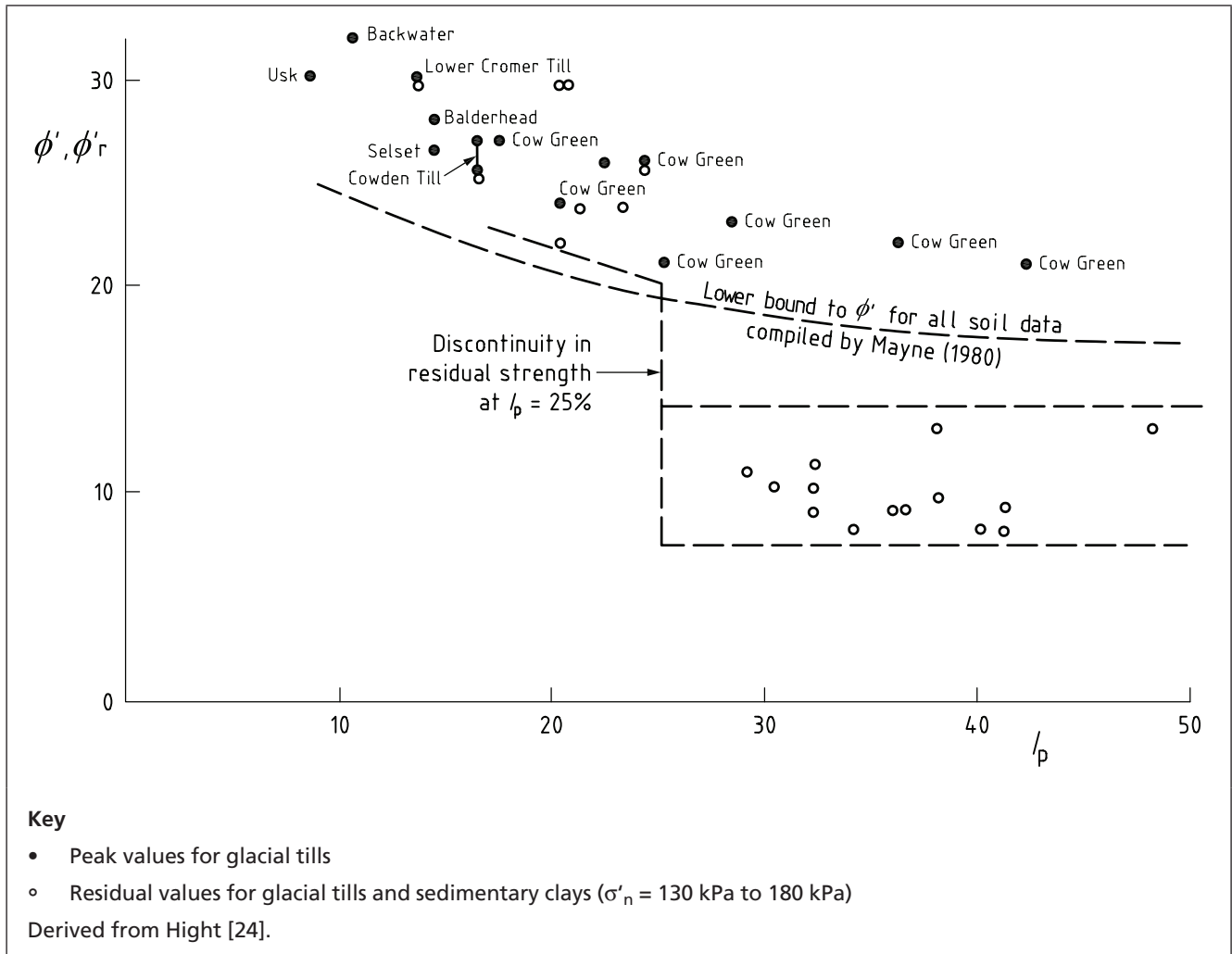


Figure 6 Variation of ϕ' , ϕ'_r with I_p



7.2.4.2 Coarse soils (and fine soils with $I_p < 25\%$)

In the case of coarse (granular) soils and fine (cohesive) soils with $I_p < 25\%$, shear box tests taken to large displacement or drained triaxial tests should be conducted until the post peak plateau is identified to obtain ϕ'_{cv} , c'_{cv} ; the values of ϕ'_{cv} from these tests are likely to represent conservative values for use in plane strain calculations. Alternatively, an estimate of the plane strain value of ϕ'_{cv} may be based on the plane strain values of ϕ'_{pk} and ψ measured in standard shear box tests, where ψ is the angle of dilation, using the relationship $\phi'_{cv} = \phi'_{pk} - 0.8\psi$ (Bolton [25]). Or the plane strain value of ϕ'_{cv} may be estimated from the angle of repose.

7.2.4.3 Fine soils (with $I_p \geq 25\%$)

For fine soils where displacements are likely to be small, and no pre-existing relic shear surfaces have been detected then it is appropriate to use design values based on ϕ'_{pk} , c'_{pk} in conjunction with the partial factors given in BS EN 1997-1:2004 and its National Annex.

The use of the critical state parameters in conjunction with the BS EN 1997-1:2004 partial factors is likely to lead to over-conservative designs for all soil types where displacements are small; however, the use of critical state parameters (ϕ'_{cv} , c'_{cv}) should be considered where significant displacements are likely to occur over the design life of the slope.

In the case of fine/cohesive plastic soil with $I_p \geq 25\%$, consideration should be given to whether residual strengths are likely to develop during the design lifetime of the slope. If relic shear surfaces are known to exist, or if sufficient displacement is likely to develop (or has already developed) such that shearing resistance will reduce (or has already reduced) to residual values along any given surface then the design values for the soil shearing resistance should be taken as the residual values. In these cases, large displacement shear box tests (either ring shear tests or repeated standard shear box tests) should be undertaken.

If significant displacement is likely to occur or the soil is brittle, the possibility of progressive failure should be carefully considered.

NOTE BS EN 1997-1:2004, 2.4.6.1 (Design values on actions) and 2.4.6.2 (Design values of geotechnical parameters) provide the option of assessing the design value directly or by derivation from the representative value by the application of a partial factor defined in Annex A. BS EN 1997-1:2004 states "if design values of geotechnical actions (parameters) are assessed directly, the values of the partial factors recommended in annex A should be used as a guide to the required level of safety".

7.2.5 Pore water pressures

The approach selected for pore water pressure monitoring should reflect the quality of data available and accuracy of results required (see BS EN 1997-1:2004, 4.5 regarding the minimum pore water pressure monitoring requirements for different categories of project). That is, the approach selected should be one of the following:

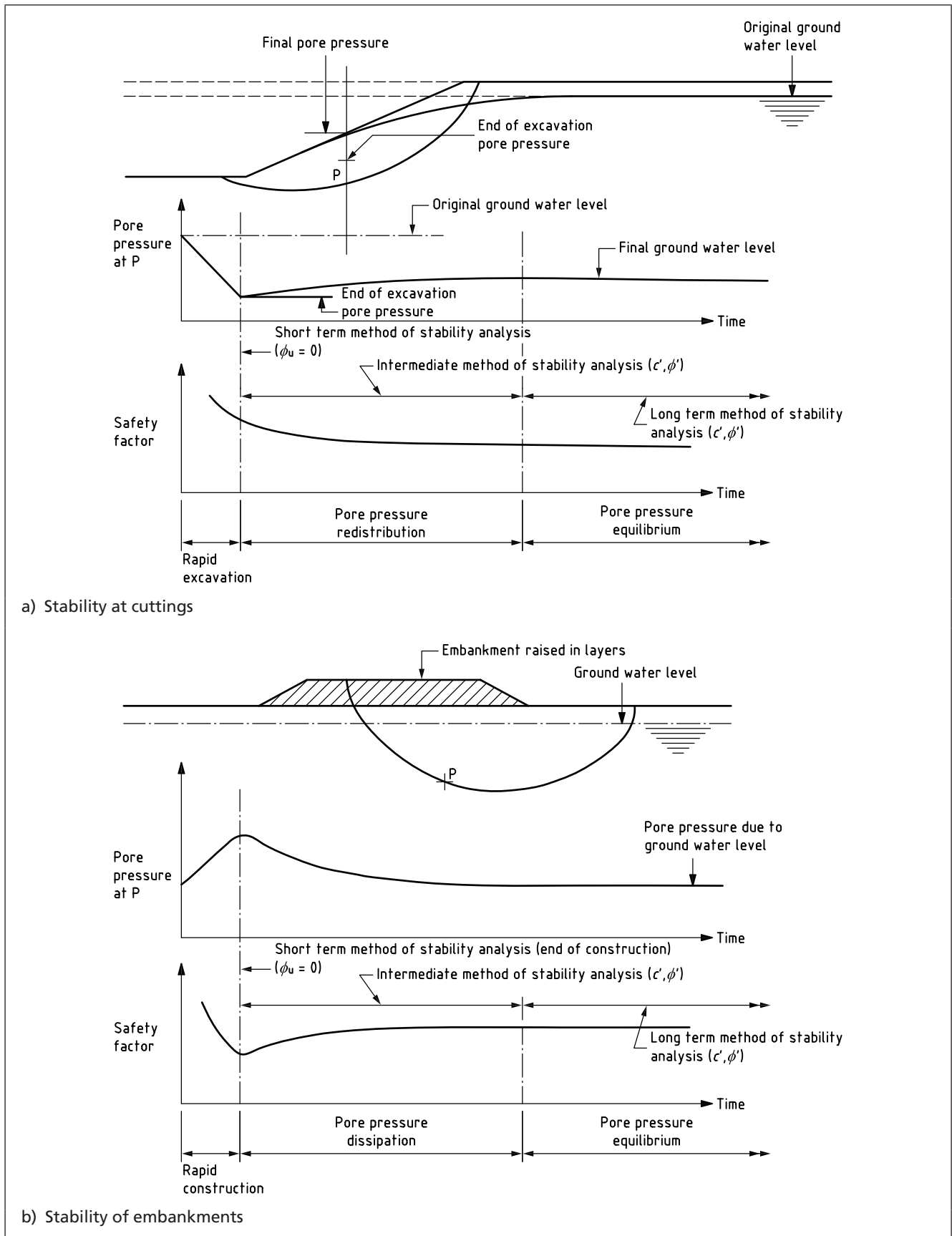
- ratio r_u – experienced based approach, only adequate for general indication of performance;
- defined groundwater table – overall generalized model determined from standpipe data, adequate for general design purposes; or
- detailed grid of pore water pressure values – piezometer monitoring required (with data on response time to specific events if appropriate), enables detailed modelling of pore water response to external influences.

The designer should assess which approach and what accuracy of data is required for the project. Having selected the approach to be adopted, conservative pore pressure values should be used in design (see BS EN 1997-1:2004, 11.3, and BS EN 1997-2:2007, 2.1.4 and 3.6).

The effects of vegetation should be considered when selecting design pore water pressures. The designer should consider time of year and proximity of instruments to trees when assessing the monitoring data.

NOTE Figure 7 shows how pore water pressures change with time after excavation of a cutting and after construction of an embankment.

Figure 7 Short and long term stability of embankment and cutting slopes



7.2.6 Local and overall stability

When preparing designs for the alignment and slopes of a cutting, the possibility of local slips or falls occurring on the face of the slopes should be considered, in addition to the overall stability against the various forms of failure described in Annex A. Local slips or falls can occur owing to the presence of random pockets of weak, unstable, or water-bearing soils, or thin layers of weak or shattered rocks; in most cases local instability may be dealt with as and when it becomes evident by adopting one or more of the remedial approaches described in 11.5. An overall flattening of the slopes due to the occurrence of these local failures may rarely be justified.

7.2.7 Modes of failure of slopes

There are a number of potential modes of failure of slopes and the designer should ensure that all relevant failure modes are considered (see Annex A).

7.2.8 Influence of construction procedure on slope stability

The designer should be aware that the following construction-related factors can influence slope stability:

- a) sequence and geometry of excavation, in particular temporary slopes and excavations should not be cut so steeply that ground movement is likely that would significantly reduce the stability of the permanent slope;
- b) effect of explosives, vibrations from blasting should be considered within the design;
- c) control of ground water, the potential for groundwater conditions during construction to have detrimental effect on the earthworks should be considered and where necessary appropriate measures shall be incorporated within the works; this might necessitate control of rate of excavation in a pervious water-bearing soil to achieve a gradual reduction in water table, or dewatering techniques to release porewater pressures trapped by low permeability strata;
- d) control of surface water, shaping the works to prevent water flow or ponding conditions where these are likely to have a detrimental effect on the earthworks; and
- e) construction of drain trenches at the base of the slope.

7.3 Design of slopes

7.3.1 Methods of design of soil slopes

The designer may use some or all of the following design methods:

- a) limit-equilibrium methods (see BS EN 1997-1:2004, 11.5.1, which requires horizontal interslice forces to be assumed unless horizontal equilibrium is checked; this excludes Janbu's original method and the Swedish circle [aka Fellenius] method, but allows, for example, Bishop's \bar{c}_1 simplified, Janbu's \bar{c}_1 modified and Sarma's methods);

- b) numerical methods (see BS EN 1997-1:2004, 2.4.1(12));
- c) physical modelling (see BS EN 1997-1:2004, 2.6);
- d) prescriptive measures (see BS EN 1997-1:2004, 2.5);
- e) observational method (see BS EN 1997-1:2004, 2.7 and CIRIA R185 [16]);
- f) stability charts;
- g) infinite slope method.

7.3.2 Methods of design of rock slopes

Unlike soil slopes, the design of rock slopes is dominated by discontinuities, and recognized references such as Hoek and Bray (in Wyllie and Mah [26]) and TRL [27] should be consulted.

The design should consider:

- a) the stability of the rock mass, which in most cases is governed by conditions in the joint system of the mass rather than by the strength of the intact rock; an assessment is required of the discontinuities within the rock mass, including any infilling;
- b) drainage requirements to manage groundwater, particularly where preferential groundwater flow is most likely (e.g. along soil/rock interface, discontinuities, permeable zones);
- c) local experience or exposures in similar strata;
- d) standard details required to deal with all the adverse conditions that can be reasonably anticipated (e.g. rock bolting, dentition work, drainage); and
- e) potential deterioration of the rock mass or discontinuities due to weathering effects during the design life of the excavated face.

In rock slope design it is particularly important that the designer should assess the ground conditions anticipated within an excavation (including potential unfavourable conditions), the proposed works best suited to deal with those conditions, and the form of inspection and design check as part of the works. For new rock cuttings, a trial excavation should usually be made to enable a check to be made of the design assumptions prior to cutting the face to the required finished position.

Weak, heavily weathered rocks can 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.

7.3.3 Factors of safety and partial factors

The verification of the overall stability of slopes should be carried out in accordance with BS EN 1997-1:2004.

The overall stability of slopes should be checked against DA1 Combination 2. For completeness, DA1 Combination 1 should also be checked if the designer considers that the loading applied to the slope (other than the mass of the ground in the slope) might control the failure mechanism rather than the ground strength parameters [see BS EN 1997-1:2004, 2.4.3.4.2(3)].

COMMENTARY ON 7.3.3

BS EN 1997-1:2004, 11.5.1 details the requirements for determining the overall stability of slopes and BS EN 1997-1:2004, 2.4.7.3.4 sets out the Design Approaches which are to be applied. The National Annex adopts Design Approach 1 (DA1), which requires verification that the limit state of rupture or excessive deformation will not occur with either of the following combinations of sets of partial factors.

Combination 1: A1 "+" M1 "+" R1

Combination 2: A2 "+" M2 "+" R1

Where "+" implies "to be combined with".

In Combination 1, partial factors in excess of unity are applied to unfavourable actions or the effects of actions whereas in Combination 2, the inverse of partial factors exceeding unity are applied to the soil parameters. This has the effect of increasing the effect of actions in Combination 1 and reducing the ground strength in Combination 2. The basic equations that govern are:

$$F_d = \gamma_F F_{rep}$$

$$X_d = \frac{X_k}{\gamma_M}$$

where

F_d is the design value of an action;

γ_F is the partial factor for that action;

F_{rep} is the representative value for that action;

X_d is the design value for a material property;

X_k is the characteristic value for that material property; and

γ_M is the partial factor for that material property.

The partial factors that should be applied to actions and to ground strength parameters are set by NA to BS EN 1997-1:2004, which are given in Table 4, Table 5 and Table 6. However, NA to BS EN 1997-1:2004 does not provide partial factors for actions for the specific situation of earthworks. In the absence of these, the values in Table 4 are recommended [based on the values for buildings given in NA to BS EN 1990:2002+A1, Table NA.A1.2 (A)]. Reference should be made to the current version of NA to BS EN 1997-1 to ensure the correct partial factors are used for design.

Table 4 Partial factors on actions or the effects of actions

| Action | | Symbol | Set | |
|-----------|--------------|------------|------|-----|
| | | | A1 | A2 |
| Permanent | Unfavourable | γ_G | 1.35 | 1.0 |
| | Favourable | | 1.0 | 1.0 |
| Variable | Unfavourable | γ_Q | 1.5 | 1.3 |
| | Favourable | | 0 | 0 |

Table 5 Partial factors for soil parameters

| Soil parameter | Symbol | Set | |
|--|------------------|-----|------|
| | | M1 | M2 |
| Angle of shearing resistance ^{A)} | $\gamma_{\phi'}$ | 1.0 | 1.25 |
| Effective cohesion | γ_c | 1.0 | 1.25 |
| Undrained shear strength | γ_{cu} | 1.0 | 1.4 |
| Unconfined strength | γ_{qu} | 1.0 | 1.4 |
| Weight density | γ_g | 1.0 | 1.0 |

^{A)} Factor applied to $\tan \phi'$ (see text of this clause for partial factor applied to residual angle of shearing resistance).

Table 6 Partial resistance factors for slopes and overall stability

| Resistance | Symbol | Set |
|------------------|----------------|-----|
| | | R1 |
| Earth resistance | $\gamma_{R,e}$ | 1.0 |

COMMENTARY ON 7.3.3 (continued)

Combination 1 involves applying partial factors to actions or the effects of actions whilst using unfactored values for the soil parameters and earth resistance. This approach is not usually relevant for checking the overall stability of a slope where earth is the main element providing resistance, since structural strengths do not provide resistance against overall stability failure and failure is controlled by uncertainty in the ground strength rather than uncertainty in the actions.

In addition the treatment of actions due to gravity, loads and water is difficult since these loads might be unfavourable in part of the sliding mass but favourable in another part. In a traditional analysis of a circular failure surface, part of the slope mass is producing a positive driving moment (i.e. it is unfavourable) and part of the slope mass is producing a negative driving moment (i.e. it is favourable) and the moments produced by the two parts depend on the position of the point about which moment equilibrium is checked. The application of different partial factors to each part of the slope introduces scope for confusion and requires a degree of complexity of analysis that is not readily available and not justified given the nature of the problem.

For this reason, a note to 2.4.2 of BS EN 1997-1:2004 states "Unfavourable (or destabilizing) and favourable (or stabilizing) permanent actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or to the sum of their effects." This note, commonly referred to as the "single-source principle", allows the same partial factor to be applied to stabilizing and destabilizing actions. When using Combination 1, it is recommended that the partial factor for the unfavourable action of the soil is applied to the weight density of the soil and the effect of this application can be summarized as follows.

- In an effective stress analysis, the effect of the partial factor is to increase the destabilizing action and to increase simultaneously the shearing resistance of the soil, which cancels the effect of the partial factor.*

- *In a total stress analysis, the increase in weight density increases the destabilizing action without increasing the shearing resistance of the soil. However, a higher partial factor is applied to the undrained strength in Combination 2 than to the permanent destabilizing action in Combination 1.*

In both cases, Combination 1 tends to be less critical than Combination 2 in almost all design situations. (Exceptions might occur when extremely large variable actions apply or the soil strength is extremely low.) Bond and Harris [28] discuss the way in which the single-source principle should be applied to slopes and embankments and show that Combination 2 results in an equivalent global factor of safety of about 1.25 for typical situations where an effective stress analysis is used.

If the single-source principle is not applied, then a special procedure has to be followed, if using commercially available software, in order to apply different factors to stabilizing and destabilizing actions. Frank et al [5] describe one such procedure, but by ignoring the single-source principle, Combination 1 becomes more critical than Combination 2 in most design situations using an effective stress analysis and results in an equivalent global factor of safety of about 1.35. However, Frank et al [5] recommend that Combination 2 normally be used for checking the overall stability of earthworks since the stability is governed by the shear strength of the soil rather than the application of the load of the earthworks.

Subclause 2.4.7.3.4.2 (3) of BS EN 1997-1:2004 states that, in circumstances where it is obvious that one of the two combinations governs the design, calculations for the other combination need not be carried out, but the designer needs to be sure that this is the case (e.g. based on past experience of similar designs). Therefore it is acceptable to base designs on Combination 2 alone (invoking the single-source principle) for many typical situations.

Where there is significant uncertainty about the density of the ground a sensitivity analysis should be undertaken [see BS EN 1997-1:2004, 11.5.1(12)].

Guidance on the use of advanced numerical methods in conjunction with the partial factors given in BS EN 1997-1:2004 is provided by Frank et al [5]; however, the designer should consider the relevance of such methods to the problem under consideration before embarking on advanced design since the overall stability of most routine slopes can be verified using limit-equilibrium methods.

The partial factors normally used for overall stability analyses may not be appropriate for slopes with pre-existing failure surfaces [BS EN 1997-1:2004, 11.5.1(8)], in which case the following approaches are relevant.

- Where the soil parameters for pre-existing failure surfaces are determined by back analysis partial factors of unity should be used for actions and the effects of actions, soil parameters and earth resistance since the objective in this case is to determine the value of the mobilized shear strength along the pre-existing failure surface.
- In the case where the residual strength of the soil is used for design purposes (whether determined from back analysis, laboratory or in-situ testing or from published data) Design Approach 1, Combination 2 is likely to govern the overall stability of the slope. BS EN 1997-1:2004, 11.5.1(8) states that partial factors normally used for overall stability need not be appropriate

for the analysis of existing failed slopes therefore lower values of the partial factors for ground strength parameters than those given in NA to BS EN 1997-1:2004 for Set M2 (i.e. the factors used in Design Approach 1 Combination 2) may be applied to residual strength. The partial factor used with the residual angle of shearing resistance should be chosen with due consideration to the confidence level of the data and the consequences of subsequent failure of the slope. Usually it should not be necessary for the partial factor applied to the residual angle of shearing resistance to exceed 1.1 provided the effective cohesion used in conjunction with that angle is set to zero.

For any slope where the consequences of slope failure are abnormally high the selection of characteristic values for the soil parameters should reflect the increased risk (see 7.4) in addition to other considerations listed in BS EN 1997-1:2004, 2.4.5.2(4) and a very cautious value might have to be chosen for the characteristic value. Alternatively, consideration should be given to increasing the partial factors on actions or the effects of actions and/or those for soil parameters.

NOTE The designer is referred to Frank et al [5], Bond and Harris [28] and CIRIA C641 [7] for examples of calculations and further guidance on design to EC7 principles. These references give worked examples of analysis by rotational, wedge and infinite slope methods, consider analysis by computer software or stability charts, and also identify some areas where differences can be expected relative to conventional global factor of safety methods of analysis.

7.3.4 Seismic effects

The designer should assess the potential seismicity of the region and, where appropriate, the requirements of BS EN 1998-5.

NOTE It is not normal to consider seismic effects for Category 1 and Category 2 structures in the UK.

7.4 Risks of failure and acceptance of deformation

7.4.1 General

The risks of failure should be considered under the following headings:

- a) movement due to failure of the ground in shear;
- b) unacceptable deformation before failure is reached;
- c) loss of service due to erosion or other external causes.

The designer should consider aspects such as the potential for uncertainty within the ground model that might increase the risk of failure, the consequences of failure, and the acceptability of deformation for the structures under consideration. The design output should identify these issues and how they have been addressed in the design.

Traditionally slopes were designed using a global factor of safety to reflect the risk and consequences of failure of a slope and this approach provided a clear and simple way of increasing or decreasing the factor of safety according to particular circumstances. Designs in accordance with BS EN 1997-1:2004 are carried out using partial factors applied to loads and materials that are described in NA to BS EN 1997-1:2004. However, the designer should ensure

that the risk of failure and consequences of failure have been adequately considered during the design. BS EN 1990:2002+A1 and [C1] BS EN 1997-1:2004 2.4.7.1(4) and 2.4.7.1(5) permit [C1] the variation of the relevant partial factors where the consequence of failure is either higher or lower than normal.

NOTE The traditional approach was for the designer to undertake additional design cases with unfactored parameters and use a global factor of safety considered appropriate for the uncertainties and consequences of failure of the slope.

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 (e.g. [C1] for drained analysis [C1] FoS of 1.4 rather than the commonly used target FoS of 1.3). A relatively low safety factor might 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 (e.g. FoS 1.1 or 1.2). Similar considerations apply to safety factors for embankments. One case of note is the stability of embankments during construction when founded on soft alluvium where the risk of unexpected failure is increased by the potential for porewater pressure migration along permeable laminations which is commonly mitigated by the use of a high global stability factor (e.g. [C1] for undrained analysis [C1] FoS of 1.5) or by using normal safety factors with a pessimistic distribution of water pressure [C1] in drained analysis [C1].

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

The designer should give careful consideration to mitigating the risk of overtopping or scour due to flood water. This may require adopting a risk-based approach in close co-operation with hydrologists to establish design flood levels.

7.4.2 Design life and serviceability

Design of earthworks should be undertaken to satisfy the requirements of BS EN 1997-1:2004, which requires that the ultimate and serviceability limit states should not be exceeded. Designers should also consider the durability of materials within the environmental conditions that will apply (BS EN 1997-1:2004, 2.3).

The concept of design life is not specifically addressed within BS EN 1997-1:2004, and reference should be made to BS EN 1990:2002+A1, which defines design life as "the assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary".

NOTE In the UK asset owners often specify requirements associated with design life or serviceability.

The designer should consider the issues that might influence ultimate and serviceability limit states of the earthworks in developing the proposed solution and determining an appropriate design methodology. BS EN 1997-1:2004, 12.6 requires the serviceability limit state design

to assess deformation; the asset owner's specification, or the intended purpose of the earthworks, should determine the criteria for allowable deformation. Seasonal movements due to the swelling and shrinkage of soils and movements due to external influences other than loading from the earthworks itself should be considered as part of the serviceability limit state design.

The concept of design life was developed for the design of structures to be constructed from materials that deteriorate with time, and hence the structure may be designed based on an assumed rate of deterioration; however, traditional earthworks are constructed from natural materials (soil or rock) that are not expected to deteriorate noticeably over the timescale of an engineering project (which is short in geological terms). Deterioration of natural earthworks materials should be avoided during the life of the works by:

- choice of materials acceptable for each class of fill and compaction control during construction, i.e. by the earthworks specification; for example selected granular fill at structures should not include argillaceous rock which will not be durable in such a setting, the rate of deterioration can be rapid but is difficult to predict and hence not suited to the design life concept;
- the maintenance regime adopted, particularly of drainage, vegetation and the activity of others using the earthwork (both human and animal);

consequently it is not usually appropriate to design earthworks of natural material that will fail a limit state after a defined period of time.

COMMENTARY ON 7.4.2

This situation changes when earthworks includes artificial materials such as gabions, geosynthetics, soil nails, embedded piles or structures that form part of the earthwork solution. For these combined forms of earthwork the structural element can be designed to a design life to reflect the durability of the engineered elements. However, the significance of the element of the works to the overall stability of the earthworks may vary, e.g. a geotextile separator is likely to have a less direct impact on stability than oversteepened earthworks constructed of reinforced earth.

For highways schemes a common approach has been to assume a design life of 60 years for earthworks which are considered as any form of slope less than 70° slope face angle and can incorporate steeper minor earth retaining structures of up to 1.5 m vertical retained height (compared to the conventional 120 years for a structure). However, assessing the relative contribution of an element of the earthworks is often difficult and can distort the decision making process if simple rules are applied. It is important that the designer considers how the earthworks will behave, consequences of failure and the design life of other elements of the scheme; for example, for a soil nailed slope of 60° face angle adjacent to a major highway, the corrosion protection of the soil nails would have to be assessed giving consideration to the potential for differential corrosion at the particular site if that could lead to progressive failure of the system.

An employer's requirement for "maintenance free earthworks" is usually unrealistic, since in most cases it is likely to be more meaningful to specify a requirement for "the design and construction of the works to be completed to achieve serviceable status of the earthworks to a design life of 60 years and be major maintenance free for the first 25 years."

7.4.3 Effect on neighbouring structures

COMMENTARY ON 7.4.3

Buildings close to embankments and cuttings can 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 can damage buildings or buried services. Upward soil movements beneath a deep basement excavation can cause damage to adjacent structures or to 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.

Where 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, the calculations to determine the serviceability limit state should be made by conventional methods applicable to structures but based on data obtained from predicted ground deformations.

7.4.4 Impact on existing slopes

In cases where earthworks will be constructed in the vicinity of existing slopes either in the form of natural slopes or earthworks, the designer should assess the potential impacts on those existing slopes.

Examples can include widening existing earthworks, forming earthworks on an existing slope, or undertaking earthworks in close proximity of other slopes. In all these cases the earthworks activities can change the loading on the existing slope or modify the surface water and groundwater flow paths, both of which could be detrimental to the stability of the existing slope and therefore require consideration within the design. Where the existing slope is identified as being of poor stability then the design of the new earthworks will require special consideration to ensure that the resulting structure is adequately stable. Other related information is provided in **7.6.11** regarding embankments on sloping ground and Clause **11** on earthworks asset management and maintenance.

7.4.5 Stabilization of existing unstable slopes

Where works are planned to stabilize an existing unstable slope (either a natural or an earthwork slope) as well as satisfying the requirements of this standard (including the information at Clause **11**), the design should require each aspect to be considered in greater detail since the construction works could exacerbate existing problems or create new problems on a slope of marginal stability.

The desk study, ground investigation and monitoring need to be planned to enable the problems of the existing slope to be adequately understood. Where possible the design should give consideration to other previous schemes that have been successful in similar ground conditions.

On existing landslips (especially in fine grained soils) it is preferable to determine both the groundwater profile and the rate of deformation in order to understand how these factors are linked to seasonal weather

conditions, and assess the likelihood of a significant acceleration in movement if conditions deteriorate. Drainage is normally an important component of slope stabilisation works; however, these should be designed in recognition of ongoing slope movements as these can damage drainage runs leading to acceleration of the movements.

Understanding of the deformation history and the stress state of the soils will be particularly important in soil types where a significant reduction in shearing resistance can be expected (see 7.2.4) and design with residual parameters may be appropriate. It is often advantageous to plan the remedial works to achieve a gradual improvement in stability to reduce the risk of accelerated deformation during construction.

For major landslips, the design should include an element of risk management since complete stabilization under all conditions might not be realistic.

There are publications on the design of slope stabilization measures which should be referred to (e.g. Bromhead [22]); however, in this situation the previous experience of the design team will be particularly important.

7.5 Earthworks drainage systems

7.5.1 Pre-earthworks drainage

The earthworks designer should assess the requirement for pre-earthworks drainage.

In cuttings there is likely to be a requirement for drainage in the form of a v-ditch (or where insufficient space is available, a filter drain) along the crest of the cutting to intercept surface water liable to flow towards the cutting. For the crest drainage to be effective the system needs to be able to discharge to a suitable outfall in order to prevent water overtopping the drain and subsequently damaging the cutting slopes. The pre-earthworks drainage design should include consideration of:

- location and form of crest drains;
- requirements for lining of drains to prevent infiltration from the base where this might lead to slope instability or groundwater pollution (the effectiveness of lining systems is often limited and sediment within v-ditches will form a natural seal so in many soil types establishment of vegetation may be the most appropriate form of seal);
- requirements for scour protection on steeply inclined drains; and
- details for the interception of field drains that are present within the footprint of the earthworks.

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

In the case where a new culvert is provided, its size, gradient and invert levels should be agreed with the appropriate water authority or agency to ensure that possible run-off from future areas of

development can be accommodated and any future re-grading of the watercourse can be carried out both upstream and downstream of the embankment crossing.

NOTE 1 Where it is necessary to provide a pipe under the embankment, it is always prudent to provide one of sufficient size to permit blockages to be cleared by working from the ends of the pipe.

NOTE 2 To avoid damage by earthworks construction plant to pipes laid at existing ground level or at shallow depths, it might be necessary to protect the pipes by means of a concrete surround or by other methods.

7.5.2 Drainage during construction

Adequate temporary drainage should be provided during the construction of earthworks. The assessment of the drainage measures required is generally the responsibility of the contractor undertaking the earthworks who should use their skill and experience to ensure the temporary drainage provided is adequate to ensure the success of the earthworks by maximizing the suitability of excavated material and minimizing the potential for deterioration of materials or instability of the works. When temporary drainage issues of particular note for the scheme are identified during the design of the permanent earthworks and drainage these should be recorded so that the earthworks contractor can make adequate provision.

COMMENTARY ON 7.5.2

Issues of note that can be addressed by temporary drainage during construction include:

- *provision of adequate permanent or temporary approved outfalls during the works;*
- *use of measures to reduce flow rates and remove silt from earthworks run-off drainage;*
- *an earthworks methodology that allows for temporary fill surfaces to be sealed and shaped to shed water and operational restrictions during periods of rainfall;*
- *installation of temporary v-ditches in cuttings as the works are progressed improves the stability of side slopes and working surfaces in silts and sands below the groundwater table;*
- *planning the works so that cuttings in permeable soils and high groundwater table are excavated so as to gradually lower the groundwater table and maximize suitability of excavated earthworks materials;*
- *there can be significant benefits of advance earthworks activities in some ground conditions, such as installing filter drains in advance of trimming to formation to draw down the groundwater table to avoid softening of the formation, or installation of well points in inter-layered silts/fine sands and clays); and*
- *during earthworks construction, care has to be taken to avoid blocking permanent filter drainage with silt from surface water run-off.*

7.5.3 Embankment under-drainage

In certain situations the designer should ensure that lower levels of the embankment are relatively highly permeable, for instance an infrastructure carrying embankment in a flood plain. A common form of this may be the inclusion of a granular starter layer (see Table 8) to aid construction and accelerate consolidation. In other situations,

e.g. flood defence, the inclusion of such an arrangement would be inappropriate. Under-drainage may also be required where band drains are being used to accelerate consolidation or to mitigate uplift.

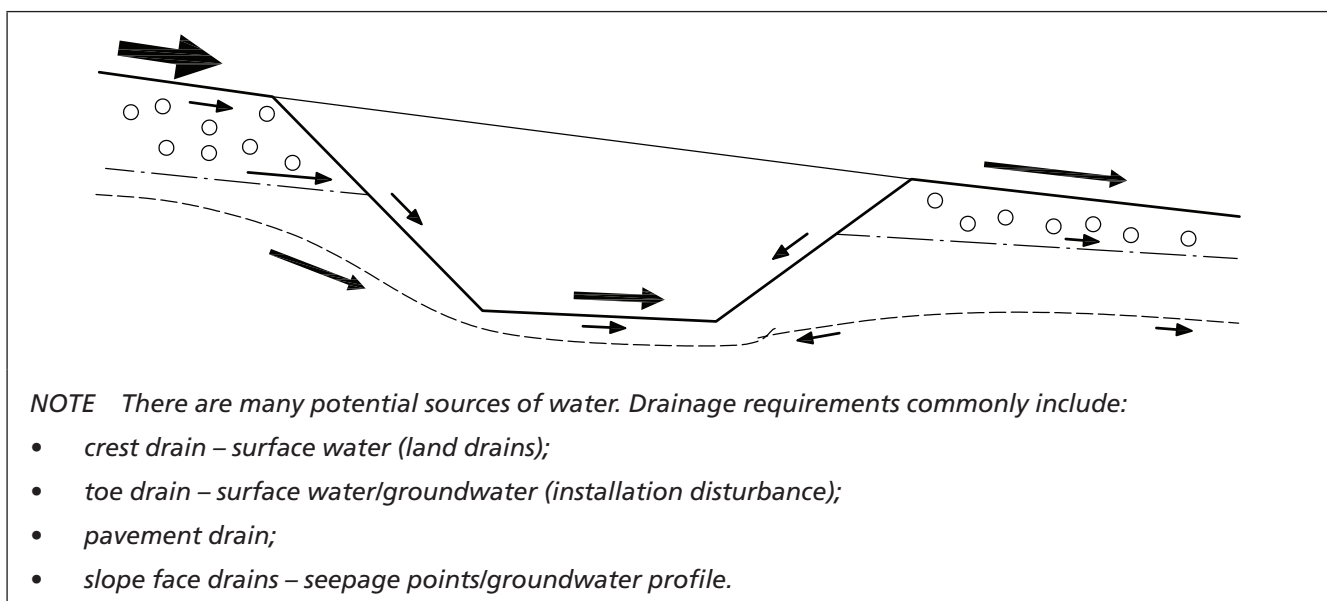
7.5.4 Earthworks drainage requirement

The earthworks designer should liaise with the drainage design team to establish suitable site constraints and practical limitations, that is:

- define the objective of the drainage system;
- assess the catchments (surface water and groundwater);
- determine the outfalls (site and environmental constraints – EA agreements);
- observe/conceptualize the existing flow paths;
- conceptualize future post earthworks flow paths;
- design to capture significant flows (see Figure 8);
- consider the likely construction and maintenance regime;
- review the consequences of system failure (may justify overdesign to avoid unacceptable flood risk);
- review potential knock on effect on adjacent users; and
- size the ditches and pipes (use simple systems wherever possible), see 7.5.5.

Where drainage is proposed as a remedial technique for the stabilization of unstable slopes the earthworks designer should give particular consideration to the most appropriate forms of drainage to suit the site conditions. The use of flexible and open drainage systems can often prove advantageous. Particular attention should be given to the consequences of system failure, in particular whether on sections of slope liable to significant movement the risk of drainage failure might exceed the potential benefit. Reference should be made to publications such as CIRIA C591 [29] for general cases, and Bromhead [22], and published papers for advanced techniques in differing ground conditions.

Figure 8 Design of earthworks drainage to capture significant flows



COMMENTARY ON 7.5.4

Guidance in relation to the provision of earthworks drainage is contained in Carder et al [30].

Guidance on the design of soakaways is provided in BRE Digest 365 [31].

7.5.5 Hydraulic design

NOTE Guidance on the design of pipe and open channel sizes is widely available in civil engineering texts (e.g. BS EN 752-4) and does not need to be repeated here.

The drainage designer should make a realistic and adequate assessment of:

- design return period;
- catchment size and type;
- run-off rate/time of entry (use of the *Flood Estimation Handbook* [32] for assessment);
- maintenance issues (e.g. scour erosion of open channels); and
- climate change.

7.5.6 Sustainable drainage (SUDS)**COMMENTARY ON 7.5.6**

Surface water drainage systems developed in line with the ideals of sustainable development are collectively referred to as sustainable drainage systems (SUDS). At a particular site, these systems are designed both to manage the environmental risks resulting from urban runoff and to contribute wherever possible to environmental enhancement. SUDS objectives are, therefore, to minimize the impacts from the development on the quantity and quality of the run-off, and maximize amenity and biodiversity opportunities. The philosophy of SUDS is to replicate, as closely as possible, the natural drainage from a site before development. There are various documents on the subject of sustainable drainage; a good starting point is CIRIA C697 [33], which provides best practice guidance on the planning, design, construction, operation and maintenance of SUDS to facilitate their effective implementation within developments.

From an earthwork perspective the most notable aspect of SUDS is to provide permeable surfaces that encourage surface water infiltration into the ground (rather than runoff from impermeable surfacing such as tarmac).

SUDS should be considered in all planning applications.

Whilst SUDS design is outside the scope of this standard, the attention of the earthworks designer should be drawn to the potentially deleterious effects that poorly planned SUDS can have on either pre-existing or newly designed earthworks, and, as far as local developments are concerned, SUDS are being included as the main form of drainage in the majority of projects being designed these days.

The earthworks designer should encourage use of SUDS to reduce the volume of surface water runoff, but consider the potential impact of SUDS on earthworks stability, i.e. aim for infiltration areas away from at risk areas to gain the benefit of SUDS and avoid additional risk.

7.6 Embankments and filled areas

7.6.1 Design of embankments and filled areas

7.6.1.1 General

The overall stability of embankments should be designed in accordance with the requirements of BS EN 1997-1:2004, 12.5 and the deformation of the embankment or filled area should satisfy the requirements of BS EN 1997-1:2004, 12.6.

Embankments and filled areas should be designed to have adequate stability against shear failure and to ensure that any deformation is within acceptable limits.

NOTE The information required before the cross section of the embankment can be designed includes:

- a) ultimate width of top of embankment;
- b) loading on top of embankment;
- c) geotechnical properties of the foundation and fill materials;
- d) restrictions on width of land available;
- e) special conditions to which the embankment would be subject, for example, tidal waters, active mining operation and natural cavities, and environmental and other economic factors which could influence the final choice of cross section, e.g. earth banks for sound screening or flattening of slopes to allow them to be returned to agriculture.

7.6.1.2 Stability

Calculation of the stability of the embankment should be undertaken using the methods of analysis described in 7.3.1. In some instances, it may be desirable to analyse embankment deformations using, for example, finite element methods to determine whether deformation is acceptable.

COMMENTARY ON 7.6.1.2

Parameters of the shear strength of the fill appropriate for use in the stability calculations are usually obtained from laboratory tests on recompacted samples. Where an embankment is built of rockfill or other granular material with side slopes not exceeding the angle of repose of the fill, it is inherently stable for all heights as long as the foundations are capable of sustaining the loads. However, the angle of shearing resistance of a well compacted granular fill can be considerably greater than the angle of repose and consequently the laboratory determination of this parameter and its use in the stability calculations can lead to a more economic embankment cross section. For rockfill embankments, where laboratory determination of the angle of shearing resistance of the fill material might be difficult, reference can be made to the approach set out at BS 8002:1994, Table 3 and Table 4. The shear strength and pore pressure parameters of clays and silts can be measured in laboratory triaxial compression tests. If the natural moisture content of the material in the field is high but the permeability characteristics are such that it can be readily reduced, the design could take advantage of the resulting improvement in shear strength.

Where embankments are constructed on sidelong ground and a layer of impermeable material underlies a significant thickness of permeable material, a perched water table can form, causing saturation of the coarser material with possible erosion or slumping where the water table emerges onto the side slope. The stability of an embankment depends not only upon the strength of the fill material from which it has

been formed but also upon the strength of the material on which it is founded. An assessment is necessary to check the ability of the foundation material to carry the required superimposed load without shear failure or unacceptable deformations. The factors governing the behaviour of soils and rocks in cuttings generally apply also to their behaviour as foundation materials for embankments. If the site contains geological features such as faults or slip surfaces resulting from previous movements, due regard has to be taken of these during the evaluation of the stability of the embankment. Techniques are available for improving the strength properties of fill and foundation materials. The effects of embankment loading on materials of low shear strength can be mitigated by various methods including, excavation and replacement, staged construction, the use of berms, or flattening the side slopes, use of geosynthetics, or undertaking ground improvement prior to construction.

7.6.2 Materials

The characteristics of the materials given in Table 7 should be taken into consideration in their use for foundations and embankments.

The strength, deformation and moisture susceptibility of foundation and fill material should be established by means of:

- a) in situ testing as part of site investigation;
- b) laboratory tests;
- c) instrumented field trials;
- d) information from previous performance.

In the case of soft ground engineering or rock embankments, field trials should be considered in order to determine the best procedures both for excavation and for forming a satisfactory embankment.

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

The selection of parameters for design should take account of both the soil grading and, in fine grained soils, the nature of the fines content, which is commonly defined by the soil plasticity.

COMMENTARY ON 7.6.2

In the field of earthworks there are differences in the fines content required for a soil to be classified as fine grained (cohesive) which reflects the different situation under consideration:

- *for geotechnical design (e.g. slope stability or settlement analysis) a soil with more than 35% fines is normally considered to behave as a “fine grained” soil (although it is important to realise that some soils with lower fines contents can still behave as a fine soil);*
- *for classification of fill materials the change point occurs at 15% fines (above which the fill is defined as cohesive under the SHW [1]), this reflects the tendency for the materials to trap excess porewater pressures during compaction; pavement foundation layer designs follow a similar approach and designers normally aim for the granular soil to be “non-plastic” as defined by plasticity testing.*

See Table 1a), Table 1b) and Table 1c) for information on soil descriptors in different earthworks circumstances. In practice the point at which the materials behaviour changes from granular to cohesive can vary resulting in what may be best defined as “intermediate soils” as illustrated in Table 1c).

Table 7 Typical characteristics of foundation and embankment fill materials

| Material type | Benefits | Disbenefits | Comments |
|---|--|---|--|
| Rock | Excellent strength and deformation properties. | Weak rocks (e.g. mudstones, shales, slates, marl and chalk) can degrade if exposed to the elements or inappropriate construction methods are used. | Rockfill placed below water or used as a permeable structure (drain) should be strong and durable. Final layers of rockfill may need 'blinding' with finer material. |
| Gravel and sand | High permeability; Resist development of excess pore water pressures; Instantaneous consolidation ; Instantaneous development of strength. | Saturated or loosely packed fine sands can develop 'quick' conditions when subjected to vibration. | Uniformly graded fine sands require tight moisture control for compaction. Final layers of uniformly graded granular materials may need "blinding" with appropriate material. |
| Clay | Low permeability (for construction of water retaining structures etc.) | Excess pore water pressures can develop during construction; Suitability for use reliant on natural moisture content; Long term post-construction consolidation. | Strength and deformation characteristics for fill and foundations primarily a function of moisture content; Foundations influenced by structure and fabric of soil derived from geological history (e.g. overconsolidated clays). |
| Silt | Intermediate permeability between clay and sand, stability can be maintained by drainage. | Strength and deformation behaviour very susceptible to instability caused by disturbance and seepage / high porewater pressures. | Cohesive soils with characteristics intermediate between clays and sands. |
| Mixed soils (clay, sand and gravel) | Soil strength can be reasonable if groundwater is managed. | Soils of this type (such as glacial till) can be highly moisture susceptible. | Properties generally determined by the predominate soil type but consideration must be given to secondary constituents as the soil can behave as either a granular or cohesive soil in different situations (see text above). Require careful choice of laboratory testing regime. |
| Interlayered high and low permeability soils | Permeable layers can be utilized as a drainage path if works are designed appropriately. | Seepage paths provided which can prove difficult to drain in cuttings. Porewater pressures within permeable layers can be unable to dissipate due to presence of clay layers causing loss of strength. Porewater pressures under embankments can be transmitted along silt and sand laminations and beds triggering instability at slope toe. | Special attention required to earthworks drainage design. |
| Peat | Highly compressible; Difficult to determine design parameters which can be very variable. Unsuitable as foundation or fill. Secondary consolidation can be ongoing over many years resulting in very large settlements. | | Where removal is impracticable within foundations, accelerated consolidation through drainage and/or surcharging should be considered. |
| Chalk | Managed correctly, chalk is a good fill. | Highly weather susceptible. Requires careful selection of compaction plant. | Clause 604 of SHW [1] relates to the handling of chalk. Additionally, the CIRIA report C574 [34] provides much information, both geotechnical and on the practical handling of chalk. This includes advice on the "chalk season", how this, traditionally the end of March to the beginning of November, may be relaxed depending on both the structure of the chalk and local weather conditions. Designers should not overly constrain this season. |
| Secondary and recycled materials | Enables sustainable development; Wide range of materials and sources available; Potential for excellent construction fill materials. | Potential contamination; Untreated putrescible/domestic waste unsuitable for fill or foundations; Continuity of source material. | Thorough investigation and suitability of sources for chemical, toxicity, aggressivity, combustibility and mechanical properties; Consequential pollution should be considered. Protocols exist for advice on testing regimes and frequencies, e.g. WRAP. |

7.6.3 Settlement of filled areas

7.6.3.1 General

COMMENTARY ON 7.6.3.1

Settlement of embankments and filled areas can occur as a result of any or all of the following forms of load which are divided into "internal" and "external" forms of loading in the following sections:

- *settlement of ground below the fill;*
- *self weight settlement of the fill itself,*
- *settlement due to load changes as a consequence of change in groundwater conditions (e.g. inundation settlement);*
- *settlement of the fill due to loads placed upon the fill (e.g. by structures).*

Additional movement can occur due to seasonal changes in moisture content of cohesive soils. This is not a form of settlement (since the movement is recoverable), however the additional movement can be incorrectly interpreted as being part of the settlement.

There are likely to be significant differences in the forms of settlement affecting different earthworks formed of fill as a result of the morphology of the fill body itself, i.e. differences between embankments, fill platforms (extensive areas of fill placed largely above adjacent ground level) and infilled hollows (such as old quarries).

Some deformation of the fill, of the foundation materials or of both can occur and the behaviour of the materials involved should be studied at the site investigation stage to determine their settlement characteristics.

The acceptable degree of settlement depends on the type of function the embankment is required to serve, e.g. to carry a highway or railway or for building developments. In some cases, the major part of the settlement should be induced before the filled area is required to be used. In the case of embankments this may often be achieved by completing the fill early in the contract and topping up as necessary during the completion stage, or by surcharging the fill by increasing the height to accelerate the settlement, the excess material being removed before completion. In the case of filled areas the problem is often more difficult to resolve (partly due to the far greater risk of groundwater inundation settlement risk) and the approach to management of the earthworks should be carefully considered.

7.6.3.2 Settlement of fill due to internal loading

The designer should consider that: where a significant thickness of fill is placed over a wide area the load due to the fill can often result in substantial settlement, which is a factor that should be considered in the design of all earthworks. In many cases the finished surface of the earthwork can have adequate bearing capacity, but the designer should consider that settlement due to causes not connected with the weight of the infrastructure or structures placed on those earthworks (e.g. low-rise buildings, pipelines, pavement) can have a serious effect on those features. A variety of potential causes of settlement of the fill that should be considered can occur and can be described as "internal loading" effects, which can be categorized as:

- a) settlement of underlying ground due to the weight of the fill;
- b) settlement of the engineered fill due to self-weight;

- c) movement in the engineered fill due to post-construction changes in ground-water level or downward percolation of water;
- d) movement in the engineered fill due to seasonal changes in moisture content or pore pressures.

Settlement of the underlying ground is an important issue for earthworks, given the large footprint of the body of fill and, as is traditional, should be calculated by standard methods described in soil mechanics texts. In the case of embankments, consideration should be given to the trapezoidal shape of the fill body and the designer should consider the potential extent of the settlement bowl which might extend beyond the footprint of the earthworks; it is often the case for structures constructed on fill that the load from the fill can have a greater influence on the underlying ground than the load from the structure.

The SHW [1] has been developed and should be used for infrastructure embankments which are generally of limited height (i.e. up to 15 m), where the trapezoidal shape of the body of fill and nature of construction enables dissipation of a large proportion of excess porewater pressure during construction provided the specification is followed. Consequently, it should be borne in mind that for these embankments self-weight settlement is not usually a major controlling factor in determining the design other than at locations where differential settlement is a concern (e.g. at the approach to an underbridge).

For conventional infrastructure embankments, it is normal practice to adopt an experience based approach of assuming 0.5% to 1% of embankment height as self-weight settlement (an approximation typical of clay fills compacted wet of optimum moisture content). In addition it may be necessary to include a monitored hold period once the earthworks reach full height (or are surcharged at a higher level) and prior to completion of any sensitive elements of the infrastructure upon the earthworks. Where the magnitude of the settlement needs to be calculated, this may be done based on the equations presented by Trenter [35]:

$$\text{Total self-weight settlement} = 0.5(\text{bulk unit weight} \times H^2)/D$$

where

H is embankment height

D is the constrained modulus, which can be approximated as $1/m_v$ or can be based on values obtained from large diameter oedometer testing or site monitoring as summarized by Trenter [35].

If an accurate calculation of self weight settlement is required then there are a number of factors that should be considered regarding the nature of the fill and compaction conditions, as presented by Charles and Skinner [36]. It should be noted that this is particularly appropriate for large bodies of deep fill that are intended to support structures, or if the adequacy of compaction is in doubt, or if cohesive fill dry of optimum moisture content and with high air voids is used.

Where the earthworks are to be undertaken for a purpose other than a typical infrastructure embankment the designer should consider the issue of self weight settlement in greater detail. Settlement of the body of fill itself can be a significant issue for large areas of deep fill placed to enable the construction of buildings, and research by the BRE should be referred to [37].

Collapse compression on inundation with water is a hazard for engineered fill with a high air-voids content, particularly where an excavation is backfilled below the likely long term groundwater level; for these cases the designer should evaluate the likelihood of occurrence and establish whether particular measures are required within the design. Collapse on inundation is a particular hazard for structures built on fill and the specification of the fill material properties and compaction of the fill should be designed to eliminate collapse potential.

Ground movements within the fill due to a rise in overall water table within the fill are also a concern for earthworks built in areas prone to flooding; for all earthworks, designers should consider whether there are areas where concentrated infiltration of surface water can be expected, and whether this could be sufficient to generate ground movements. In many earthworks situations water pipelines will be present and leaks from these pipes commonly result in ground movements, which is difficult to fully mitigate against; however, the designer may include measures to ensure that settlement has completed prior to installation of the pipe runs, and consider whether there are any locations where a leaking pipe could lead to a major failure.

It is important to note that simply setting a compaction requirement that field dry density is at least 95% of the maximum dry density obtained from the standard Proctor (2.5 kg) compaction test will not necessarily eliminate collapse potential in many fills and consequently this should not be assumed to be an adequate compaction specification. Where there is a strict requirement to limit settlement within the body of the fill the designer should assess whether general fill in accordance with the SHW [1] is adequate, and if not then either use selected granular fill, or adopt a more stringent specification requirements as described at BRE Digest 427 [37] (see 7.6.4 and 8.4).

The designer should give consideration to the potential influence on the earthworks (or infrastructure/structures on those earthworks) of ground movements associated with seasonal changes in moisture content. The main concerns are where these seasonal changes are accentuated by vegetation resulting in significant shrinkage of clay fills (see 7.10.1 and 11.7.3), and this may necessitate restrictions on where high-plasticity cohesive fill can be placed within embankments.

7.6.3.3 Settlement of fill due to external influences

COMMENTARY ON 7.6.3.3

Settlement of the fill due to imposed loads can be as a result of:

- *dead load - i.e., structures;*
- *live load - i.e., traffic; and*
- *collapse of underlying ground (see 7.2.2).*

The designer should assess the fill material and compaction requirements to limit settlement to an acceptable magnitude (both settlement of the fill and due to imposed load), which might require greater compactive effort than SHW [1].

Fill which is to form a foundation for buildings should undergo particularly strict quality control, and the specification should be prepared with this in mind (see Clause 8). From a technical standpoint, a period of monitoring following completion of filling and prior to construction of structures is prudent.

The designer should consider whether a surcharge due to traffic or similar live loads needs to be considered in settlement assessments since loading of this type is normally transient and of short duration. Traffic loads are normally local to the surface of the earthwork and usually contribute only a small proportion of the total earthwork load thus they may generally be ignored in settlement assessments unless there is a particular reason to take account of short term transient loads.

7.6.4 Selection of material properties for earthworks fill

An important activity for every earthworks project is the selection of material properties for the fill; this is considered to be a design activity regardless of whether it is undertaken by a contractor or a consultant. The material properties should be chosen to ensure that the engineering design assumptions are satisfied as well as addressing construction practicalities.

The material properties for earthworks fill should be selected to ensure that:

- the material can be trafficked, placed and compacted during construction of the earthworks;
- the earthworks will be stable during and after construction;
- excessive settlement or heave will not take place.

For the majority of fill materials the acceptable material properties should be related to limits applied to either moisture content, MCV or shear strength e.g. see Table 6/1. It is strongly recommended that only one of these properties is used for a particular acceptability limit.

For most coarse soils the upper and lower acceptability limits should be selected by reference to a particular ratio of dry density to the maximum dry density. The values are determined from dry density/moisture content relationship tests, which are illustrated in general terms in Figure 9. The most commonly adopted criteria are 95% of the maximum dry density determined from the 2.5 kg light dynamic compaction test or 90% of the maximum dry density determined from the vibrating hammer test for bulk earthworks fill. A higher value up to 100% of the maximum dry density is required for fill that will support structures where settlement is more critical. It is recommended that the air voids content at the proposed lower acceptability limit is checked to ensure that excessive air voids will not remain within the fill at the chosen compaction ratio; however, an air void content less than 10% may not be feasible with some uniformly graded coarse soils.

It is important to note that the maximum dry density and optimum moisture content are not fundamental soil properties and the values are dependent on the compactive effort imparted to the material.

For fine soils the upper acceptability limit (see Figure 10) e.g. minimum MCV, should be chosen in relation to the requirements for placement of the fill, stability of slopes, and settlement of the fill due to internal loading (see 7.6.3). These requirements may vary for different end uses of the earthworks, which will determine the fill properties of greatest importance, e.g. permeability for a flood bund, or in-situ density for structural fill. The lower acceptability limit (minimum moisture content, maximum MCV or maximum shear strength) should be selected to reduce the air voids in the material to a value that will restrict the potential for excessive movement after compaction. A maximum of 10%

air voids for bulk earthworks fill and 5% air voids for earthworks fill that is to support structures are commonly specified values. Research at TRL led to the development of the compaction requirements of Table 6/4 of the SHW [1] which are intended to achieve these values for the relevant classes of fill provided that the moisture content of the material is appropriate. Further guidance on the degree of compaction achieved using the methods specified in Table 6/4 is provided in HA44/91.

When there are specific requirements to limit the internal settlement for large bodies of fill that will carry structures (as described at 7.6.3) then the approach of selection of design parameters beyond that which would normally be considered under the SHW [1] approach may be developed. One methodology that may be used is proposed in BRE Digest 427 [37], whereby:

- the moisture content upper and lower acceptability limits of the fill are chosen based on OMC from both the standard Proctor (2.5 kg rammer) and the modified Proctor (4.5 kg rammer) compaction tests (i.e. relatively dry material for fine soils), see Trenter [35] for further details;
- and the method of compaction is selected to ensure heavy compaction is delivered (which is likely to be in excess of the SHW standard methods); and
- the earthworks are monitored to ensure a high in-situ density and low air voids are achieved.

It should be noted that a fine soil that is at Point A on Figure 10 will not benefit from further compaction and the strength could be reduced due to the generation of excess porewater pressure if further compactive effort is applied. Excess porewater pressures weaken the fill layers affected, which limits the effectiveness of compaction of subsequent layers of fill on fill; therefore a pause of a few days should be accommodated to allow dissipation prior to recommencing earthworks. By contrast the dry density of a soil at Point B should increase if additional compactive effort is applied.

NOTE Fills with a significant proportion of coarse particles represent a problem for determination of acceptability criteria, as these soils often prove inappropriate for either laboratory testing or in-situ density testing. BS 1377-4:1990 sets an upper limit of 10% of particles coarser than 37.5 mm and 30% coarser than 20 mm above which standard laboratory compaction tests are not applicable since the fill is classified as being "Grading Zone X". However, if the Zone X criteria are strictly applied, then many UK materials used as fill are classified as untestable by virtue of a relatively low granular content (e.g. well-graded glacial till). This is actually detrimental to the management of the earthworks project. Trenter [35] provides methods for adjustment of the results to allow for the influence of coarse fraction.

An experience based approach is recommended for these coarse soils to determine the most appropriate method for testing and management of the fill. For gap graded or well graded fills (granular or cohesive) the earthworks engineer may judge that there is a matrix of testable material that will strongly influence the performance of the fill. In many cases it may be appropriate to remove coarse particles to facilitate laboratory testing and base the acceptability criteria on the finer fraction of material.

Acceptability criteria based on moisture content may be used for very coarse granular fills, such as Class 1C and Class 6B of the SHW [1]. Compaction using Method 5 of SHW Table 6/4 may provide a general

approach but performance should be reviewed on site. The construction and analysis of trial embankments should be used to provide definitive site and source specific guidance for compaction of very coarse fills.

The above is a limited summary only; designers of earthworks should have an awareness of the various issues that might influence the fill material that they will utilize.

COMMENTARY ON 7.6.4

It is useful for the earthworks engineer to have an understanding of both the underlying principles of fill material behaviour and the development history of earthwork engineering. The latter is important since earthworks is not a well defined science, and to resolve certain practical difficulties the standard approaches draw upon previous work. Of particular importance in the development of the subject is the testing undertaken by the Transport Research Laboratory to develop the method specification that is included with the SHW [1], details of the TRL research were recorded by Parsons [38]. Field trials by the Building Research Establishment showed the importance of control of air voids content of fill materials incorporated in earthworks for future building development (e.g. BRE Digest 427 [37], Charles et al [39]).

Informative descriptions of the history and principles that underlie earthworks are included in a number of published documents including:

- HA44/91 [17] and HA70/94 [18];
- Trenter and Charles [40] re building on earthworks;
- Reeves et al [41];
- Trenter [35].

These documents provide guidance on the selection of appropriate parameters for earthworks materials (and limited comment on the selection of suitable tests for the practical control of the construction of earthworks).

Fine soils and weak argillaceous rocks that are placed dry in a relatively loose condition are prone to collapse on subsequent wetting (Charles and Watts [42]). It is particularly important that the air voids content of these materials is restricted to prevent collapse settlement. Where possible it is advisable to avoid use of such fills in situations where inundation by floodwater or groundwater is likely.

7.6.5 Compliance testing

The Designer should select the appropriate form of compliance testing for the earthworks. The selection of material properties should consider the feasibility of performing compliance testing relative to the selected acceptability criteria and the constraints imposed by the contract and construction operations.

Relationship testing should be used to determine the correlation between compliance tests that will be used to control the earthworks (such as MCV) and the fundamental soil properties upon which the earthworks design is based (such as undrained shear strength). An illustration of the relationship test concept is provided at Figure 11. The relationship testing should be used to determine the acceptability limits for the chosen compliance tests. The correlation testing should be carried out during the ground investigation phase but may also be required during the construction phase to address natural variation of materials encountered.

Designers should maintain awareness of developing technologies for in-situ and laboratory testing.

Figure 9 Determination of acceptability limits for coarse soils using relationship testing data

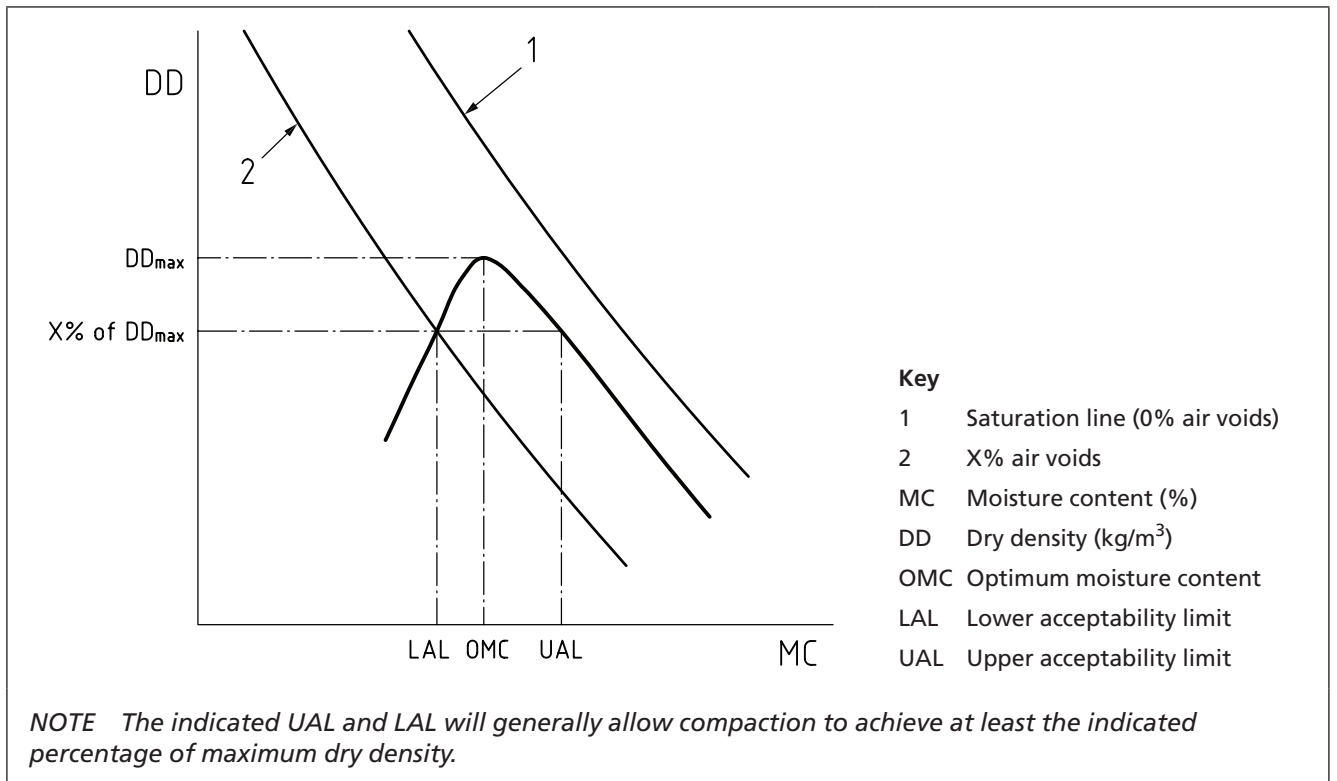


Figure 10 Determination of acceptability limits for fine soils using relationship testing data

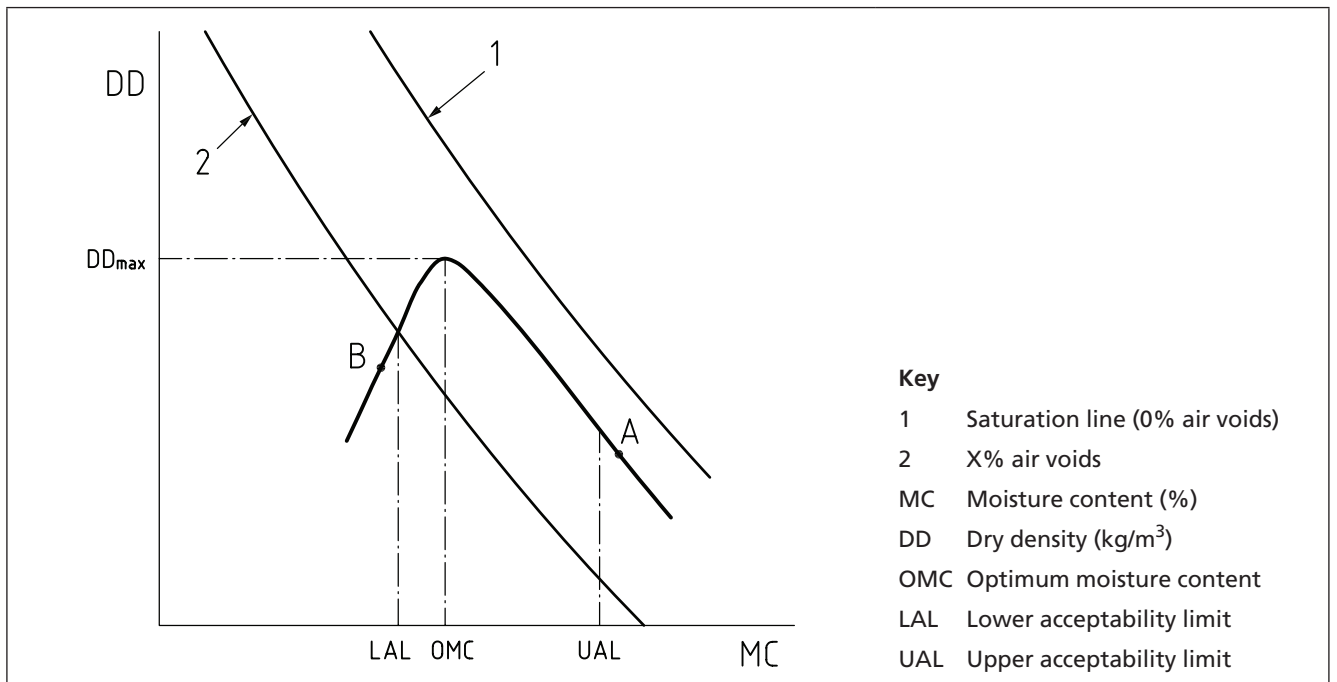
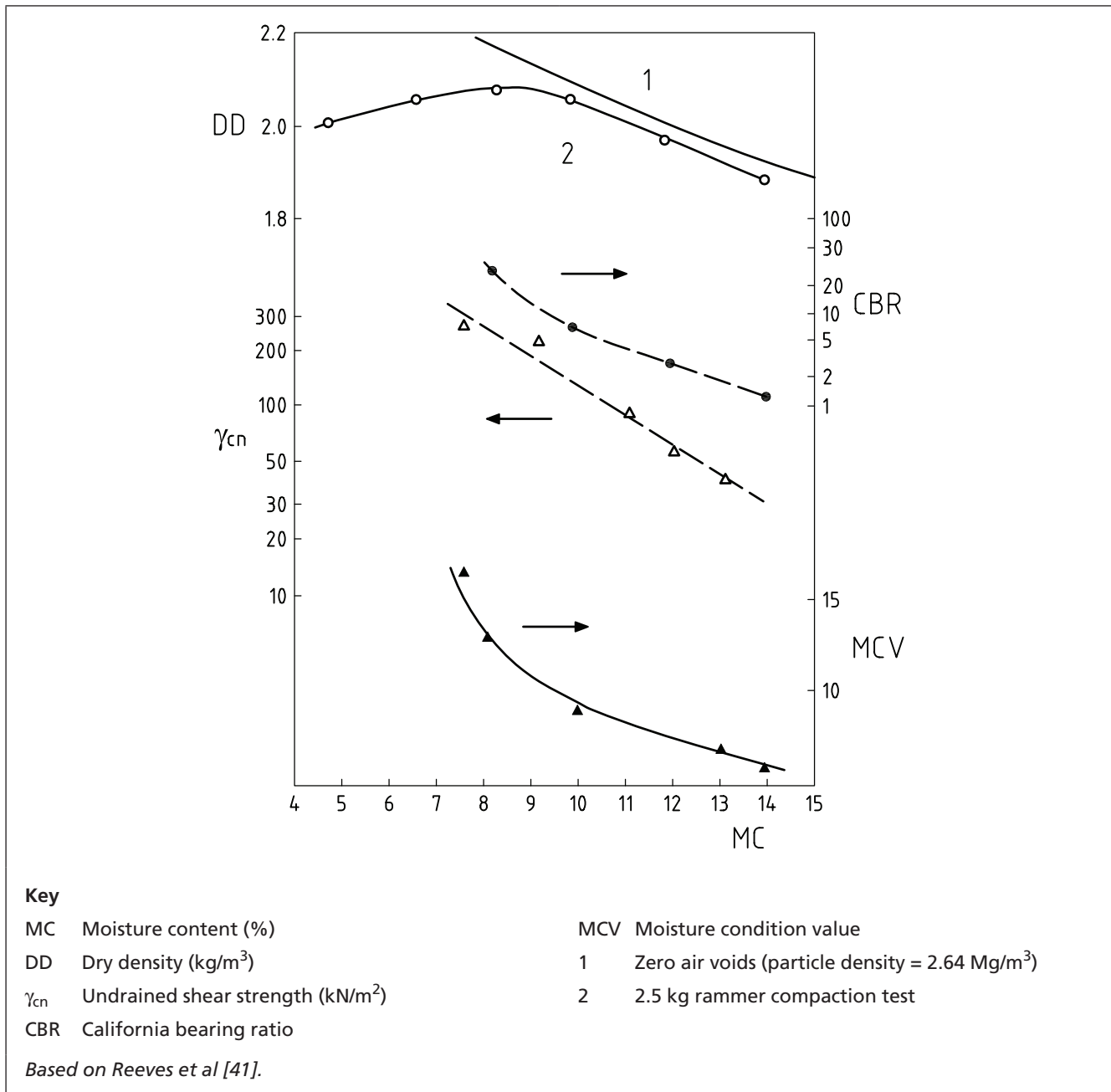


Figure 11 Example of relationship testing



COMMENTARY ON 7.6.5

On most civil engineering projects, the rate of earthworks construction is usually a critical activity. Related to this is the need for rapid turnaround of the results from compliance testing linked to the contract specification. Delays in this process increase the volume of material placed and compacted for which compliance is unproven. When assessing the appropriate form of compliance testing for an earthworks project the designer should be aware of these testing limitations.

Material failing to conform to the specification might require remedial treatment. In the worst case, this can entail excavation of the non-conforming material and its disposal off site. This is wasteful of material and site resources, including plant, fuel, labour and time. The site control and testing procedures should be devised to minimize this risk.

Tests such as the undrained triaxial test, optimum moisture content and Atterberg Limits are not generally appropriate for routine earthworks control, either in the equipment required by a site laboratory or in the time/personnel resource required.

Available rapid methods for determining suitability of cohesive materials include the hand shear vane and the moisture condition value (MCV) test. Both can be carried out in situ and provide immediate results. If in-situ density is required as a control mechanism, the nuclear density gauge is proven technology that may be used.

Additionally, there are several techniques which provide a quick assessment of CBR values; these include the Dynamic Cone Penetrometer and the MEXE cone penetrometer.

7.6.6 Use of potentially contaminated, site-won fill

The earthworks designer should carefully consider the implications of potential contamination in site-won fill. Expert advice should be sought in relation to potentially contaminative previous land uses, regulatory requirements and testing regimes. See also SHW [1] Clause 601.

The earthworks designer should refer to EA guidelines that are current at the time of the design in order to remain aware of current legislation. It is advisable to discuss proposals for use of these fills with the EA (and HSE if an occupational health problem is suspected) at as early a stage as possible. The earthworks designer should avoid the temptation to overspecify the requirements; in general terms if the fill meets the contract terms and is acceptable to the EA then the contractor should consider using it. It may often be appropriate to obtain input by a waste management/human health risk assessment specialist to assess the suitability of the material for reuse.

A sampling and testing plan, comprehensive in both location of sample points and determinands analysed should be prepared to assess the source of material. Recommendations have been published (e.g. BS EN 14899) and have been incorporated by EA in their guidance; it is, however, strongly recommended to seek the advice and assistance of a contaminated land specialist in this.

NOTE Alongside the chemical nature of the material, the earthworks designer will commonly need to consider physical re-processing methods that will be necessary in order to ensure that fill materials will meet the physical requirements of suitable fill (e.g. screening to remove oversize particles).

Designers should be aware that the chemical characteristics of some materials might limit the applications for use.

7.6.7 Stabilized and modified materials

Designers should consider the use of stabilized or modified materials to maximize the use of site-won materials, and should make use of published guidance such as HA 74/07 [43].

COMMENTARY ON 7.6.7

The use of lime for treating cohesive materials and enabling them to be used on site has been established within the UK for a considerable time as has the use of cement to treat granular materials. More recently a two stage process of using lime followed by cement on cohesive materials has been developed – details are provided in the SHW [1].

The two main applications within cohesive soils are:

- *modification/improvement which is a process to render unacceptable bulk fills acceptable and simply uses lime;*
- *stabilization which is used for higher quality uses such as capping/ subbase material or for slope repairs and uses lime together with additional binders such as cement, ggbs, pfa etc. in order to prevent potential swelling effects owing to high sulfur contents.*

There is an extensive suite of European standards which have been developed over the past few years [see BS EN 13286 (all parts) and BS EN 14227 (all parts)].

Britpave (<http://www.britpave.org.uk/>) provide extensive guidance on procedures and considerations that can be undertaken if the option for stabilization is considered. Additional information on the performance, materials, mixture design, construction and control testing of hydraulically bound mixtures for pavements is available from the Concrete Centre [www.concretecentre.com/publications/].

7.6.8 Use of secondary aggregates and recycled materials

Published guidance (see commentary) should be followed on the use of secondary aggregates and recycled material. Data on compaction, durability and environmental aspects, such as leaching, should be sought from potential suppliers before confirming use in design. The designer should seek to minimize overall environmental and economic impact. However, there can be instances where primary aggregates carry the least cost, both in environmental impact and commercial economy.

COMMENTARY ON 7.6.8

Government policy encourages the use of these materials; this is captured in SHW [1], where recycled aggregate is specifically permitted in Table 6/1 for many Class 6 materials.

Although, in general use, the term "recycled aggregate" is used to cover all non-primary material, there are differences between recycled and secondary aggregates. The former have been recovered from previously used material (e.g. crushed concrete and masonry), the latter are by-products of an industrial process (e.g. PFA, china clay stent). Whilst different in origin, both types are covered by legislation to control the process of recovery (and licensing of this by EA) and taxation.

WRAP (Waste and Resources Action Programme) provide information on recycling on their webpages (<http://www.wrap.org.uk/>), which includes Aggregain (<http://www.aggregain.org.uk/>), specifically for recycled aggregate in construction. This includes a directory of suppliers with distance from a defined location.

NISP (National Industrial Symbiosis Programme) <http://www.nisp.org.uk/> exists to create symbiotic links between businesses to reduce waste by keeping material in the chain of utility.

Examples of practical research initiatives that have resulted in guidance notes for designers in order to promote certain recycled materials (e.g. Winter et al [15]), or options in particular settings (e.g. Brampton et al [44]).

In addition, there are a number of materials exchange initiatives, business and publicly funded, with a presence on the internet. As this is a fluid marketplace, the designer is encouraged to search for themselves.

7.6.9 Filling into water

7.6.9.1 Standing water

NOTE Standing water is the term applied to ponds, lakes, canals and water-filled mineral workings.

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

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

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

7.6.9.2 Tidal, river and flood waters

In tidal and flood waters the effects of the rise and fall of the water level and of wave action on the embankment should be given special consideration and techniques such as are necessary in the design of maritime structures should be considered. Where a sudden rise or fall in the level of the water can occur, precautions should be taken to avoid external erosion and to mitigate the effects of sudden drawdown.

NOTE 1 This condition can occur where an embankment crosses the flood plain of a river where the embankment is, for most of the time, on dry ground but where, under flood conditions, erosion of the slopes of the embankment in the vicinity of a bridge or culvert is possible owing to the increase in velocity of the flood water passing through the opening.

Where flowing water against the earthworks face can be expected then measures should be included to prevent erosion of the earthworks. The earthworks engineer will need to consider the risk of erosion and options available for protection, but is likely to require input from a specialist with experience of design of erosion protection to ensure that site conditions are properly understood and that design, installation and maintenance factors are properly allowed for.

NOTE 2 When the risk of erosion in port, coastal and river engineering is judged as sufficient to require the use of rock fill for erosion protection then reference can be made to CIRIA C683 [45]. References are available for river engineering, such as Escaramia and Wallingford [46] and Hemphill and Bramley [47]. For less severe erosion cases then options of green engineering can prove very effective to protect the face of the embankment, examples are given by River Restoration Centre [48].

7.6.10 Filling adjacent to structures

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

- a) filling over large pipes and culverts; in these cases it is important that fill is brought up equally on each side of the structure to prevent unbalanced loading and that great care is taken with the first layers of fill over the top of the structure;
- b) against abutment and wing walls of bridges and retaining walls of all kinds;
- c) around and between skeleton abutments, buried piers and bank seats.

Because satisfactory compaction of fill adjacent to structures is often more difficult to achieve owing to the restricted nature of the operation, it is usual practice to specify particular types of fill, such as selected granular materials (including specialist fill such as pulverized fuel ash), in the immediate area of the structure. Satisfactory compaction to reduce to a minimum differential settlement between backfill and structure is important enough to warrant the use of more expensive materials. Both the type of compaction plant and the method of compaction may be modified from those used in general embankment construction to prevent the development of excessive horizontal forces on foundations, retaining walls or piles.

NOTE Transition zones are commonly utilized to manage the settlement difference between embankments and structures, good practice guidance is provided with UIC 719 [49]. The problem of design of remedial works due to inadequate transition zones at existing structures is a complex issue upon which there is little standard guidance.

7.6.11 Filling over compressible ground

There are various circumstances where earthworks will be required over soils liable to significant settlement, such as soft ground (e.g. alluvium), compressible (e.g. loose Made Ground), collapsing ground (e.g. loess and karst geology), and unstable areas (e.g. land prone to mining subsidence); the designer should assess the magnitude of the risk and give consideration to the acceptable level of deformation for the proposed earthwork and determine an appropriate design logic to suit the site conditions. Guidance is provided in various references, e.g. CIRIA SP32 [50] (currently under revision) and Charles and Watts [42].

NOTE Methods of constructing an embankment over compressible ground include:

- excavation and replacement of the poor material;
- grouting;
- consolidation of the soft material by surcharge;
- staged construction or controlled rate of filling;
- improvement of the engineering properties of the soft material by ground improvement techniques;
- modification of the engineering properties of the soft material by the use of additives such as lime or cement;
- use of lightweight fill;
- drainage of the soft material by the installation of horizontal or vertical drains;

- *reduction in the gradient of the side slopes and / or the provision of berms;*
- *use of synthetic reinforcement; and*
- *use of piles.*

The selection of the method of construction proposed should give particular attention to the potential implications on the environment or adjacent structures and earthworks.

7.6.12 Embankments on sloping ground

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

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

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

7.7 Stability of temporary cuttings and open excavations

The overall stability of slopes for temporary cuttings and open excavations should be determined in accordance with the principles of **11.5.1** of BS EN 1997-1:2004 and the guidance given in **7.3**.

The designer should select appropriate soil parameters for use in the design of temporary slopes. In some cases it may be reasonable to rely on the short term (undrained) parameters where the designer is satisfied that insufficient time is available for a significant rise in porewater pressure to take place. However, this decision must be carefully considered as the transition to partially drained conditions occurs relatively quickly in some fine grained soils in the UK.

NOTE The overall effect of excavation for a cutting is to temporarily increase the stability of the slope due to reduced porewater pressures. With time the reduced porewater pressures rise towards higher equilibrium values with a consequent reduction in the shearing resistance of the soil mass. Thus the most critical conditions for temporary slopes to cuttings and open excavations occur some time after the formation of the slope (see Figure 6). The rate at which the porewater pressures rise towards equilibrium depends primarily on the soil type; for low permeability soils the process of reaching equilibrium porewater pressures may take decades whereas the porewater pressures in a highly permeable soil can reach equilibrium immediately following excavation.

7.8 Trenches with sloping sides

When excavating a trench for which it is not intended to provide additional support, the following should be considered:

- a) the nature of the ground which should be suitable so that the sides of the trench can stand up at a stable angle without support for the required time;
- b) that dewatering of the ground and trench can be effectively carried out to prevent the sides slipping or the trench flooding;
- c) that the permanent work can be installed safely in the trench and that the design of any pipe or structure to be constructed in the trench takes account of open trench conditions in its design;
- d) that excavation plant and equipment selected is appropriate to open trench conditions.

7.9 Planning for construction

NOTE 1 Clause 9 contains further details of construction issues.

When designing earthworks, consideration should be given to how the job will actually be delivered in practice, which is likely to necessitate an assessment of some or all of:

- a) earthworks buildability;
- b) suitability of excavated material for re-use;
- c) cut/fill balance, mass profile, mass haul;
- d) utilizing surplus or unsuitable material on site (discussion on good practice e.g. HA 55/92 [51] which is an advice note now widely implemented as standard good practice);
- e) disposal of surplus materials (all surplus material both unsuitable and suitable);
- f) the earthworks programme;
- g) bulking factors;
- h) trafficability;
- i) effect of weather;
- j) management of potential impact on adjacent/existing structures;
- k) monitoring requirements;
- l) designing for earthworks safety;
- m) maintenance practicalities, e.g. 1:1.5 slopes make grass cutting too difficult;
- n) environmental impact, e.g. flora, fauna, discharge consents (see EIA agreements at planning approval);
- o) emergency response in the event of a major instability.

NOTE 2 Planning earthworks is an essential part of the design and construction process, whether the task involves a small volume of fill over a few months or multiple cut to fill operations over two or more years. The most effective solutions are possible if designer and contractor work in conjunction prior to any operations commencing: some types of contract make this more achievable than others.

If such liaison is not possible, the designer should ensure that they are not carrying out a theoretical exercise, but are setting the framework within which major logistical work has to be executed; a poorly thought out, impractical solution being likely to adversely affect deliverability and, ultimately, the cost.

The planning issues raised and discussed in Clause 9 should be considered holistically by both designer and contractor: there is considerable interaction between them; it is likely that, whilst no individual issue will be dominant, priorities will become apparent as the process is planned in detail; these priorities are also likely to be different from site to site.

NOTE 3 On any type of project, the issue which is likely to present the longest delay is that of obtaining planning consent for any disposal, borrow pit or extraction operation. The timescale for obtaining consent is at least 12 months; greater if objections are sustained. It might be necessary to produce an Environmental Impact Statement to the satisfaction of the Environment Agency. The appointment of an independent planning consultant may be beneficial.

7.10 Vegetation

7.10.1 Use of vegetation to assist surface stabilization

The designer should be aware that vegetation plays an important role in stabilizing slope faces and also acts to reduce erosion (CIRIA C708 [52]). The roots provide a reinforcing action and their need for water will reduce the in situ moisture content of the soil. The effects are principally developed in the surface layers although larger plants such as trees can extend to considerable depths and research has been undertaken into the potential benefits of such techniques as willow poles (Hiller and Macneil [53]).

Grasses are the most common form of control for near-surface stability and details of the appropriate seed mix and sowing requirements are included within Appendix 6/8 of the SHW [1]. The establishment of grass should follow quickly after the spreading of the topsoil on the slopes and it may be necessary to adopt a proprietary retaining system on the steeper slopes to ensure that the topsoil remains in place while the grass establishes. A range of seed mixes may be used to reflect local environmental needs.

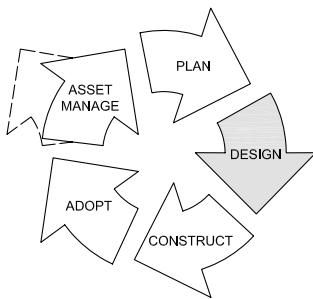
It should be noted, however, that there are potentially significant disadvantages to the use of vegetation which need to be taken into account, particularly when dealing with clay soils. Seasonal effects have a pronounced effect on such soils leading to significant movements as they dry out/shrink in the summer months and swell in the winter months and such movements may lead to distortion within the upper layers of embankments leading to unacceptable ride quality as has been noted on a number of railways. However, research indicates that trees and shrubs can delay the onset of progressive failure of clay slopes (O'Brien [54]) For these reasons careful consideration of the long-term effects of vegetation, in particular trees and large shrubs, should be undertaken as part of the planting regime for the individual slopes.

Designers should avoid retaining or planting large trees near the crest of clay slopes. Large trees have adverse loading effects and tend to dry the soil. Trees placed at the base of slopes can, however, have beneficial effects. When considering any planting scheme the designer should seek advice on long term maintenance of soft estate. Due consideration should be given to the potential for vegetation interfering with the intended use of the earthworks, e.g. trees obscuring sight lines.

See Clause 11 for information on the effect of vegetation on existing earthworks.

7.10.2 Problematic vegetation

Designers should be aware that there are various invasive plant species that should be carefully managed (see CIRIA C679 [55]).



8 Specification of earthworks fill materials

8.1 General

A specification should adequately describe the design requirements, be easily understood by the parties to the contract, be practicable and capable of both enforcement and measurement, and not be unnecessarily costly or time consuming in its application. It should be capable of being monitored by an effective form of quality assurance procedure.

NOTE 1 There are three main types of specification for earthworks used in the UK:

- *method;*
- *end-product;*
- *performance.*

Method specification defines how compaction should be conducted in terms of the types of compaction plant, method of operation, number of passes of the plant and the final thickness of the compacted layer. In the UK the SHW [1] has been developed from research using full scale testing of plant (Parsons [38]) and should be used as the preferred approach. However alternative specifications are not precluded and some relevant information is provided in 8.3 and 8.4 to illustrate the issues that designers should consider when assessing the suitability of an alternative form of earthworks specification.

When the SHW [1] forms the basis of the earthworks specification the document should preferably refer to the SHW rather than repeat the clauses from the SHW (e.g. "The Specification for earthworks shall be Series 600 of the Specification for Highway Works dated 20xx").

NOTE 2 The content of the SHW [1] is updated on a regular basis by the Highways Agency. Within this standard references to the SHW are undated throughout, and consideration was given to the content of the SHW at the time of preparation of this standard.

Any reference made to the SHW [1] within an earthworks specification should state clearly the version of the SHW upon which the specification is based. When invoked as part of a contract document then the edition of the SHW should be stated at the start of the document.

The SHW [1] 600 series may be used as the specification for earthworks within an overall contract specification that is not based on the SHW.

The SHW [1] should be used to satisfy the compaction requirements of BS EN 1997-1:2004, 5.3.3 and the testing requirements of BS EN 1997-1:2004, 5.3.4.

Engineered fills which are used to produce suitably shaped landforms for structures should be constructed to high standards to minimize the risk of ground movements causing damage to property built on shallow foundations. Specifications based on those developed for highway embankments are not necessarily appropriate for fills on which *buildings* will be founded since acceptable settlement is likely to be significantly smaller for a building than for a road; hence a more stringent specification might be necessary than for highway purposes (see 7.2).

Highway schemes are often major civil engineering projects, whereas schemes involving low-rise buildings founded upon engineered fill are often relatively small in scale. Control procedures should be appropriate to the scale of project and criticality of settlement tolerance (see 7.4.2). Control procedures for large highway projects may not always be the most suitable for fill being placed as, for example, part of a small housing development.

NOTE 3 However, the SHW [1] has reached an extensive level of use across UK industry, and it is set out in such a way that allows the designer to tailor the requirements to suit the scheme; consequently it provides the most suitable document for incorporation into BS 6031 by the approach set out in 8.2.

8.2 Specification of earthworks by SHW approach

8.2.1 Required documentation

The appropriate appendices should be provided to enact several of the SHW [1] clauses. As a minimum Appendix 6/1 (including Table 6/1), Appendix 6/2 and Appendix 6/3 should be provided. Appendices 6/8, 6/12, 6/14 and 6/15 should usually be provided. Other appendices should only be supplied when the specific works covered are proposed as part of the scheme. The designer should avoid excessive paperwork for relatively simple schemes as this can hide the important details of which the parties involved need to be aware.

8.2.2 Compaction requirements

It is important that the designer decides whether a material is to be controlled by "method" or "end product" compaction; attempting to combine both approaches for the same material is not appropriate. However, it should be realized that a limited amount of testing should be undertaken during method compaction to verify that the method proposed or adopted is appropriate. Similarly, end product controlled compaction should be monitored to ensure that the criteria and consistency will be achieved.

NOTE 1 The Highways Agency in the UK has adopted a standard classification of earthworks materials. A summary of the classification, first published in Reeves et al [41], is presented in Table 8.

NOTE 2 Method compaction is designed to deliver 90% compaction by BS 1377 compaction test for general fill and 95% for class 6 or end-product fill. The compactive effort stipulated in SHW [1] Table 6/4 is designed to produce an adequate state of compaction (usually 10% air voids or less) at a conservative (low) moisture content for the particular class of soil (see SHW [1] NG 612 for more details). See 7.6.4 for further explanation.

NOTE 3 There may be variations in testing procedure and/or interpretation of test results to take account of local variations in soil characteristics, e.g.:

- *testing moisture content at various gradings can be beneficial, e.g. the moisture-susceptible (< 425 µm) fraction of glacial till, or < 20 mm fraction of Class 2C fill in order to relate to other earthwork relationship test data; and*
- *many of the standard tests included in BS 1377 cannot be usefully employed on some of the UK soils primarily due to their coarse nature (> 10% of material is retained on the 37.5 mm sieve). In order to overcome such problems and still enable classification testing to be performed on earthworks materials local variations to these methods have been adopted together with suitable acceptance criteria.*

Table 8 Classification of earthworks materials in the UK by the Highways Agency

| Type | Class | Description | Typical use |
|------------------------|---|---|---|
| General granular fill | 1A | Well graded granular material | General fill |
| | 1B | Uniformly graded granular material | |
| | 1C | Coarse granular material | |
| General cohesive fill | 2A | Wet cohesive material | General fill |
| | 2B | Dry cohesive material | |
| | 2C | Stony cohesive material | |
| | 2D | Silty cohesive material | |
| | 2E | Reclaimed pfa cohesive material | |
| General chalk fill | 3 | Chalk | General fill |
| Landscape fill | 4 | Various | Fill for landscape areas |
| Topsoil fill | 5A | Topsoil or turf existing on site | Topsoiling |
| | 5B | Imported topsoil | |
| Selected granular fill | 6A | Selected well graded granular material | Below water |
| | 6B | Selected coarse granular material | Starter layer |
| | 6C | Selected uniformly graded granular material | Starter layer |
| | 6D | Selected uniformly graded granular material | Starter layer below pfa |
| | 6E | Selected granular material | For cement stabilization to form capping – class 9A |
| | 6F1 | Selected granular material (fine grading) | Capping |
| | 6F2 | Selected granular material (coarse grading) | Capping |
| | 6F3 | Selected granular material recycled bituminous/asphaltic materials, etc.) | Capping |
| | 6F4 | Selected/imported (unbound) granular material that conforms to BS EN 13285 (fine grading) | Capping |
| | 6F5 | Selected/imported (unbound) granular material that conforms to BS EN 13285 (coarse grading) | Capping |
| | 6G | Selected granular material | Gabion filling |
| | 6H | Selected granular material | Drainage layer |
| | 6I | Selected well graded granular material | Fill |
| | 6J | Selected uniformly graded granular material | Fill |
| | 6K | Selected granular material | Lower bedding for: |
| | 6L | Selected uniformly graded granular material | Upper bedding for: |
| | 6M | Selected granular material | Surround to: |
| | 6N | Selected well graded granular material | Fill to structures |
| | 6P | Selected granular material | Fill to structures |
| 6Q | Well graded, uniformly graded or coarse granular material | Overlying fill for corrugated steel buried structures | |
| 6R | Selected granular material | For stabilization with lime and cement to form capping – class 9F | |
| 6S | Selected well graded granular material | Filter layer below subbase | |
| Selected cohesive fill | 7A | Selected cohesive material | Fill to structures |
| | 7B | Selected conditioned pfa cohesive material | Fill to structures and reinforced soil |
| | 7C | Selected wet cohesive material | Fill to reinforce soil |
| | 7D | Selected stony cohesive material | Fill to reinforce soil |
| | 7E | Selected cohesive material | For stabilization to |
| | 7F | Selected silty cohesive material | form capping to: |
| | 7G | Selected conditioned pfa cohesive material | Overlying fill for corrugated steel buried structures |
| | 7H | Wet, dry, stony or silty cohesive material and chalk | For stabilization with lime and cement to form capping – class 9E |
| 7I | Selected cohesive material | | |
| Miscellaneous fill | 8 | Class 1, class 2 or class 3 material | Lower trench fill |
| Stabilized materials | 9A | Cement stabilized well graded granular material | Capping |
| | 9B | Cement stabilized silty cohesive material | |
| | 9C | Cement stabilized conditioned pfa cohesive material | |
| | 9D | Lime stabilized cohesive material | |
| | 9E | Lime and cement stabilized cohesive material | |
| | 9F | Lime and cement stabilized well graded material | |

NOTE Clays and cohesive soils are shown in shaded rows.

8.2.3 Testing requirements

The designer should provide a table that clearly sets out the testing requirements for earthworks materials. This may be in the form presented in Table NG1/1 of SHW[1]. The designer should include either the frequency or number of tests dependent on the size or duration of the works being undertaken.

NOTE Table 9 is based on Table 3/1 in HA 44/91 [17] as an example.

Table 9 Example of classification and acceptability testing table

| Material class | Requirement | Suggested frequency |
|--------------------------|--|---|
| 1 General granular fill | Grading/uniformity coefficient MC/MCV | Twice a week 1–2 tests per 1 000 m ³ of material up to a maximum of 5 per day |
| | IDD of chalk | Twice a week |
| 2 General cohesive fill | Grading MC/MCV/PL/shear strength | Twice a week 1–2 tests per 1 000 m ³ of material up to a maximum of 5 per day |
| | IDD | Twice a week |
| | Bulk density (pfa) | 1–2 tests per 1 000 m ³ of material up to a maximum of 5 per day |
| 3 General chalk fill | MC | 1–2 tests per 1 000 m ³ of material up to a maximum of 5 per day |
| | IDD | Daily |
| 4 Landscape fill | Grading/MC/MCV | Daily |
| 5 Topsoil | Grading | Daily |
| 6 Selected granular fill | Grading/uniformity coefficient | One test per 400 t of material |
| | I_p /LL | Daily |
| | LA coefficient/IDD | Weekly |
| | OMC/MC/MCV | One test per 400 t of material |
| | Organic matter/total sulfate content | As required or weekly |
| | pH/chloride ion content | As required or weekly |
| | Resistivity | As required |
| | Undrained shear parameters | As required |
| 7 Selected cohesive fill | Grading/MC/MCV | One test per 400 t of material |
| | IDD | As required or daily |
| | I_p /LL | As required or weekly |
| | Organic matter/total sulfate content | As required or weekly |
| | pH/chloride ion content | As required |
| | Resistivity | As required |
| | Undrained and drained parameters | As required |
| | Permeability | As required |
| | Coefficient of friction/adhesion | As required |
| 8 Miscellaneous fill | MC/MCV | Daily |
| 9 Stabilized materials | Pulverization | One test per lane width per 200 m length |
| | MC/MCV | One test per lane width per 200 m length |
| | Bearing ratio | One test per lane width per 200 m length |

The types of test should be related to the material properties specified in SHW [1] Appendix 6/1, including a check test to ensure that the required density has been achieved if required by the designer/overseeing organization.

The preferred method of specifying moisture limits on clays in the UK is the MCV, which is quick to measure on site and for which there is a substantial base of experience; typical values commonly used are:

- Class 2A (wet cohesive) 8–12;
- Class 2B (dry cohesive) 12–16;
- Class 2C (stoney cohesive) 8–16.

These limits may be varied depending on site-specific relationship testing to fundamental properties, as described in Table 10 (based on Table 4/2 in HA 44/91 [17]).

Table 10 Classification and acceptability tests

| Test | Applicable material type | Purpose | Reference |
|--|-------------------------------|---|---|
| Moisture content | All | Classification/stabilization | BS 1377-2, BS 812-109 |
| Atterberg limits | Cohesive | Classification | BS 1377-2 |
| Particle size distribution | All | Acceptability/classification | BS 1377-2 ^{A)} |
| MCV | Cohesive and/or some granular | Acceptability/trafficability | BS 1377-2, Clause 632 of SHW [1], TRRL LR 1034, TRRL LR 130, TRRL LR 90 |
| Maximum density and optimum moisture content | Mainly granular | Acceptability/compatibility | BS 1377-2, BS 812-109 |
| CBR | All except coarse granular | Trafficability/stabilization/classification | BS 1377-2, BS 1924 (both parts) |
| Triaxial (quick) | Cohesive | Acceptability/trafficability | BS 1377-2 |
| Chemical tests | All | Acceptability | BS 1377-2 |
| Relationship testing ^{B)} | All | Acceptability | BS 1377-2 |

^{A)} The requirements of BS 1377-2 may be added to, to include all sieve sizes quoted in Table 6/2 of the SHW [1].

^{B)} Testing soils at various moisture contents to study the change in soil properties.

The MCV should not be used for stoney clays if there is insufficient matrix (typically less than 50%–55%) for the test and in such cases reliance on moisture content is necessary (Oliphant and Winter [56]).

NOTE An example of classification and acceptability criteria is given in HA 44/91 [17], Annex A.

SHW [1] Appendices and Tables should be developed to reflect local knowledge of materials and experience of particular equipment.

8.3 Specification of earthworks for minor works

The SHW [1] has been developed for large highway schemes, however it may be used effectively on minor projects. As a minimum, this should include: Appendices 0/1, 0/2 and 1/5 together with the appropriate 600 series appendices, e.g., Appendix 6/1. Further guidance may be found in the *Notes for Guidance to the Specification for Highway Works* [2].

8.4 Additional requirements for deep fill areas/buildings and structures

Collapse compression upon groundwater inundation is a major hazard for buildings and other structures on significant thicknesses of fill; therefore the specification of placement and compaction of the fill should be designed to eliminate collapse potential.

NOTE The risk of collapse upon inundation is particularly high where fill is placed below the potential groundwater level (e.g. infilling of a quarry), but is present at many other sites due to risk of inundation following a water main burst.

It should be noted that, the collapse potential of some fills will not be eliminated despite the achievement of a field dry density equivalent to at least 95% of the maximum dry density achieved using the British Standard compaction tests (see BS 1377-4). Consequently, in such cases, dry density should not be relied upon to provide an adequate measurement for compaction specification. Where there is an unacceptable risk of collapse upon inundation the specification should include a requirement for all fill to be compacted to < 5% air voids. See Charles et al [39] and BRE Digest 427 [37].

8.5 Alternative earthworks specifications

8.5.1 General

NOTE As described in 8.1, it is assumed that the default situation in the UK is that earthworks are undertaken in accordance with the SHW [1] (subject to any additional requirements to address specific risks described within this standard).

The SHW [1] is commonly adopted for earthworks; however, if an alternative specification is used (e.g. *Model specification for fills*, Trenter and Charles [40], or a company in-house earthworks specification) then the specification should provide as a minimum the following information:

- a) types of materials permitted for use in the earthworks together with material properties;
- b) performance requirements to be met;
- c) requirements for the disposal of unsuitable material;
- d) requirements for placement, spreading and compaction of the earthworks materials;
- e) requirements for the treatment of exposed surfaces;
- f) requirements for the testing and verification of compliance.

This subclause sets out the recommendations for any alternative to the SHW; if the earthworks specification conforms to these recommendations and addresses the requirements of this standard then those earthworks may be considered as conforming to this standard.

8.5.2 End product

For this form of specification the designer specifies the degree of compaction necessary for the given material by reference to criteria linked to either serviceability or ultimate limit states; the level of compaction required should be expressed in terms of selected geotechnical properties e.g. percentage of maximum dry density and is supported by rigorous on-site testing.

An end product specification may be used to control earthworks provided the approach will adequately control the various issues that affect earthworks. For instance the specified parameters should not concentrate on the issue of stiffness (or shear strength) alone since the control of air voids in compacted material is important in restricting the potential for excessive settlement if an increase occurs in the moisture content of the material. Where an end product specification is adopted, the employer's requirements may set overall targets to be achieved without detailing the methods used to achieve the targets. In this case the following minimum requirements should be addressed (acceptable limits may be set on these criteria or this may be stated as being for the earthworks designer to assess).

- Materials used should be chemically suitable for the environment in which they are used; some material might require treatment (e.g. stabilization or remediation) and consent prior to use.
- Materials used should be durable (not prone to deterioration) and non-biodegradable.
- The earthworks should provide a stable finished surface that will not suffer unacceptable post construction settlement or movement.
- The earthworks should provide a surface of sufficient stiffness (and or shear strength) for the intended end use (if a stiffness value to be achieved is specified then the value will need to consider both the imposed load and settlement induced by the loaded area).

NOTE That the works can be constructed, maintained and demolished safely are requirements of the CDM 2007 Regulations [10].

The above criteria are associated with showing that the works should be constructed so that they are suitable for the proposed end use. To achieve these objectives the earthworks contractor and the designer should consider a range of practical issues, and to deliver this should effectively require that a method of working/specification is put in place (whoever writes this document will effectively become an earthworks designer).

8.5.3 Performance specification

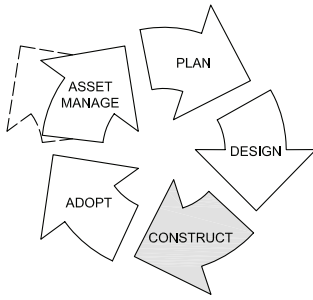
A performance specification should be designed in terms of the required serviceability limit state: e.g. "the maximum differential settlement should not exceed 1:200 over a defined length, five years after construction and the maximum settlement in any one area should not exceed 25 mm".

This may be considered an onerous form of specification from a contractual viewpoint as it seeks to place the risk for all future events on the Contractor and might be very difficult to monitor in practice (for example, see Virginia TRC [57]).

COMMENTARY ON 8.5.3

The way that control of earthworks is being developed in the USA and Europe is of direct relevance to future earthworks construction in the UK. Intelligent compaction or continuous compaction control has the potential to improve infrastructure performance, reduce costs, reduce construction programmes and improve site safety.

A compaction control approach using modulus and moisture content (plus air voids) has the potential to fit well with performance specifications and could be monitored in real time using roller-mounted devices (see Mooney and White [58]).



9 Construction of earthworks

9.1 General

At this point in the process the team should have developed the design and specification for the works, but these documents alone will not deliver a successful earthworks project; achievement of this goal depends greatly on the practical experience of the construction team, and in particular the earthworks manager and earthworks foreman.

The construction team should plan the earthworks construction to optimize the use of the fill materials available, prepare a mass haul diagram to ensure that the proposals can fit with other aspects of the construction works, and programme activities such as haul roads and drainage to ensure that the fill material does not deteriorate during the works. The construction team should take account of the recommendations in Clauses 5 to 8.

NOTE These activities rely on experience of earthworks, the ground conditions at the site, and the climatic conditions of the area. Without this experience, there is a considerable risk of potentially good material being rendered unsuitable and the project running into major difficulties.

The following subclauses provide some general comments on these aspects of earthworks and how the works should be undertaken. These comments should not however replace the construction team's practical experience.

9.2 Organization of earthmoving operations and construction practice

Earthmoving operations by their nature present considerable risk; safe systems of work and good working practices should be adopted to minimize that risk. Provision should also be made for emergency response in event of a major instability.

9.3 Planning of the earthmoving operations

The earthworks planning process should be ongoing from project conception through the design and construction phases of the project. The advice of staff with practical experience of earthmoving operations should be taken into account at all stages of that process.

Earthworks planning should take account of factors which can have a major impact on the programming of earthmoving operations such as:

- contractual constraints;
- environmental constraints;
- agricultural constraints;
- constraints imposed by the presence of adjacent infrastructure;
- weather;
- site investigation data;
- control of settlement and temporary stability of slopes and embankments;
- the discovery of archaeological remains;
- the presence of protected species or habitats.

Land acquisition should be planned not only to allow for construction of the permanent works but to take account of land required for temporary road or rail diversions, site compounds, welfare facilities, tips and borrow pits, delivery and storage areas, haul roads and turning areas, hard standings for craneage, services, runoff and pollution control, watercourse and temporary utility diversions.

Road and railway projects are characterized by being linear worksites accessible from both ends and with few intermediate access points: whilst the provision of access points for work activity, along the site might not be necessary, additional access points for use in an emergency should be planned.

Earthmoving operations should be planned to be constructed in such a way as to minimize risk to site personnel and the public. Planning should allow for the safe movement of vehicles and personnel. Traffic flow should follow a logical pattern in which conflict between vehicles and between vehicles and personnel is avoided. Traffic flow patterns should minimize the need for reversing, and one-way flow patterns are often useful in this respect. Appropriate signing and traffic control measures should be devised and realistic speed limits should be set. An appropriate network of well-maintained access and haul roads, and passing and turning points should be planned and sufficient land for these should be acquired during the land acquisition phase. Particular attention should be paid to loading, tipping and spreading areas. Pedestrians should be kept separate from traffic routes and safe systems of work devised to allow supervisory staff, banksmen, etc. to work safely and effectively. Site rules should be drawn up to reflect these objectives.

Any use of explosives for rock excavation should be undertaken in accordance with BS 5607.

9.4 Earthmoving plant and equipment

All earthmoving plant and equipment used should conform to the requirements of BS EN 474 (all parts) or BS EN 500-4. It should be maintained in a serviceable condition in accordance with the manufacturer's recommendations.

NOTE Plant and equipment not conforming to these standards can present increased risks due to deficiencies such as poor operator visibility.

Plant and equipment should be selected on the basis of suitability for the planned operations and the expected site conditions. Environmental considerations should also be taken into account. Excavating, hauling and compaction equipment should all be sized to be compatible with each other; large plant requires fewer movements so reducing the overall number of vehicle movements.

Use should be made of remotely operated fittings, such as visibility aids, self sheeters, hydraulic doors and interchangeable tools, on wagons and plant to enhance safety by reducing operator input away from the safety of the cab.

Plant and equipment should not be used in inappropriate situations, e.g. road compaction plant should not be used for earthworks compaction.

Plant inspections should be done on a regular basis and all hired plant should be included in the process prompted by receipt of the delivery ticket on site.

9.5 Operator fitness

Only operators who have demonstrated their competence, and are authorized to do so, should be allowed to operate earthmoving plant and equipment. It is recommended that a competence assessment scheme such as that run by the CPA (www.cpa.uk.net) should be adhered to on site.

Operators should be subject to occupational health checks to confirm their fitness for work. Such checks should be no less stringent than for road going vehicle operators.

A drug and alcohol policy should be implemented on site which should reflect laws for driving on the highway or better.

9.6 Site interfaces and demarcation

The site boundaries should be physically established before earthmoving operations begin. Permanent or temporary fencing or walls should be erected as appropriate. Barriers that are more robust should be used where the site is adjacent to live road or rail traffic.

The number of crossing points over public highways, bridleways, farm roads, etc. should be minimized. Traffic at crossing points should be appropriately controlled and lit as necessary.

Temporary demarcation of site traffic routes, working areas, haul road boundaries, underground and overhead service locations, drainage or settlement ponds and sterile or contaminated areas, etc. should be provided as appropriate.

Wheel washes should be provided at site exit points. Road cleaning equipment should be deployed to prevent surface contamination of roads and footpaths affected by the works. The use of concrete or bituminous surfaced areas or temporary matting at exit points and crossings can reduce the carry over of contamination.

The runoff of contaminated water from the site should be controlled by the use of drainage channels along with settlement and other treatment facilities; contamination can arise from suspended soil particles in the runoff as well as from spillage of fuel, lubricants and chemicals used on site.

Water spraying vehicles should be deployed to control environmental dust nuisance.

Gates, entrance security checkpoints, etc. should be located to prevent site traffic obstructing existing roads or footpaths. Parking areas for delivery vehicles and employee vehicles should be provided. Materials delivery vehicles should be subject to similar traffic management procedures as site vehicles.

Co-operation and co-ordination between adjacent projects should be encouraged through the planning and management of works that occur near or on mutual site boundaries.

9.7 Workplace transport practices

Good working practices should be adopted to assist in the mitigation of workplace transport risk during earthmoving operations. Consideration should be given to both the workforce and the public.

Information on routes and parking or delivery points should be provided to all drivers, operators, delivery drivers and visitors during a site induction and should be updated from a manned and controlled site entrance.

Pedestrian and cycle routes should be kept separate from vehicle routes and working areas. Further division between heavy earthmoving plant and light wheeled traffic should be considered where practicable. Supervisors should aim to work from vehicles and use radio to communicate with plant operators. Surveying and earthworks testing should be kept separate from earthmoving operations. Movement of site personnel, including visitors, should be preferably by vehicles and walking should be designed out of the works where reasonably practical to do so.

Welfare and messing provisions should be managed to minimize the need for pedestrian movements. Personnel transport from remote locations should be provided if appropriate.

Fuel distribution and emergency maintenance carried out on site should be undertaken in demarcated areas.

The need for banksmen should be carefully assessed and they should be deployed sparingly. Signalling to operators as to where to load or tip may often be done from their cabs by the excavator operator or by the operator of the machine spreading the tipped loads or remotely by radio. End tipping over unprotected faces should be avoided by the use of an attendant dozer.

Haul roads should be of good construction, well drained, properly maintained and demarcated. Speed limits on haul roads should be appropriate to the traffic using them and enforced by traffic calming measures such as changing layout, chicanes, speed bumps, hanging gantries, traffic lights and proactive enforcement by supervisory staff.

Signing, traffic control measures and lighting at crossing points should all reflect current highway practice (see DMRB Volume 8 [59]).

The wearing of hi-visibility clothing by personnel and the use of flashing beacons on vehicles should be standard practice.

9.8 Material delivery

Delivery of materials to site should be managed by the contractor. In planning deliveries the contractor should consider; the physical constraints imposed by the site and access routes, the quantities of material involved, off-loading provision, security and protection of goods, and opportunities to avoid double handling.

Consideration should also be given to:

- a) the packaging materials used so as to minimize waste generation and disposal costs;
- b) the time and frequency of delivery, arrangements for being received on site, restrictions on roads and areas outside the site, e.g. deliveries past schools;
- c) the induction process for the driver, security clearances or other requirements that the driver has to have to enter the site (in addition to those required to drive the vehicle or transport specific goods);

- d) information for the driver such as site entrances and exits, site plan and hazards (cranes lifting loads/machine movements), delivery areas, turning areas, traffic restrictions, and means of escort if required;
- e) contingency plans;
- f) a plan/safe system of work for the material storing requirements, e.g. stacking, flat on floor, in a covered building, with a view as to how this might affect their subsequent removal by others.

Storage areas should be located in one place with materials being distributed out in smaller quantities as required.

9.9 Control and monitoring of earthworks

9.9.1 Inspection of excavations

Excavations for cuttings and foundations should be inspected to confirm the design assumptions were appropriate.

9.9.2 Stability and settlement monitoring

Monitoring of earthworks stability and settlement should be undertaken and recorded. The monitoring regime should be designed and implemented to ensure that:

- a) adverse ground movement is detected during and post construction;
- b) resultant damage to structures can be minimized; and
- c) the design assumptions have been satisfied.

The information from monitoring should be used to initiate and control works to prevent damage where necessary.

9.9.3 Observational methods

The observational method may be adopted for the control and monitoring of earthworks (e.g. its principles are routinely used on earthworks projects for construction of embankments over soft ground, or excavation adjacent to sensitive structures). It should be undertaken in accordance with the principles set out in CIRIA R185 [16]

COMMENTARY ON 9.9.3

The observational method in ground engineering is a continuous, managed, integrated process of design, construction control, monitoring and review which enables previously defined modifications to be incorporated during or after construction as appropriate. All these aspects have to be demonstrably robust. The objective is to achieve greater overall economy without compromising safety. The method's origins are found in the development of "modern" soil mechanics theories in the late 1940s, when an integrated process for predicting, monitoring, reviewing and modifying designs evolved. In the 1990s there was a noticeable increase in its use, and extension of its principles. The OM has been recognized as a design method in design codes such as BS EN 1997-1:2004.

9.9.4 Earthworks control testing

The minimum amount of control testing should be given in the specification for earthworks in conformity with Clause 8. However, the contractor may choose to do more testing to ensure the specification is met, and to assess the sensitivity of materials for handling purposes.

Even when using an end product specification, the contractor should develop a methodology to control the works in order to ensure that the final end product will be achieved.

NOTE This can be particularly relevant for end product earthworks for developers where the contractor can effectively become the earthworks designer to meet a limited set of criteria.

9.10 Embankments

The method of embankment construction should normally be determined by the earthworks contractor. The compaction plant should be chosen to suit the nature of the fill and the scale of the operation, and reference should be made to SHW [1] Table 6/4.

NOTE Guidance on the formation of embankments are provided in SHW [1] Clause 612

9.11 Excavations

The method of excavation should normally be determined by the earthworks contractor based on the scale of the excavation, materials to be removed and availability of plant. Temporary drainage during construction should be installed to minimize deterioration of fill material and sub-grade (see 7.5).

NOTE 1 Current practice for mechanical excavation is to use backacters and face shovels.

Within the vicinity of buried services, hand-digging to locate the services should be undertaken.

Where buried structures or rock are to be removed, more specialist techniques should be specified (see 9.12).

NOTE 2 Guidance on the formation of cuttings and cutting slopes, and excavation for foundations are provided in SHW [1] Clauses 603 and 604, respectively.

9.12 Excavation in rock

9.12.1 General

The contractor should determine the ease of excavation in rock from interrelationship between a number of physical parameters, the most important of which include the intact rock strength, degree of weathering and the nature and spatial distribution of discontinuities within the rock mass.

NOTE 1 Very strong, unweathered, weakly discontinuous rock masses (e.g. coarse crystalline igneous and metamorphic rocks, or massive sandstones and limestones) require a significant amount of inputted energy to break them and enable their excavation. Weaker, highly fractured or weathered rock masses can be excavated with relative ease, similar to that of a soil.

There is a range of methods which may be considered for breaking out rock taking into account the noise, dust and vibration generated, the geotechnical properties of the rock mass, the amount of rock to be excavated and the environment in which the work is to be undertaken; these include:

- drill and blast (see BS 5607 and BS EN 791);
- mechanical and chemical bursting techniques;
- ripping;
- backacters and face shovels;
- impact hammers (see BS EN 12111); and
- roadheaders (see BS EN 12111).

Specialist advice on the most appropriate method to use for a given project with unfavourable rock mass characteristics and environmental scenario should be sought from an experienced earthworks contractor.

NOTE 2 Further guidance can be found in Pettifer and Fookes [60], Fookes and Sweeney [61], Caterpillar Performance Handbook [62].

9.12.2 Special considerations for blasting

Careful consideration should be given to the consequences of blasting where:

- a) the site is in close proximity to structures or populated areas;
- b) the site is adjacent to transport corridors;
- c) the site adjoins public buildings such as schools or hospitals;
- d) unacceptable levels of noise, fume or vibration would be generated by the blasting;
- e) damage might be caused to excavation supports or the surrounding ground, rendering the design unsafe; or
- f) excessive fracturing of the resultant material for fill purposes.

Where a structure(s) might be influenced by ground borne vibrations derived from blasting operations, a full photographic structural survey should be conducted by an appropriately qualified person prior to commencing excavation.

When moving rock broken out using explosives, care should be taken to ensure no undetonated explosives remain in the material to be excavated.

NOTE Further guidance can be found in SHW [1] Clause 607.

9.12.3 Control of overbreak

In order to minimize overbreak, specialist techniques such as rock sawing, pre-split blasting, etc. should be employed. Specialist contractors should be consulted on the most appropriate method to use for the situation.

NOTE Overbreak frequently occurs within rock masses having unfavourable spatial distribution of the discontinuities, particularly within confined excavations, depending on the excavation method employed. This can lead to destabilization by undermining and loosening of the rock as well as the generation of excess spoil and the need for imported materials and/or concrete to make good formations, etc.

9.13 Earthworks balance and material suitability

The design of the earthworks should seek to achieve a cut-fill balance and thus minimize the volume of offsite disposal or the volume of imported fill. The contractor should maintain this balance. However, factors of which the contractor should be aware, and which can alter the cut-fill balance include:

- unforeseen, unsuitable material;
- unforeseen, contaminated material incapable of remediation;
- unforeseen factors affecting the execution of the works (e.g. traffic management constraints, land access constraints, or programme constraints, all of which can create local surpluses or shortfalls);
- inappropriate use of material with specific performance criteria (e.g. use of structural backfill for embankment construction);
- bulking which occurs when any material is excavated and re-compacted;
- poor management of surface and ground water or construction plant resulting in suitable fill becoming unsuitable.

The effect of pavement design on the earthworks balance should not be underestimated.

9.14 Programme and weather windows

In programming earthworks, the following should be considered.

- Adverse weather, which is probably the single greatest cause of delays to an earthworks programme.
- The Met Office (www.metoffice.gov.uk) provides historic weather records on a regional and seasonal basis, which should be consulted when programming earthworks.
- Freezing weather can be a limited opportunity to progress the works by moving materials along haul roads unusable due to softening by rainfall.

9.15 Physical constraints

When creating a mass haul diagram the following physical constraints to bulk movement of materials should be considered:

- railways;
- significant watercourses;
- major roads (for which a plant crossing is not permitted);
- deep valleys too steep for efficient plant working (requiring a major structure to bridge);
- hills which are to be tunnelled;
- poor ground conditions, e.g. bog, or other ground requiring treatment before trafficking; and
- environmentally important sites (e.g. SSSIs).

Means of overcoming these constraints may include:

- delaying earthworks in the section affected until the structure bridging the constraint is able to take construction traffic;
- temporary bridging, e.g. Bailey bridge;
- transporting material across the constraint by conveyor belt or similar;
- moving material around the constraint by other means, e.g. road lorries;
- relocation of flora and fauna.

NOTE Each of these solutions has its benefits and drawbacks. The optimal solution will depend on the circumstances.

9.16 Interface with other project activities

Activities which might disrupt bulk earthmoving productivity should be considered carefully and with the full involvement of the earthworks management team.

If the critical path network of a project identifies work for which the related earthworks have to have reached a certain stage, the mass haul diagram should incorporate this.

NOTE The prime earthmoving season is a finite number of working hours; any other activity on a project which reduces these is potentially creating an unrecoverable loss of production. Earthworks subcontracts are let with definitive outputs which have to be achieved for the plant to be operating commercially and effectively.

9.17 Resolving material surplus

If, after all possible changes to vertical alignment have been input, a surplus of material remains, various options may be chosen.

- Stabilization to produce pavement foundation or base: this is doubly advantageous in reducing imported primary or recycled aggregates. Suitability of the material for stabilizing should be ascertained. The likelihood of this should have been foreseen at design investigation stage and the appropriate testing carried out. (See HA 74/07 [43].)
- Modification to produce suitable fill: mixing with a lower percentage of lime than required for stabilizing, or blending with other materials (e.g. PFA) may be used to revert material that is too wet to be acceptable, which reduces the volumes of unsuitable material for disposal and import of suitable to replace.
- Synergy with other projects: the solution may depend on another project in terms of cost-effective road-haulage distance of material (timing is critical).
- Disposal alongside the project: in an urban environment this is unlikely to be available; in an agricultural situation, selection of location for a suitable landform may involve considerable research. However, the filling of valleys for agricultural betterment and the creation of false cuts for environmental screening are options.

NOTE These are likely to attract landfill tax if not incorporated in the works at design stage (i.e. land obtained by compulsory purchase order). If taken at a later date by a contractor, agreement of the landowner (including royalty) and planning consent will be required.

- Road haulage to offsite disposal facility: for inert material in a rural/agricultural project, this should be seen as the last resort and, as the most un-environmentally sensitive solution (unsatisfactory).

9.18 Resolving material shortfall

The options that may be chosen are likely to be as follows:

- synergy with another project with a surplus: as above, location and timing are critical;
- import from commercial sources: market rates will apply and the material is likely to be processed/recycled granular;
- borrow pit: as with disposal, selection of location will involve much research.

NOTE It is unusual for this solution to be carried out other than by the main works contractor. Landowner agreement and ascertaining that the material is, in fact, suitable, are essential. Planning consent is required. The essence of a borrow pit is for acceptable material to be dug and replaced with unacceptable. In reality this might not be feasible, at least in part. In any event, the final landform will be a condition of any consent granted.

9.19 Quarrying for aggregates and selected fills

Distinct from a borrow pit to supply general fill for earthworks, the opportunity may exist to extract locally available sands, gravels and rock by a quarrying operation contiguous in location and time to the project.

NOTE A planning application is unlikely to succeed unless the operation can be shown to provide significant environmental betterment over procurement from the existing marketplace. Any consent will take at least a year, much longer if objections are raised.

9.20 Design, construction and maintenance of haul roads

Earthmoving plant generally uses the haul route along the line of the project, the vertical alignment of which changes as cuts are excavated and embankments filled; provision should be made for plant to maintain this route, which, at a minimum, would be a motor grader to remove rutting before this reduces the efficiency of the plant.

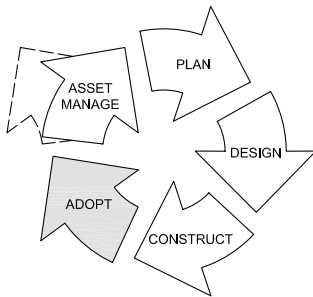
In some circumstances, a dedicated, constructed haul road might be required; as failure of this haul road might significantly disrupt the earthmoving process, the design and construction should be substantially adequate in considering the volume of traffic it has to sustain. Maintenance of the haul road, e.g. patching should be carried out before deterioration results in structural failure.

9.21 Plant selection

Plant should be selected to suit both the desired rate of output and the nature of the fill to achieve desired excavation, transport and compaction of the fill. Earth moving plant manufacturers supply information on this topic (e.g. *Caterpillar Performance Handbook* [62]). It is important to recognize that heavy compaction plant can over-compact, raise pore water pressures, and consequently weaken certain fine-grained soils, which is a practice that should be avoided. The use of motor- graders and water bowsers to regularly maintain working areas and control dust during construction can have significant benefits that should be considered.

9.22 Geotechnical feedback

Geotechnical feedback should be provided, in proportion to the complexity of the earthworks (see 6.5).



10 Adoption

10.1 Post-construction evaluation and monitoring

10.1.1 General

Post-construction monitoring may be considered important in two specific ways, i.e.:

- confirming that the works are performing correctly and that any performance criteria have been satisfied; and
- providing information on the performance and design of the works which may be of value in future projects.

However, careful consideration should be given to the need for any post-construction evaluation and monitoring given implications for any future remedial works and any regime should be planned to be both simple and robust.

10.1.2 Evaluation of embankments/fills

10.1.2.1 Methods of evaluation

It may be necessary to monitor both the settlement and stability of the fill and underlying foundation after construction subject to the form of specification adopted and the anticipated long-term performance of the fill and /or foundation. Any requirements should be clearly set out in the Geotechnical Design Report and the contract documentation e.g Appendix 6/12 in SHW [1]. See also BS EN 1997-1:2004, Section 4.

The extent and method of monitoring depend on the nature of the ground, the significance of the project/fill and the accuracy required. This may range from simple levelling to a comprehensive range of instruments installed to measure accurately the deformations occurring in the embankment and foundation.

Any trigger levels and associated action plans should be clearly set out in the Geotechnical Design Report. The duration and scope of any post-construction monitoring should be re-assessed following observations made during construction and the results should be evaluated and interpreted on a regular basis.

10.1.2.2 Monitoring techniques

The following techniques are commonly employed on site:

- surface levelling stations to measure settlement of the fill surface;
- settlement plates to measure settlement of the fill thickness;
- magnetic extensometers to measure settlement at incremental depths within the fill;
- settlement gauges to measure settlement of the underlying foundation;
- piezometers to measure the water level within the fill; and
- inclinometers to record any lateral movements within the fill/foundation.

Further information on available techniques can be obtained from standard publications such as Dunnicliff [21] and from instrumentation suppliers.

10.1.3 Monitoring of slopes

10.1.3.1 General

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

- a) *Deformation*. Settlements in the upper part of the slope and bulging towards the toe may indicate incipient failure by a rotational shear slide (see Annex A).
- b) *Cracking*. A series of cracks in the vicinity of and sub-parallel to the crest of a slope may indicate sliding, as do en echelon cracks at the lateral boundaries of incipient movement. Hexagonal or random pattern cracking indicates drying shrinkage.
- c) *Fissuring*. Opening of joints and fissures in a rock slope indicates incipient translational or toppling failure (see Annex A).
- d) *Seepage*. Water carrying soil particles seeping from a slope face indicates internal or seepage erosion (see Annex A).
- e) *Gullying*. Channels eroded on a slope face indicate the need for protection against surface erosion.

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

Where there are concerns about the short or long term stability of cutting slopes it may be desirable to install instrumentation to give warning of incipient instability, to enable remedial measures such as the installation of drainage, grouting or anchoring to be undertaken before the stage of failure is reached.

NOTE Suitable methods of monitoring slopes are described in 10.1.2.2.

10.1.3.2 Water pressure

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

In homogeneous permeable soils pore pressures may be monitored by plumbing water levels in simple standpipes (see BS 5930:1999+A1). In layered soils or soils of moderate to low permeability the response time of standpipes to changes in pore pressure may be considered inadequate to detect critical conditions in sufficient time to take remedial action. In these cases pore pressures should be monitored by properly sealed and protected piezometers (BS 5930:1999+A1) with their tips located in each critical soil layer at a number of locations along the slope. Water levels in the piezometers may be monitored by plumbing down the riser pipe or by connecting a series of piezometers to a gauge house by means of pneumatic, hydraulic or electrical transmission and recording systems (see Dunnicliff [22]). Precautions should be taken against damage to a piezometer installation from construction and maintenance operations and from the effects of frost and vandalism.

10.1.3.3 Monitoring surface and sub-surface movements

Monitoring of ground surface movement in both horizontal and vertical planes may be carried out by field survey methods. The particular methods used should depend on the accuracy required.

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

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

- a) precise levelling using a geodetic level and Invar staff;
- b) triangulation using first order theodolites (reading to one second of arc);
- c) trilateration with special EDM equipment.

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

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

10.2 Incorporation of GFR into health and safety file

The geotechnical feedback report (see 6.5) should be incorporated into the project Health and Safety File along with design reports, as-built drawings, etc. This enables the asset management process to start.

For simple projects feedback may be limited, apart from inclusion in the Health and Safety File.

NOTE The relevance of retaining or distributing further information will depend on the scope, extent and future maintenance requirements of the project.

10.3 Formal handover

All relevant information on the earthworks covering ground investigations, design, construction and monitoring should be assembled prior to completion and formally handed over to the asset owner's maintenance team.

11 Earthworks asset management

11.1 General

Any earthworks should be managed by an appropriate asset management system to ensure that acceptable performance is achieved and that the earthworks do not present a risk to users. The nature of the management system may vary to reflect the use of the earthworks and the risks they pose.

The key texts on asset management to which reference should be made are CIRIA C591 [29] and CIRIA C592 [63]. The recommendations of these CIRIA documents are relevant to all owners of earthworks who should manage their assets in accordance with these recommendations and link as appropriate with adjacent land users.

COMMENTARY ON 11.1

Since the 1990s the large infrastructure operators (road, rail, waterways, flood defences) have implemented earthworks asset management systems, and it was these groups that were actively involved in the preparation of the key documents on the asset management of infrastructure earthworks,

Given the significance of the CIRIA documents to this clause, Table 11 is provided to aid cross reference. This clause gives commentary and recommendations on the subject, but the reader is advised to see CIRIA documents for more detail.

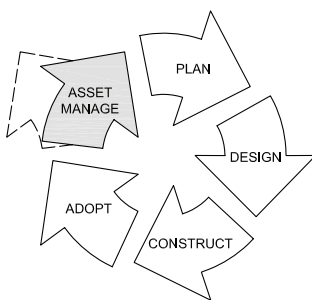


Table 11 Relevant sections of CIRIA C591 and C592

| BS 6031 subclause | Aspect | C591, cuttings | C592, embankments |
|-------------------|---------------------|------------------|----------------------------|
| 11.2 | Asset management | 2.3 to 2.6 | 2.3, 2.4, 2.6 |
| 11.3 | Inspection | 4.1 | 4.1 to 4.6, Appendix A3 |
| 11.4 | Assessment | 4.2 to 4.4 | 4.7 to 4.10 |
| 11.5 | Fault mitigation | — | — |
| 11.6 | Monitoring | 4.4.4 | 4.8.3 |
| 11.7 | Maintenance issues | 5.8, 7.5 and 7.6 | 5.8, 7.5, 7.6, Appendix A6 |
| 11.8 | Renewal works | 5 and 6 | 5 and 6 |
| 11.9 | Systems integration | 2.2.3 | 2.2.3 |
| 11.10 | Climate | — | — |

11.2 Asset management systems

An asset management system should include:

- a) asset catalogues;
- b) specification of performance and required duty;
- c) management and business strategies/plans;
- d) risk registers;
- e) consideration of whole life costs.

For asset management to be successful an accurate catalogue of assets should be established so that all subsequent activities can be planned and correctly referenced.

NOTE GIS database systems with a map-based front end have proved to be a valuable tool to assist in the task of delivering asset management. Systems are set up to suit the asset type and specific owner's requirements such as risk issues that are to be managed.

11.3 Inspection

For an earthworks asset management system to be successfully delivered it is necessary to develop an appropriate earthworks inspection system that should then be routinely implemented. The inspection approach varies between earthwork owners: in all cases it should involve an observation and recording of the site conditions at the time of the inspection. The following factors should be considered in developing an inspection regime:

- inspection frequency;
- types (cyclical or ad hoc);
- data collection techniques;
- uses.

The frequency of cyclical inspections should be set considering the age and condition of the infrastructure together with the likelihood, mode, consequence and remaining time to failure.

NOTE In all inspection systems there will be a trade off between the cost of undertaking the inspections (i.e. the level of detail required) and the benefit to be gained for managing the asset. The concept of “touch distance” is valuable, where inspections are undertaken within touching distance of the asset the accuracy of the data collected will be greater; however, in some cases data capture at a distance can form a valuable part of an asset management system.

Where touch distance inspections are not carried out the asset maintainer should manage the risks of potentially missing information. The asset maintainer should select a system that suits their requirements and ensures that the data collected is adequately assessed.

Where possible a record should be made of the drainage system as an integral part of the earthworks inspection and assessment. The inspection should also establish the presence and overall condition of any additional stabilizing structures (retaining walls, soil nails etc.) within the earthwork. Some of the stabilizing structures might require inspection by other disciplines and so harmonized inspection programmes should be considered.

The inspection should not only focus on the specific asset being inspected but should be watchful of nearby activities that may adversely affect the asset. The information collected from inspections should be used to identify faults and monitor asset condition and degradation rate.

11.4 Assessment

The assessment process should involve viewing one or more inspection records, considering the significance of the data and assessing the appropriate engineering response.

Assessments should normally be carried out on earthworks where little information is available on the original construction, but they may be applicable to more modern structures where conditions might have changed, e.g. groundwater profiles. They may also be applicable where assets are to be modified or subject to revised usage.

NOTE Geophysical techniques can have benefit in profiling long linear structures or in establishing the scale of damage from burrowing animals. Further information may be obtained from CIRIA C591 [29] and C592 [63].

11.5 Fault mitigation

If instability or other loss of functionality occurs, the following options may be adopted in increasing order of impact:

- a) increased inspection;
- b) monitoring;
- c) routine maintenance, e.g. unblock drains; that is, minor works (no need for design normally as the works are standard/repeatable activities and there is no change to the earthwork proposed); this is often referred to as “preventative maintenance” as investment in this form of maintenance ought to prevent the onset of slope instability; these activities are ideally undertaken by the maintainer as part of a programme of routine maintenance;

- d) service restriction, either while mitigation is put in place or stabilization is undertaken;
- e) remedial works if significant intervention like an earthworks fix is needed (e.g. stabilization of an embankment); which means a return to the design stage, as illustrated in Figure 1; at this point a designer is needed as the works will be one-off activity designed to suit the particular site; these activities are often referred to as "reactive" (or proactive) maintenance;
- f) withdrawal from service.

11.6 Monitoring of existing earthworks

COMMENTARY ON 11.6

The purpose of monitoring earthworks (see Clause 6 for detail) includes the following:

- a) *where movements have occurred, gain design information on the depth area and lateral extent;*
- b) *determine the rate of movement and establish if it is constant, accelerating or decelerating;*
- c) *establish in-situ water pressures and any variation over time;*
- d) *mitigation of risk until repairs are implemented;*
- e) *to establish the effects of adjacent construction or modification to earthworks both during the construction stage, and in the long term;*
- f) *obtain quantitative performance data where analytical assessment is believed to have produced conservative results; and*
- g) *confirm design assumptions.*

The designer of the monitoring system should define these aspects in advance of implementation of the scheme:

- 1) objectives;
- 2) techniques;
- 3) required accuracy;
- 4) frequency;
- 5) trigger levels;
- 6) data collection and reporting; and
- 7) action plans.

NOTE There is a vast range of instrumentation available and further guidance can be obtained from [21]. Recent developments in this field include web based, cable free or fibre optic systems which offer remote, real time monitoring that has the potential to be integrated to automatic alert alarms. Owing to the rate of technological development in this field discussion should always be held with specialist instrument manufacturers and installers.

In establishing trigger levels, the likely magnitude of normal background movements should be considered. (For example, seasonal vertical movements of up to 50 mm have been recorded on some London Underground embankments due to effects of vegetation on high plasticity clay.) Therefore the monitoring system should be put in place sufficiently in advance of control periods so that the range and pattern of background movements can be established.

For new or modified construction, monitoring can have a role to play in finalizing design through either the approach recommended in BS EN 1997-1:2004 (see 7.2.4 of this standard) or the observational approach itself (see [21]).

11.7 Maintenance issues

11.7.1 General

During the original construction, designers should have considered potential maintenance issues, and aimed for solutions that minimize the maintenance requirements and result in easy to maintain earthworks. When this approach has not been taken there is likely to be an increased risk of earthworks failures; however, in practice all earthworks should undergo routine maintenance in order to achieve the intended design life. Aspects of maintenance should include:

- drainage – surface water and earthworks drainage;
- vegetation (leaves, tree falls, slope stability issues, seasonal movement on clay soils), see Scott et al [64], CIRIA C591 [29] and CIRIA C592 [63];
- animal damage (rabbits, badgers, etc.); those responsible for maintenance should be aware of the relevant legal obligations; see *The Management of Problems involving Badgers* [65];
- prevention of further erosion;
- scaling of rock slopes.

Good records of maintenance works undertaken as part of the asset management process should be maintained.

11.7.2 Drainage

NOTE 1 A description of drainage systems for slopes is given in 7.5.3.

Regular inspection should be undertaken to ensure that the drains are working effectively and that they are not becoming silted up or blocked as a result of pipe fractures or slope deformations. Where access to a drainage system is available through manholes, the manhole covers should be lifted at regular intervals, silt traps cleaned out and pipes examined for blockage and rodded and flushed as necessary.

A watch should be kept for infiltration of soil into open or closed joint piped drains, and remedial measures taken if there are signs of appreciable internal erosion of soil into the pipes, or indeed if they are not carrying any water at all; in this latter case it could be that water is being discharged into the slope at some point where the drain is broken and this might present a threat to its stability. In addition, water seeping out of the ground might also indicate a pipe fracture; the fracture should be located and repaired with as little delay as possible.

Special inspection should be made at times of heavy rainfall to check whether or not any of the drains are surcharged or are carrying eroded soil.

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

NOTE 2 Coarse backfill to drains can become clogged in time and require replacement. When this is done the insertion of a filter fabric to surround the backfill can keep clay and coarser particles out of the collector system.

11.7.3 Vegetation

Vegetation should be managed to prevent trees encroaching onto the earthworks or obscuring signage, lighting or signalling equipment.

COMMENTARY ON 11.7.3

Trees also require management because they can give rise to seasonal movements of clay earthworks that can adversely effect the stability and performance of structures founded upon them including road pavements, rail track and services. There is also evidence that successive cycles of movement can, in the long term, give rise to a reduction in the mass strength of the earth structure possibly leading to failure [69, 53, 55].

11.7.4 Animal damage

Burrowing animals should be controlled as damage from burrowing activity can undermine the stability of an earthworks and where severe enough can lead to collapse particularly where loading is applied close to the burrows or water retaining structures are involved.

Certain animals such as badgers are legally protected and appropriate procedures should be followed in dealing with them [68].

11.7.5 Erosion

Attention should be given to the potential for erosion due to action of water (drainage, heavy rainfall, water courses and floods, etc.) on the earthworks [70]. In particular, drainage should be maintained and where appropriate, upgraded to accommodate changes in adjacent land use.

When water stands against earthworks, soft or loose material can in time be eroded by wave action and this action should be considered separately from general flood conditions.

COMMENTARY ON 11.7.5

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

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

Existing surface protection can be loosened by suction action of the waves. In tidal waters the constant raising and lowering of ground water levels can cause migration of soil particles from slopes.

Closely related to water as an eroding agent is frost. Alternate freezing and thawing loosens the surface of rock cuttings and opens cracks and fissures that could have been caused by the construction processes or occur naturally.

11.8 Renewal/remedial works

Reference should be made to CIRIA C591 [29] and CIRIA C592 [63] for guidance on the selection of options for renewal works. Reference should also be made to 7.7.2 of this standard, thus re-initiating the earthworks design, construct, adopt and manage cycle.

11.9 Systems integration

Earthworks can carry a number of different infrastructure elements such as services, drainage or signage and integration should be implemented to ensure that all required duties are met. It is important to co-ordinate all maintenance activities so that best use is made of time and the works are integrated to prevent damage by piecemeal working.

Asset managers should be kept informed of and approve all activities to be undertaken involving the earthwork, as it is possible for overall performance to be adversely affected by the activities of others, e.g., an excavation for a service at the toe of the slope could trigger slope movement.

Damage to assets, either reported directly or identified through inspection, should be logged and tracked so that common causes can be identified and recurrences can be prevented.

11.10 Climatic factors

11.10.1 General

COMMENTARY ON 11.10.1

The stability and performance of earthworks, particularly those in a poor condition, is often related to climatic events. The principal driver is rainfall either in the form of short-lived storm events, generating rapid flow failures, or longer lasting events which will modify ground water profiles. Another important factor is evapotranspiration which removes moisture through either direct evaporation or uptake and transpiration from vegetation. Evapotranspiration will have a positive effect on stability by reducing pore water pressures but, in the case of embankments formed of high plasticity clay, can lead to unacceptable seasonal deformations of the supported infrastructure.

Understanding the connection between climatic events and the performance of earthworks enables trigger levels to be selected in order to provide a warning of impending performance reduction; appropriate mitigation measures such as increased inspection, monitoring, maintenance or service restriction should then be implemented.

Climate change should also be considered as future climatic events might be different from those that earthworks have typically experienced to date. Therefore, it is important to understand the current climatic trigger levels that impact earthworks performance so that the significance of likely future changes may be determined.

11.10.2 Rainfall

Rainfall can be measured as a total quantity within a set period of time, either annually to enable comparison between different climatic patterns, or over the short term, say mm/hour, to indicate the intensity of a particular event. The latter approach can be linked to statistical analyses to assist with the selection of return periods for design storm events. However, in order to understand the overall impact of rainfall on earthworks a cumulative approach should be adopted. This entails comparing rainfall over a particular time period, such as a specific week or a month, with the long term statistical average for that same time period.

11.10.3 Soil moisture deficit

COMMENTARY ON 11.10.3

The soil moisture deficit (SMD) is a means of quantifying the combined effects of rainfall and evapotranspiration. It represents the cumulative reduction in soil moisture content below field capacity as evapotranspiration exceeds rainfall. In simple terms it represents the amount of rainfall (in mm) required to bring the soil to its full capacity. Any further precipitation would either pass through the profile, run off or pond. Thus the SMD is higher in the summer as evapotranspiration exceeds rainfall and low or even zero during the winter periods as rainfall increases and moisture uptake from vegetation decreases. As the SMD takes into account the effects of rainfall, evaporation and vegetation it represents a useful indicator for linking climatic events to the performance of earthworks.

A comparison of SMD values for the London area with the occurrence of slope failures over the period January 1988 to January 2001 was undertaken by Ridley et al [68]. It demonstrated that slope failures occurred more frequently during the winter period when SMD values were low and that the calculated values for deciduous trees appear to correlate better with the failures than those for grassed sites. Seasonal movements on clay soils are more problematic in the summer months and that not all summers are equal in this respect. Therefore, SMD values should be a good indicator of the likelihood of seasonal movement adversely affecting earthworks.

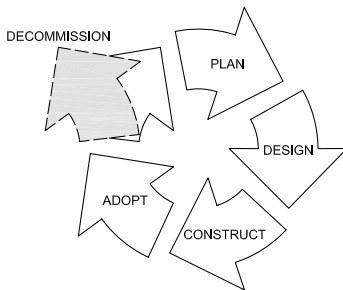
The Meteorological Office can provide SMD data on a regional basis through MORECS (Met Office Rainfall and Evaporation Calculation System). The information is provided on a weekly basis for a grid of 40 km by 40 km MORECS squares for both grass and tree covered sites. Obviously a certain amount of averaging takes place over each square but it is nevertheless a useful index that is readily available and can also be used to investigate historic events.

11.10.4 Climatic indicators

From the information in 11.10.1, 11.10.2 and 11.10.3, climatic indicators may be based on both rainfall and SMD that should be used to predict the performance of earthworks both in terms of slope failure and seasonal movement.

However, care should be taken in adopting trigger levels developed in one region for another as local variations can occur.

In order to properly manage risks a schedule should be developed of earthworks at risk from climatic extremes so that appropriate mitigation can be put in place as trigger levels are exceeded.



12 Decommissioning and disposal of earthworks assets

12.1 General

The removal of earthworks should not normally present significant health and safety risks.

12.2 Decommissioning

Major infrastructure owners are likely to have their own established procedures for the decommissioning of assets, however, in the absence of such guidance the following aspects should be considered in the design and implementation of decommissioning.

- a) *Stability.* Shallow surface movements of decommissioned slopes are likely to be acceptable provide that there are no safety implications. Deep seated movements should be prevented if there are any safety implications or obligations to third parties.
- b) *Maintenance.* The need for any ongoing maintenance operations should be identified.
- c) *Inspection.* The need for any ongoing inspection requirements should be identified.
- d) *Access.* It should be decided if the asset is to remain accessible after decommissioning and what measures are to be taken to prevent unauthorized access.
- e) *Systems integration.* Consideration should be given to the identification and protection of services or interfacing structures, etc., that might pass through or about the decommissioned asset.

12.3 Disposal

This is likely to be driven by property disposal considerations but all information relating to the earthworks should be made available to the new owners.

12.4 Partial removal

Where the partial removal of existing earthworks is a part of a new earthworks project, a comprehensive search should be made for records relating to the existing earthworks. If available these should form part of the information on which the design of the new earthworks is based.

Where technically possible, surplus material from the partial removal operations should be incorporated into the new earthworks.

Section 3: Temporary excavations, trenches, pits and shafts

13 Temporary excavations

COMMENTARY ON CLAUSE 13

This section covers the design and construction of temporary excavations with vertical or near vertical slopes which require some form of excavation support to be stable. Temporary excavations which are designed to be self supporting (with soil reinforcement e.g. soil nails, where necessary) are covered by Section 2.

13.1 General

Procedural controls of temporary works for excavations should be undertaken in accordance with BS 5975, Section 2.

All temporary excavations should be provided with safe means of access and egress including means of escape in an emergency. There should be adequate working space in the excavation along with walkways and ladders between working areas where necessary. All temporary excavations should be subject to an assessment of the risks involved as described in 4.2. This should be in proportion to the complexity of the excavation being undertaken.

The upper perimeter of the excavation should be adequately guarded to protect persons both in and around the excavation and to prevent vehicles from falling in.

NOTE 1 Temporary excavations can be confined spaces depending on their layout (see Confined Spaces Regulations ACoP [69]).

NOTE 2 Further guidance on safe working in excavations can be found in HSE publication HSG 185 Safety in Excavations [70].

13.2 Design considerations

The design of the excavation support system should take into account:

- the extent and nature of the works to be undertaken in it;
- the extent and nature of the support system;
- ground and groundwater conditions;
- requirements for adequate working space and clearance between supports within the excavation;
- the method and sequencing of backfilling along with the removal of the support system; and
- plant available for construction of the excavation.

13.3 Site investigation

Site investigation for temporary excavations should be undertaken and reported on, in accordance with the principles in Clause 6. The investigation should be in proportion to the complexity of the temporary excavation and its support system, prevailing ground conditions and the sensitivity of adjacent structures to disturbance.

As part of these investigations, ground water levels should be monitored over as long a period as possible to assess seasonal or tidal variations which could be relevant and to eliminate misleading data arising from the monitoring process.

13.4 Ground conditions

13.4.1 General

Knowledge of the ground conditions gained from the site investigation should be used:

- to determine the appropriate method of excavation and related plant requirements;
- to determine the appropriate form of support to the sides of the excavation and to ensure its adequacy;
- to determine suitable means of maintaining the excavations free from ground water;
- to ensure that any potentially buoyant structures which might be constructed within the excavations will not be subjected to water pressures sufficient to cause uplift forces at any stage of their construction.

13.4.2 Influencing factors on construction methods

The following factors should be considered when determining the methods of excavation and excavation support system required:

- the purpose of the excavation;
- the ground profile;
- short term stability of excavated faces;
- geotechnical factors (see 13.4.3);
- the depth and extent of the excavation;
- groundwater table and fluctuations in its level;
- variability in ground permeability, including the risk of piping/artesian conditions;
- adjacent structures and services; and
- the driveability of sheet piles or other lining systems.

13.4.3 Geotechnical factors

Geotechnical factors affecting the safety of the excavation which should be considered include:

- the nature of the ground;
- the short- and long-term soil strength;
- the ground water regime;
- the presence of rock, and discontinuities of the rock mass;
- the presence of made or previously disturbed ground.

NOTE More comprehensive recommendations and guidance are given in Clause 7.

13.4.4 Sources and control of ground water

Consideration should be given to controlling water which can adversely affect the stability of an excavation including:

- rainfall and surface runoff;
- shallow subsoil water;
- land drainage;
- inflow from below the ground water table;
- artesian conditions;
- inflow from damaged services.

The provision of dewatering, drainage or pumping schemes should be considered as appropriate to ensure the excavation remains suitably dry. Likewise dry conditions can and may be achieved by the construction of an impermeable curtain around the excavation. The effects of such activities on the local groundwater regime and adjacent structures should be considered.

13.5 The design of stable slopes and supports to excavations

13.5.1 General

For routine excavations, in stable ground conditions, information on the loads to be resisted or the load resisting capabilities of the excavation support system should be available from suppliers' literature or websites or by reference to standard texts such as CIRIA R97 [71].

In other situations geotechnical engineering principles using strength parameters obtained from site investigation work should be used to design the excavation support system in accordance with accepted procedures.

13.5.2 Magnitude and distribution of lateral soil pressures

Lateral pressures on the support systems of excavations should be calculated from first principles (allowing for soil structure interaction) or in the manner described in CIRIA C517 [72].

13.5.3 Stability of base of excavation

13.5.3.1 General

Consideration should be given to preventing base failure in deep excavations from:

- uplift or "boiling" of granular soils due to large seepage forces caused by high hydrostatic heads; or
- by heave or shear deformation of soft saturated cohesive soils due to overstress.

13.5.3.2 Water-bearing permeable soils

Ideally, sheet piles supporting an excavation below the ground water table in granular soils should have a sufficient depth of penetration below the base of an excavation to reach an impermeable stratum and thus provide a cut-off to the flow of water beneath the toes of the piles preventing upward seepage at the base of the excavation.

Where this is not possible sheet piles should be driven to a prescribed depth of penetration below the base of the excavation to restrict inflow of ground water.

Care should be taken to ensure the head of water outside the sheet piling is not sufficient to cause a sufficiently steep hydraulic gradient over the length of the seepage path to give a velocity of upward seepage which could cause instability of the soil particles.

The design of deep excavations in water-bearing granular soils should also take into account that boiling can occur as a result of strong flow from a permeable layer underlying less permeable soil at the base of the excavation.

If necessary, ground water lowering methods should be used to lower the external head of ground water, or alternatively the sheet piling should be driven to a deeper penetration to lengthen the seepage path so decreasing the hydraulic gradient and thus reduce the tendency of the ground to boil.

13.5.3.3 Soft cohesive soils

Failure by heaving can occur in deep excavations in soft cohesive soils through overstressing of the soil in the region of the base of the excavation. Conventional methods of stability analysis should be used to predict the likelihood of base failure by shear deformation. It is recommended that a conservative approach is taken to setting the characteristic strength of the soil.

13.5.3.4 Movements at base of excavation

The following should be considered when determining the magnitude and rate of upward movement at the base of an excavation:

- a) the reduction in vertical stress caused by the removal of soil from within the excavation;
- b) the nature of the strata underlying the base of the excavation;
- c) the ground water conditions;
- d) upward movements which take place are caused by immediate elastic strain which occurs simultaneously with the deepening of the excavation and by long term volumetric strains due to moisture content changes; in stratified cohesive deposits which display high horizontal permeabilities, heave caused by volumetric strain can be rapid;
- e) variations in the magnitude of upward movement which are generally greater at the centre of the base of the excavation than at the periphery; the magnitude of heave may be predicted by elastic theory but the rate of heave cannot be reliably predicted on the basis of theory and few field measurements have been made.

13.6 Practical considerations

13.6.1 General

There are various practical considerations relating to excavations which should be considered before and during the excavation work.

13.6.2 Methods of excavation and types of support

COMMENTARY ON 13.6.2

No matter what method of excavation is used, ground displacements occur both within and immediately surrounding an excavation. These ground displacements depend partly on the geological structure and are principally due to elastic strains. In cohesive soils, volumetric strains due to changes in moisture content also take place. If the method of excavation and the type of support are unsuitable for the particular ground conditions, then shear deformations or shear failures of the soil or failures due to hydrostatic pressures can occur. Vibrations from construction equipment can cause consolidation of cohesionless soils or have a detrimental effect on existing structures in a weak condition. The sequence of excavation and installation of lateral supports has a significant effect on the stresses and strains induced in the ground.

It may not be practicable to prevent significant vertical and lateral ground displacements immediately beyond the limits of an excavation, although careful design of the support system will help to minimize displacements. The effects of the inevitable movements on any adjoining structures should be considered. It may be necessary to underpin adjoining structures before commencing an excavation, in order to protect them from the ground displacements. Alternatively, and with the agreement of their owners, damage to the adjoining structures may be accepted which should be repaired after completion of the permanent work. However, the damage should not be such as to cause any danger to the occupants of these structures, or to the general public. A simple construction procedure is desirable since alterations to a complex construction sequence, when unexpected variations in ground conditions are encountered, is often difficult. The work should not be undertaken without experienced supervision, and inspection should be made several times each day to ensure that stable conditions are being maintained.

Narrow trenches may sometimes be excavated with unsupported vertical faces, depending on rate of construction, soil type and strength and depth of trench excavation. It is essential that trenches are supported where people are required to enter them. Stability conditions should be regarded as unfavourable even in firm and stiff clays and in fissured and closely jointed rocks.

It may be economical to incorporate support systems such as steel sheet piling, concrete diaphragm walls, or contiguous bored pile walls in the permanent construction.

No lateral supports for any part of an excavation should be altered or dismantled except under the direction of the designer or a competent person possessing adequate knowledge and experience. Where an observational method is being used, a store of suitable materials should be kept on site to provide immediate strengthening, if found necessary.

13.6.3 Existing buildings, buried structures and services

The age, types of construction and the type and depth of foundations of existing buildings which would be affected by the excavation should be ascertained before commencing work on site. An appraisal of the dead and superimposed loads from the foundations of existing buildings should be made since the stability of the excavation can depend on an accurate prediction of imposed loading on the selected system of retention for the excavation. All buildings and buried structures which are likely to be affected by the excavation work should be surveyed with the representatives of the owners, and a report on the condition of the structures prepared. The report should contain photographs of any building defects. Significant structural cracks should be instrumented. Appropriate instrumentation should be installed to permit any building movements to be monitored; however, it is advisable to include simple strain gauge devices and levelling points.

Where ground anchors used for excavation supports pass beneath existing buildings and infrastructure, the effects of the drilling and grouting processes used to install the anchors should be considered. The permission of the land owners should be obtained for the installation of anchors beneath their property.

The design and construction of excavations and their retention systems should also take into consideration the prior location and safe support of all services such as water and gas mains, and buried structures such as underground tunnels and sewers.

13.6.4 Disposal of spoil

13.6.4.1 General

General recommendations and guidance on waste management is given in 5.4.2. Arisings from trench excavation should be dealt with in the same way as other earthworks spoil.

13.6.4.2 Temporary spoil and material heaps

Temporary spoil and material heaps should be sited to interfere as little as possible with the work to be carried out. Whilst, for convenience in handling, it might be necessary to place them near excavations, the following points should be borne in mind:

- a) they should not interfere with free access to the excavation (in trench work it is desirable to place the material which is to be used for backfilling on one side of the trench only);
- b) they should be so constructed that there is no danger of the spoil slumping in wet weather and entering the excavation;
- c) spoil heaps should not be placed in such a position as to endanger the stability of existing works above or below ground or of the excavation, the sides or side supports of which should be so designed as to be capable of withstanding the additional stresses due to any superimposed load.

Spoil heaps should be graded to safe slopes taking into consideration the nature of the material and the effects of wet weather. With coarse sand or clean gravel the natural angle of repose of the tipped

materials should remain substantially unaltered in wet weather, but with materials that soften and slump, e.g. clays, silts, mudstones, etc., a substantial reduction in slope should be anticipated and an adequate distance maintained between the periphery of the spoil heap and the edge of excavation.

The clearance between the toe of the spoil heap and the edge of the excavation should give sufficient working space at all times, and for this purpose the clearance should be a distance equal to the depth of the excavation with a minimum width of 1.50 m.

14 Construction procedure

14.1 Support of temporary excavations

14.1.1 General

Temporary excavation support systems should be selected, for one or more of the following purposes:

- the protection of persons within and around the excavation;
- the control of movement of the ground around the excavation perimeter; or
- to minimize the excavated area.

14.1.2 Performance criteria

When selecting and designing ground support systems the following should be considered:

- the capability of the chosen system to prevent water ingress through the sides and/or the base of the excavation;
- the capability of the chosen system to accommodate service crossings, variations in plan layout of the excavation and stop ends;
- access into the excavation and the provision of adequate working space in the excavation;
- where necessary, the potential for installing, supporting and removing the support system, incrementally as work progresses;
- installation of the support system so that operatives are always working from within a protected area;
- the use of ground anchors in place of strutting and propping to reduce the potential obstructions within the excavation;
- soil nailing incorporating geotextile mesh as necessary to support earth faces (see CIRIA C637 [73] and BS 8006-2);
- the means for handling structural members particularly in wide and/or deep excavations, during installation and removal, and for supporting them during use; and
- provision of a suitable upstand or barrier to reduce the risk of falls into the excavation.

Suppliers of support systems, their literature and websites should be consulted for information on the types, performance capabilities and capacities of systems which are available; systems may include trench boxes to BS EN 13331-1 or be of traditional construction utilizing

materials such as timber, steel sections or precast reinforced concrete. BS 6031:1981 should be referred to for guidance on the use of timber as the dominant structural material.

14.1.3 Buildability criteria

There are a number of issues relating to the buildability of the support system which should be considered:

- means and sequence of installation;
- maintenance of the system when in use;
- backfilling requirements;
- ease of striking parts or all of system without compromising safety;
- protection against falls of system components due to unintended removal of load;
- temporary support and reinstatement of service crossings.

NOTE Practical guidance on these issues can be found in HSE "Safety in Excavations" HSG 185 [70], BS 6031:1981 and suppliers' literature and websites. BS 6031:1981 is obsolescent, but is considered to contain important information on timber support, and is still available from BSI.

14.1.4 Inspection criteria

NOTE There is a legal duty to inspect the support system to ensure its continuing integrity in use.

An inspection should identify:

- signs of overstressing, movement or loosening of support members;
- excessive continuing deflection;
- mechanical damage;
- abnormal ingress of water or ground;
- any monitoring system(s) relevant to the stability of the excavation or adjacent structures; and
- fluid loss, where proprietary hydraulic systems are employed.

14.2 Temporary support by permanent structure

Consideration should be given to using components forming part of the permanent structure as support for temporary excavations: such components include diaphragm walls and secant or contiguous piled walls for ground support along with floor slabs to support the walls.

15 Trenches

15.1 Construction methods

When designing a trench, consideration should be given to the following factors which influence the method of excavating, supporting and backfilling it:

- purpose and location of the trench;
- size of the trench including due allowance of access, egress and adequate working space for construction of the permanent work;

- the length of trench to be open at any one time and the period for which it is to remain open; no trench should be left open longer than necessary;
- nature of ground including information from any trial pits and intermediate boreholes, etc.;
- avoidance of excavation in loose or made ground or alongside earlier backfilled excavations, all of which circumstances require extra care;
- removal of ground water (see 13.4 and 15.5);
- statutory obligations (see 15.2);
- obstructions above and below ground level;
- buried services.

15.2 Statutory obligations

General recommendations and guidance on the statutory bodies and infrastructure owners that should be consulted in advance of excavation works being undertaken are given in Clause 5.

The requirements of the New Roads and Streetworks Act 1991 [74] including requirements for backfilling and reinstatement should be taken into account.

15.3 Excavation procedure

15.3.1 Trenchless techniques

The use of trenchless techniques should be considered as a minimally disruptive alternative to open trench excavation. Such techniques should also be considered when obstructions on the line of trench cannot be disturbed, or where the depth is too great for open trenching to be cost effective or practicable. (See CIRIA SP147 [75].)

15.3.2 Methods available

The trenching techniques including excavation equipment and support regimes should be compatible with the purpose of the trench and the environment in which the trench is excavated.

15.3.3 Trenches with vertical sides

When excavating a trench with vertical or near vertical sides and for which it is intended to provide additional support, the following should be considered.

Support should be provided for all vertically sided trenches of any depth where ground conditions so require it, including shallow trenches where operatives are required to work kneeling.

Support systems should be selected and installed in a manner that does not involve risks to operatives due to instability of the sides of the trench. Details of various methods for supporting vertical sides or providing a protected working space are given in proprietary shoring manufacturers' literature or in HSG 185 [70].

15.3.4 **Bottoming of trenches**

Whatever the permanent work, some manual trimming can be necessary in the trench bottom. During this operation disturbance of the soil at formation level, particularly in clays, silts and fine sands should be minimized. Depending upon the purpose of the trench consideration should be given to laying gravel, broken stone or weak concrete as soon as the formation is exposed to form a protective layer over such soils. Groundwater should meanwhile be kept below formation level.

In rock, excavation should be taken down to below formation level and then uniformly compacted sand or other fine granular material, or concrete placed to produce a true bottom. Where the longitudinal gradient is steep, the material placed should be sufficiently coarse to resist erosion by a permanent flow of groundwater along the base of the trench.

15.4 **Hand excavation of trenches**

Only when it is not practicable to excavate mechanically should hand excavation be undertaken.

Given the variety of sizes and types of excavator now available and the availability of trenchless techniques, conditions necessitating large-scale hand excavation should be extremely rare but could include:

- a) ground too steep for a machine, or working space very restricted;
- b) road and railway crossings where a machine would interfere with traffic;
- c) sites where cables, mains, drains and other obstructions are known to exist;
- d) paved surfaces or lawns where damage to the surface by a machine cannot be tolerated;
- e) very bad ground which is incapable of supporting the weight of a machine.

NOTE The Manual Handling Regulations 1992 [76] apply to hand excavation.

15.5 **Methods of dewatering trenches**

Water should not be allowed to enter or accumulate in the bottom of a trench or excavation, and pumping or dewatering facilities should be provided to deal with ground water or surface water inflow. The disposal of water from trench excavations might require measures to prevent pollution of watercourses (see 7.5); CIRIA C515 [77] should be consulted for further information.

15.6 **Backfilling and reinstatement of surface**

Backfilling and compacting around any pipes or conduits should be done in accordance with the asset owner's requirements or a nationally recognized specification (see also Clause 8).

NOTE The New Roads and Streetworks Act 1991 [74] contains requirements for backfilling and reinstating excavations in highways.

16 Pits and shafts

16.1 General

Pits and shafts should be constructed on the principles described in Clause 12, Clause 13, Clause 14, and BS 6164.

16.2 Methods of support of excavations

Depending on the depth required, ground and groundwater conditions, etc., the following methods of support for pits and shafts should be considered:

- cantilevered or propped, contiguous or secant piling or diaphragm waling;
- cantilevered or propped, sheet piling;
- segmental concrete segments (placed by underpinning or caisson sinking techniques) or sprayed concrete;
- excavation support systems described in Clause 12;
- tubular steel casing;
- the use of ground anchors.

The excavation of pits and shafts should be carried out in accordance with the recommendations of this standard.

In addition, the recommendations of BS 6164 should be complied with.

Where the lining of pits or shafts is supported by ring walings or ground anchors; and is intended to be self supporting or to be supported without the use of props, consideration should be given to applying the "observational method" (CIRIA R185 [16]).

16.3 Wells and hand excavated piles

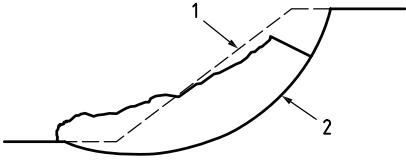
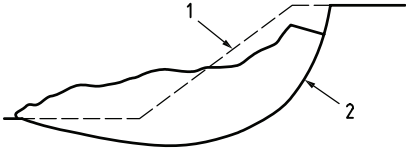
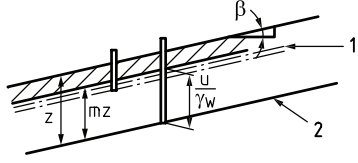
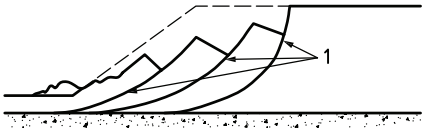
Wells and piles in inaccessible locations can be sunk as small diameter deep shafts by hand excavation techniques. The recommendations of BS 6164 should be strictly adhered to.

Annex A (informative) Potential modes of failure of slopes

Table A.1 illustrates the forms that slope failures often take when they exceed the ultimate limit state (the serviceability limit is not covered here but is just as relevant).

Definitions are given in Table A.1 from BS 6100-3 for different forms of landslip, the global definition of which is given as "03 25009 slip, landslip – movement of a mass of soil (01) or rock (03 23027) by gravity *NOTE Often a rotational displacement.*" The reference numbers within these definitions are a system established within the BSI system to enable cross referencing and therefore are included within this table.

Table A.1 Definitions of potential modes of failure of slopes

| Failure mode | BS 6100-3 definition (all numbers in brackets refer to definitions of terms within the BS) | Sketch |
|---------------------------|--|--|
| Rotational – Circular | 03 25010 rotational slide – rotation of a mass of soil (01) along a curved slip surface (03 27025) 03 25011 circular slide – rotational slide (03 25010) on a slip surface (03 27025) that is approximately circular |  <p>Key 1 Original profile 2 Slip surface</p> |
| Rotational – Non-circular | 03 25012 non-circular slide – rotational slide (03 25010) on a slip surface (03 27025) that is not wholly circular |  <p>Key 1 Original profile 2 Slip surface</p> |
| Translational | 03 25013 translational slide – movement of a shallow mass of soil (01) in a plane roughly parallel to the slope (01) due to a weakness on the plane For limiting equilibrium: $\gamma z \sin \beta \cos \beta = c' + (\gamma - m\gamma_w)z \cos^2 \beta \tan \phi'$ If $c' = 0$: $\tan \beta = \frac{(\gamma - m\gamma_w)}{\gamma} \tan \phi'$ |  <p>Key 1 Water table 2 Slip surface</p> |
| Compound | 03 25018 compound slide – movement of a soil (01) mass that combines the characteristics (01) of a rotational slide (03 25010) and a translational slide (03 25013) |  <p>Key 1 Slip surfaces</p> |

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Table A.1 Definitions of potential modes of failure of slopes (*continued*)

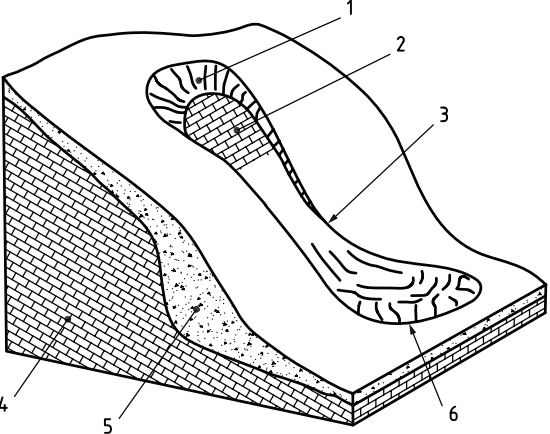
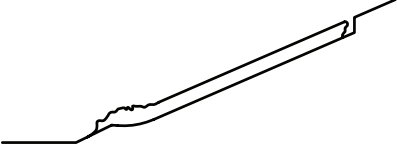
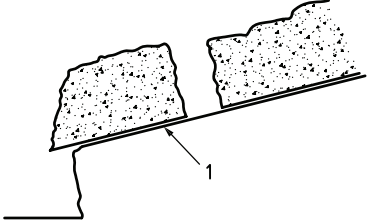
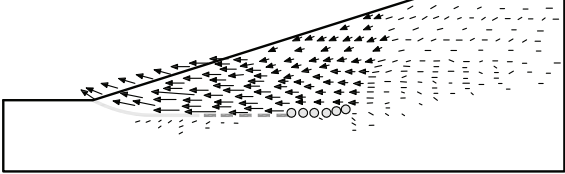
| Failure mode | BS 6100-3 definition (all numbers in brackets refer to definitions of terms within the BS) | Sketch |
|---------------------|--|--|
| Flow slides | <p>03 25014 flow slide, mud flow – translational slide (03 25013) in saturated soil (03 23021), caused by a sudden increase in pore water pressure (03 27011), in which the soil (01) flows as a viscous fluid</p> <p>03 25017 debris slide – translational slide (03 25013) of debris, forming a mantle on a slope (01) or the disturbed material at the toe of a rotational slide (03 25010), when rainfall (05 29004) or diverted surface water (01) causes downward movement of the debris</p> <p>Includes debris flows, see Winter et al [78]</p> |  <p>Key</p> <ul style="list-style-type: none"> 1 Scarp 2 Source area 3 Flow track 4 Bedrock 5 Superficial deposits 6 Run-out lobe |
| Slab slide | <p>03 25015 slab slide - translational slide (03 25013) in which the sliding mass remains more or less intact.</p> <p><i>NOTE</i> Usually occurring in the weathered (01) surface of a slope (01).</p> |  |
| Block slide | <p>03 25016 block slide – translational slide (03 25013) in which a block of relatively strong rock (03 23027) or stiff to hard clay (BS EN 12670) moves down a slope (01) as a unit</p> |  <p>Key</p> <ul style="list-style-type: none"> 1 Sliding surface |
| Progressive failure | <p>Progressive failure can occur in a mass of brittle soil when it is loaded non-uniformly. Failure first develops along a rupture surface or zone within part of the soil mass and as the post-peak strains within the failure zone increase, the soil strength within the failure zone reduces from peak towards residual. Final rupture of the soil mass occurs before the failure surface has developed fully.</p> <p>See Potts et al [79]</p> |  |

Table A.1 Definitions of potential modes of failure of slopes (continued)

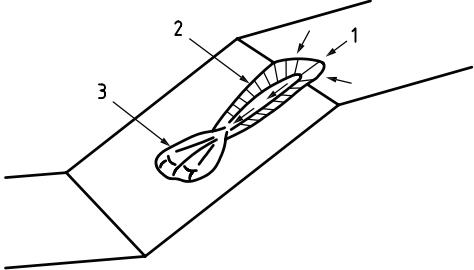
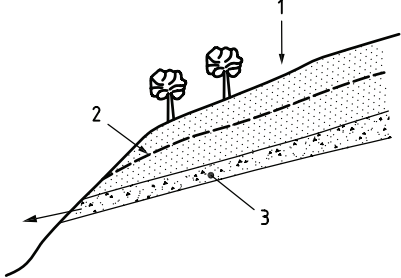
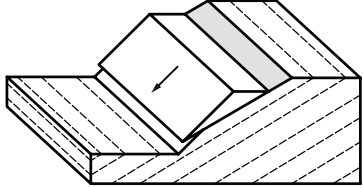
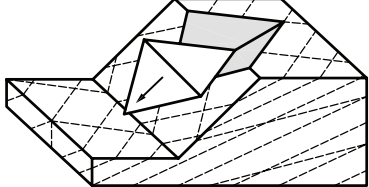
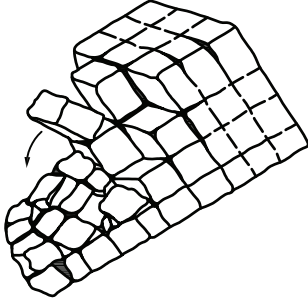
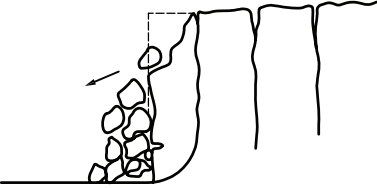
| Failure mode | BS 6100-3 definition (all numbers in brackets refer to definitions of terms within the BS) | Sketch |
|----------------------------|---|---|
| Scour | Removal of soil from the ground surface by surface water which might be flowing within a watercourse, or be in the form of floodwater or surface water run-off. Scour is a common problem for slopes, river banks or around structures. On slopes scour erosion can quickly lead to the development of gullies. Surface water run-off erosion of earthworks are also referred to as "washouts". |  <p>Key</p> <ul style="list-style-type: none"> 1 Concentration of surface water flow 2 Gully 3 Washed out soil |
| Internal erosion | Loss of soil from a slope face as a consequence of seepage of groundwater from a preferential flow path at the slope face (often referred to as piping), or the slumping of a saturated mass of soil promoted by water seeping through a slope (commonly referred to as slumping or sloughing). |  <p>Key</p> <ul style="list-style-type: none"> 1 Recharge zone, upslope 2 Piezometric pressure in confined channel 3 High permeability channel |
| Rock slope – plane failure | A plane failure 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. |  <p>Key</p> <ul style="list-style-type: none"> ← Direction of movement --- Discontinuity trace ▭ Slip plane |
| Rock slope – wedge failure | 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. |  <p>Key</p> <ul style="list-style-type: none"> ← Direction of movement --- Discontinuity trace ▭ Slip plane |

Table A.1 Definitions of potential modes of failure of slopes (*continued*)

| Failure mode | BS 6100-3 definition (all numbers in brackets refer to definitions of terms within the BS) | Sketch |
|--------------------------------|---|---|
| Rock slope – toppling failures | Toppling failures occur in rock slopes where discontinuities behind the face are steeply inclined. |  <p data-bbox="863 674 911 701">Key</p> <p data-bbox="863 712 1193 739">↩ Direction of movement</p> <p data-bbox="863 750 1145 777">- - - Discontinuity trace</p> <p data-bbox="863 788 1177 815">⊥ Open fracture or void</p> |
| Rockfalls | Falls occur from steeply cut faces in soils, e.g. in excavations for trenches or foundation pits when only short term stability is required. Cracks open behind the face as a result of stress relief or drying shrinkage. Failure occurs near the base of the free-standing column of soil bounded by the crack system, and the mass of soil falls forward or slides into the cut. |  |

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