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The conservation and management of unconsolidated geological sections

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The conservation and management of unconsolidated geological sections

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Foreword

Geological SSSIs were initially selected by a rigorous site selection process, carried out by the Geological Conservation Review (GCR) between 1977 and 1990 (Ellis and others, 1996). This resulted in the identification of over 3000 GCR sites nation-wide. From these, English Nature designated 1240 sites for their notified geological interest. The large number of sites is a testament to the range of geology and landforms present in the British Isles, which for its size, offers the most varied geology in the world (Bennett and others, 1997). Many inland sites of geological importance are associated with active and disused quarries, roads and railway cuttings, as well as through building/construction development that create new geological exposures.

However, geological SSSIs can come under pressure from the demands on land use from other interested parties, notably landowners, mineral operators and developers. These development pressures include the infilling of quarry sites for landfill, their restoration to agriculture, or through development on quarry or cutting floors.

Where the geological exposures are composed of relatively strong rocks that can be maintained at steep face angles, then conservation has enjoyed some success; mainly because the loss of development land is relatively small and maintenance requirements are light. However, English Nature regularly encounters difficulty in conserving and managing faces in unconsolidated, or soft, rocks. These difficulties arise as a result of both long-term slope stability and slope degradation issues, as well as from stabilisation works carried out or required during the development of the site.

This report, commissioned by English Nature from Wardell Armstrong, addresses the technical issues involved in conserving unconsolidated sediments, in particular Quaternary deposits. These are generally composed of sands, gravels and clays, deposited in the last 2 million years. These sediments are of particular importance as they contain detailed information on the recent development of the British Isles, during and since the last Ice Age, including the archaeological record. The report provides an outline of site investigation techniques and reviews a variety of stabilisation methods that may provide options for conserving this group of SSSIs. It will be of interest to developers and planners, as well as conservation staff.

This is a research report and its findings have not necessarily been adopted as English Nature policy and practice. English Nature's views have now been published in a guidance booklet entitled *The Conservation of Soft Sediments on Geological SSSIs*, available from English Nature's enquiry service.

Anna Wetherell
Senior Geologist – Stratigraphy
March 2004

Contents

1. Introduction

1.1 Terms of reference

In accordance with instructions received from M A Hodson of English Nature, dated 9 October 2002, Wardell Armstrong have been instructed to carry out a study entitled:

The conservation and management of unconsolidated geological sections

The Project Reference Number is EIT33-04-012, and the study is defined in the English Nature Project Brief, dated August 2002.

The project became effective from 14 October 2002.

1.2 Background

English Nature is the statutory agency responsible for the conservation and management of geological sites identified as being of national and international significance within England, with associated organisations undertaking similar roles elsewhere in the UK. Conservation, in the broadest sense, centres around the **'notification'** under the system of **'Sites of Special Scientific Interest'** (SSSI) designation. This provides statutory safeguards to these sites through the planning process.

Two of the principle aims of geological conservation are:

- through notification, the conservation of key sites and exposures to facilitate study by experts;
- through management, the maintenance of visible and accessible exposures of geologically important sections for scientific study, training and general interest.

Geological SSSIs were initially selected by a rigorous site selection process, carried out between 1977 and 1990 (Ellis and others, 1996), the **Geological Conservation Review (GCR)**. This resulted in the identification of over 3000 GCR sites nation-wide. From these, English Nature designated 1,239 sites for their notified geological interest. The high number of sites is a testament to the wide range of geological deposits and landforms present within the British Isles, which for its size, offers the most varied geology in the world (Bennett and others, 1997). Many of the inland sites of geological importance are associated with active and disused quarries, roads and railway cuttings, and through building/construction development that create new geological exposures.

However, geological SSSIs can come under pressure from the demands on land use from other interested parties, notably landowners, mineral operators and developers. These development pressures include the infilling of quarry sites for landfill, restoration to agriculture or through development on quarry or cutting floors.

Where the geological exposures comprise relatively strong, indurated rocks that can be maintained at steep face angles, then site conservation has enjoyed some success; mainly because the loss of development land is relatively small and because maintenance requirements are light. However, English Nature regularly encounters difficulty in conserving and managing faces in unconsolidated sequences. These difficulties arise as a result of both long-term slope stability and slope degradation issues, as well as from stabilisation works carried out during site development by other parties. The latter have had significant impact on the degree of exposure of geological faces, to the extent that in some instances geological sections have been removed or rendered unavailable for inspection.

Greenlands Quarry, Purfleet, Essex

An example of this is at the former Greenlands Quarry in Purfleet, South Essex, where geologically important Thames Terrace Gravel deposits are now concealed following the re-development of the site as the Queen Elizabeth Distribution Park. The circumstances which led to this are described in one of the case histories included in the addendum to this report.

In using the term 'unconsolidated' deposits, it is understood that English Nature are alluding predominantly to the geologically youthful Quaternary deposits, such as sands and gravels, and clays. However, other older soft sediments and weathered materials, such as brecciated and reworked chalk can be 'unconsolidated' and could also be included in this classification.

1.3 Scope of works

The purpose of this work is to investigate thoroughly the causes of slope failure, particularly in man-made cuttings and quarries. In addition it is to consider the range of remedial options currently available along with their environmental impacts. Solutions will be investigated that offer the prospect of facilitating future site development and slope stabilisation, with minimum impact on the geological interest.

This work has been carried out in two Phases, the English Nature Project Brief issued in August 2002 described these as follows.

Phase 1. Production of a technical report covering:

- a study of the factors affecting unconsolidated sediments and their management for conservation and development;
- a detailed study of the options for managing and stabilising sites, and the incorporation of geological conservation of soft sediment faces within development proposals. This should also consider how various techniques resolve particular stability issues;
- literature/web search of relevant articles/publications for conservation engineering and planning aspects.

Phase 2. Preparation of a handbook and a series of guidance notes covering:

∉ guidance handbook for planners and developers who need to incorporate the consideration of soft sediments in their work;

∉ guidance note for English Nature area teams, who need to understand technical language, management requirements and options and causes of slope instability;

∉ a short guidance note for the HSE on dealing with unconsolidated sediments on designated areas;

∉ an article that English Nature will submit for publication, summarising the issues and promoting the guidance notes.

This report, including the appended case study reports constitutes the results of the Phase 1 works.

1.4 Report objectives

This report provides the background and technical rationale for a series of Guidance Notes, published by English Nature concerning the legal and technical issues surrounding the conservation of geological sections of recognised national and international importance. The report is aimed at the various Stakeholders that are most usually involved with SSSI management issues, particularly where this includes a planned change in land use in the area close to or within a SSSI.

Stakeholders involved with the management of SSSIs will include:

∉ English Nature;

∉ landowners;

∉ developers;

∉ funding agencies;

∉ mineral operators;

∉ Local Planning Authorities;

∉ technical experts.

Academic specialists and local communities also play an important role.

The principle outputs, other than this report, are the guidance handbooks and notes. The first guidance handbook is:

Guidance handbook for planning applications and developments including Sites of Special Scientific Interest (SSSIs) involving unconsolidated geological soil deposit.

This handbook provides basic advice to all interested parties and summarises:

∉ the legislation available to regulatory bodies;

∉ the duties of Local Authority Planning Departments;

∉ the obligations of landowners, property developers and mineral operators.

A second guidance note is:

Interpretation of management proposals for developments including Sites of Special Scientific Interest (SSSIs) involving unconsolidated geological soil deposits.

This has been prepared specifically for English Nature area teams who need to understand technical management requirements, the causes of instability, and guidance on the assessment of proposed management options.

A third, short guidance note is:

The health and safety implications of the management of Sites of Special Scientific Interest (SSSIs) involving unconsolidated geological soil deposits.

This has been directed at officers of the Health and Safety executive to provide information on the specific difficulties that unconsolidated deposits present to site users and the public. This has been prepared for the benefit of Health and Safety officers that do not have geological or geotechnical specialisms.

Users of the guidance notes are directed to this report where they require background and technical information to support guidance recommendations and considerations.

1.5 Report structure

This report is presented in 10 Sections that provide reference material, guidance and recommendations for a wide range of Stakeholders.

With such a wide ranging remit, it has not been possible to cover all issues in detail. Rather, the report gives general information on a wide range of topics from the legal obligations of regulatory bodies, site developers and landowners to the principles of basic soil mechanics, site investigation methods and slope stabilisation methods.

Table 1.1 summarises the content of each report section and gives report users the information to navigate through those sections that will provide the information needed by any particular Stakeholder group.

Table 1.1 Report structure

Section and title	**Summary of content**	**Typical users**
1. Introduction	Terms of reference, report objectives and structure.	All Stakeholders.
2. The Quaternary System	Describes the type and distribution of the unconsolidated sediments that are the principle concern of this study.	All non geological Stakeholders.
3. Previous Studies	Describes the scope of previous investigations / research projects and technical publications relating to geological conservation issues.	All Stakeholders.

Section and title	Summary of content	Typical users
4. Stakeholders	Describes the roles of the regulators including English Nature, Local Authorities and the Health and Safety Executive. Sets out the regulatory framework behind development planning issues in the commercial and mineral extraction sectors.	All Stakeholders but especially private landowners and technical experts that have no detailed knowledge of planning issues.
5. Engineering Geology	Introduces the differences between geological definitions of unconsolidated sediments and those definitions required by geotechnical engineers to prepare rational design for slope stability.	English Nature and Local Authority Planning Officers that have no detailed knowledge of geotechnical issues.
6. Slope Stability	Defines the causes of slope instability and methods of stability analysis.	English Nature and Local Authority Planning Officers that have no detailed knowledge of geotechnical issues.
7. Site Investigation and Data Collection	Describes the accepted methods of site evaluation and investigation. Includes a summary of the method most suited for investigating unconsolidated soil sections.	English Nature and Local Authority Planning Officers that have no detailed knowledge of geotechnical issues.
8. Generic Stabilisation Methods	This section describes the methods typically used for earth retaining structures.	English Nature and Local Authority Planning Officers that have no detailed knowledge of geotechnical issues.
9. Alternative Stabilisation Methods	Discusses alternative conservation methods that may have specific applications for unconsolidated SSSIs.	English Nature.
10. The Way Forward	Defines a methodology whereby management of SSSIs is achieved through compromise and negotiations with other Stakeholders.	All Stakeholders.

1.6 Case studies

Appended to the report are the findings from a series of **case studies**. The purpose of these studies was to examine existing SSSIs to assess their current conservation/management condition and to provide suggestions on how, if possible, exposure conservation could be improved.

In practice, the **case studies** illustrate a range of problems with respect to SSSI conservation. As such, they form a catalogue of potential conservation issues, with suggestions about how these might be overcome.

A summary of the **case studies** and the issues that each raise is included in Table 1.2.

Table 1.2 Summary of case studies

Site name and location	**Background**	**Conservation issues**
1. Greenlands Quarry, Purfleet, Essex	Chalk deposits exposed by historic quarrying operations are overlain with a sequence of Thames Terrace Gravels. Examination of these terrace deposits at Purfleet have indicated that they represent one of the best defined sections of this geological period, containing a unique succession of fossils and structures.	The English Nature requirement to be able to easily access the Thames Terrace deposits has not been met. Constraints to development space, including the new access road and site boundary fence at the crest, have led to a need to re-grade the terrace deposits and apply surface treatment to promote re-vegetation. As a result, visible access is now not available.
2. Lion Pit Tramway Cutting, West Thurrock, Essex	The cutting sides display a complex sequence of Pleistocene Thames deposits overlying and banked against Upper Cretaceous Chalk deposits. The section has not been stratigraphically correlated to the established Thames Terrace sequence and therefore it requires considerable further work.	Options have been considered covering the future conservation of a series of exposures within the cutting. Recommendations have been made concerning the design process for these exposures along with potential stabilisation options.
3. Barnfield Pit, Swanscombe, North Kent	Former quarrying operations has exposed Pleistocene deposits which lie on an eroded surface of Thanet Sand and Chalk. The site is most famous for the discovery of Lower Palaeolithic human remains in the UK and is arguably the most important site in the British Pleistocene. It is a National Nature Reserve.	There are presently no satisfactory exposures of unconsolidated deposits. The study suggests three options for long term conservation as follows; identify key areas where the sequence is clearly displayed and cover them with topsoil to protect the sections, which could be removed when study is required; identify key areas where the sequence is clearly displayed and engineer slopes to create permanent open sections, possibly contained within a permanent structure. removal of a “peel” of the sequence which would be preserved in an off site location.

Site name and location	Background	Conservation issues
4. Wolston Gravel Pit, Warwickshire	Former quarrying operations have exposed approximately 15m of Pleistocene deposits which are part of a thick succession of beds associated with the glaciation of the English Midlands. The site is the type locality for the Wolstonian, the penultimate cold stage of the Pleistocene in the UK. Since quarrying stopped, the former void has been backfilled with landfill. However, a planned exposure was retained at the southern end in an example of the conservation technique known as the 'conservation void'.	The conservation methods have not worked well in this instance mainly because a poor understanding of geotechnical issues at the time of its formation led to its formation in an adverse position with respect to stability. In particular, issues regarding groundwater and surface water control were not addressed. Recommendations have been made to indicate the types of drainage measures that would be required to establish a lasting conservation exposure. Considerations are also given to the option of abandoning the existing SSSI and replacing this with alternative sections at other locations.
5. Black Rock, Brighton, Sussex	Black Rock is a 24 metre high cliff section of raised beach deposits located behind Brighton Marina. The site is a key section of outstanding importance for Quaternary stratigraphy and provides a valuable record of former sea levels and changing environmental conditions. During April 2001, following an exceptionally wet autumn and winter, a major slope failure occurred causing damage to structures within the marina complex and has resulted in the closure of a public right of way. Since the failure, the City of Brighton and Hove have commissioned studies to investigate the causes of failure and methods of future stabilisation. These have yet to be implemented.	The case study has highlighted further conflicts of interest between, in this case English Nature (along with other members of the scientific community) and the City of Brighton and Hove. Proposed slope stabilisation measures, comprising rockbolts/soil nails and wire meshing, are deemed essential by the City in order to give security to the public right of way. However, these are opposed by English Nature as they will result in the loss of important geological exposure. The case study reviews this conflict, examines the proposed stabilisation methods and suggests alternatives and compromises.

2. The Quaternary period

2.1 Introduction

The Quaternary period represents the geological period that started about 1.8 million years before present (BP) and continues up to the present day. The beginning of the Quaternary period is often considered to be synonymous with the beginning of the Ice Age, and as such, many of the surviving Quaternary deposits and landforms are attributable to the processes that were occurring during the various periods of glaciation (cold periods of ice-sheet development) and inter-glacials (temporary, warmer periods of ice sheet retreat).

Quaternary deposits

Quaternary deposits are geologically relatively young. As such, they usually have little or no primary or secondary cementation, ie: they are unconsolidated. These deposits are weaker than many older (consolidated) geological formations and therefore form less stable exposed sections.

This section presents a background to the origin, nature and geographic distribution of Quaternary deposits in the UK. In many cases, the mode of formation of these deposits, as well as the environmental conditions under which they were produced, determines their physical characteristics.

An understanding of the distribution of a particular formation can have implications on the management of sections where these are exposed.

The cold climatic conditions that prevailed during the ice age episodes also had effects on older strata that were exposed at ground level during this period. These ‘periglacial’ effects included frost shattering and the creation of intraformational shear zones (Worsley, 1977). During the cold and temperate periods of the Ice Age, an array of deposits were laid down, including:

Typical deposits that accumulated during glacial and inter-glacial periods

- sediment accumulations (sands, gravels and finer grained deposits) on river flood plains, in lakes and in coastal areas;
- glacially derived tills and boulder clays;
- accumulations of wind blown sand in dune complexes;
- organic matter in peat bogs;
- solifluction deposits.

The Quaternary period is of scientific interest not only to geologists, but also to archaeologists, as younger deposits may include artefacts associated with early humans. Furthermore, the Quaternary land surfaces may also provide evidence of human occupation and settlement.

Archaeological perspective on the preservation of important Quaternary sites
Geological and archaeological sites of importance frequently overlap, allowing for joint study as is often included as a planning condition. When site development groundworks are being carried out in areas where there is potential for either exposing or impacting upon geologically significant deposits, or uncovering artefacts or human occupation relics, specialists are usually retained to maintain a ‘watching brief’. The long-term objectives of the archaeological community differ in emphasis from those of the geologists. It is generally the case that archaeologists will accept the re-concealment of uncovered sections once they have been recorded. This is known as site ‘preservation’. In contrast, geologists strive to maintain the availability to revisit SSSIs for the purpose of further scientific study and/or training, this is termed ‘conservation’.

This section presents a brief review of the Quaternary period in order to provide the background to subsequent sections dealing with slopes formed in these deposits.

2.2 Geological classification

Traditional geological classification of the deposits of the Quaternary period is largely based on two factors; these are chronology and landform.

The chronological classification is defined mainly by various periods of glaciation (ice sheet advance) and intervening inter-glacials (ice sheet retreat). Overall, the Quaternary is divided into two epochs, known as the Pleistocene (1,800,000 to 10,000 years BP); which included all the glacial phases and all but the current interglacial; and the Holocene, which covers the post glacial period between about 10,000 years BP to the present day.

Definition of Quaternary deposits is often based on the geological process(es) that led to their formation. These were often glacial or fluvial in origin and therefore largely relate to landforms on a localised or a larger scale. Some examples of these landforms and the types of deposit that were formed as a result of these processes are summarised in Table 2.1.

Table 2.1: Typical Quaternary landforms with geological and geotechnical deposit descriptions

Quaternary Landform	Geological Provenance	Typical Geotechnical Properties
Drumlin	A streamlined, oval shaped hill usually composed of till (a poorly sorted, unstratified assemblage with grains ranging in size from clay to boulders). Its long axis is parallel to the direction of flow of the ice sheet beneath which it was formed.	Well graded accumulations of clayey sand with variable gravel and coarser material. Usually no stratification and therefore strength and permeability are similar in vertical and horizontal directions.

Quaternary Landform	Geological Provenance	Typical Geotechnical Properties
Esker	A sinuous ridge of sand and gravel deposited by a melt-water stream flowing beneath an ice sheet or glacier.	Well graded to uniformly graded assemblages of clean sands and gravels, with few fines. Usually well drained, of relatively high permeability. Similar properties in vertical and horizontal directions, but physically of limited lateral extent.
Kame	Origin uncertain, but is often described as a mound of sand and gravel, originally deposited on top of a static ice sheet, and remaining as a topographic feature after the ice melted.	Can be formed from a wide range of source materials. These might be leached of fines at the surface but would usually be well graded including a mixture of coarse and fine grained particles within the centre of the accumulation.
River Terraces	A wide expanse of granular through to fine grained deposits accumulating within river flood plains. Following incisions these deposits are left as fragments of former valley floors which now stand above the active floodplain.	Generally granular with well graded mixtures of sand and gravel, inter-bedded with uniformly graded sands. Fines (silt and clay) rarely exceed 10%. Layering can cause variations in horizontal and vertical engineering properties. In the Thames Valley area, thin layers of brickearth, (possibly wind blown clay silt accumulations) often result in the presence of a low permeability layer close to the top of the Terrace Gravels. These River Terrace deposits are the most commonly encountered deposits during developmen
Lake Deposits	Known as lacustrine deposits, these sediments accumulated at the base of pre-historic ‘still-water’ lakes. The sediments are often show seasonal variations in grain size, with thin layers of coarser sandy layers representing spring run-off and clay/silt laminations representing summertime deposition.	Lake deposits often show highly variable grain size distributions with layers of sand inter-bedded with laminations of clay and silt. As a result, these deposits show marked variations in vertical and horizontal geotechnical properties, notably permeability.

General undifferentiated unconsolidated deposits		
Drift	Used by the British Geological Survey and others to refer to superficial deposits arising from the action of ice. Generally poorly sorted mixture of sediment varying in grain sizes from clay to boulders deposited at the base of an advancing or retreating glacier. Deposits occur extensively over the northern and western UK.	Well graded accumulations of clayey sand and sandy clay with variable gravel and coarser material, although this can include purely granular deposits depending on the source materials Usually no stratification and therefore strength and permeability are similar in vertical and horizontal directions. However, this a widely used term and as such it meaning can be broad. Clarification should be sought.

Since the end of the Ice Age, other Quaternary deposits have generally included:

Holocene deposits

∉ alluvium, which has accumulated on present day flood plains;

∉ lake bed deposits;

∉ loess deposits, collections of wind blown sand;

∉ accumulations of organic matter in peat bogs or poorly drained depressions.

2.3 Distribution

Glacial drift deposits, including unstratified, poorly sorted tills along with bedded sand, and sand and gravel deposits are present over much of northern UK, extending down to a line roughly between the mouths of the River Severn in the west and the River Orwell in the east. This line also demarcates the approximate southern limit of ice coverage during the Quaternary Period. However, most ice related landforms, including drumlins, eskers and kames are only associated with the latest ice advance that occurred at the end of the last glaciation, because older forms have been reworked by subsequent ice advances. These are therefore generally found north and west of a line running roughly between the mouths of the Rivers Severn and Tees.

River terrace deposits largely comprise coarse sand and gravel deposits, which have accumulated at elevations that are above the modern floodplain levels of major rivers and other drainage systems. These terraces have since become isolated as the rivers have cut downwards to lower levels or where river alignments have changed. These are particularly well developed adjacent to the historical alignments of the Thames, Trent and Severn/Avon. The nature and distribution of River Terrace deposits are well described by Clayton (1977).

Locally, the **Terrace Gravels**, particularly those associated with the Thames valley, may contain or be overlain by a cover of silty sandy clay, and often known as **'brickearth'**. The origins of this material is contentious but many believe that it represents wind blown loess deposits (clay silt and fine sand) that has been reworked by river action.

Thin **loess deposits**, including blown silt and fine sand, are present in many areas of southern England and more sparsely elsewhere. It is believed that these were deposited during cold, dry episodes of the Quaternary and may include the **brickearths** described above. The distribution of these is described by Catt (1977).

Glacial lake deposits have formed wherever bodies of water became entrapped by ice or soil (moraine) dams. An example in the Midlands, was Lake Harrison. As a result of seasonal variations in rate and grain size of soil sediments, lake deposits may contain a wide range of sediment types including soils with grain sizes ranging from clays and silts to sands and gravels and are frequently laminated (varved).

Naturally derived slope deposits form an important part of the Quaternary record and are often referred to as 'head'. They formed principally south of the ice limits during the glacial phases when erosion was accelerated because of the arctic climate and lack of vegetation. Accumulations of frost shattered ground, saturated by meltwater, would flow off slope sides during thaws and collect within valleys to form unstratified accumulations, similar to till. Where the head contains significant quantities of chalk, it is often referred to as 'coombe' or 'coombe rock' if cemented, although these terms may be replaced by local descriptions.

The Quaternary chronology since the end of the ice age, about 10,000 years BP, is known as the Holocene. Away from coastal regions, the Holocene is represented principally by **alluvium**, within the floodplains of major rivers, or peat derived deposits. Alluvium consists mainly of silts and clays with thin bands of sand. Lenses and bands of peat may also be present. These **lowland (or fen) peats** are normally composed of sedges and rushes. By contrast, **upland peat** (or moss) is formed by the rapid growth of mosses.

2.4 Key points

The surviving deposits of the Quaternary Period can be summarised as comprising:

Quaternary deposits

∉ generally uncemented and therefore weak from an engineering perspective;

∉ steep sections, often exposed through mans activities, are potentially unstable in the long-term;

∉ surviving deposits are varied in composition and distribution;

∉ sediment accumulations depend on landforms and may result in rapid vertical and lateral variations in the resulting deposits;

∉ deposits may include compressible organic materials such as peat;

∉ the variations in sediment types result in a local changes in the geotechnical characteristics of Quaternary deposits;

∉ the effects of periglacial activity can result in the weakening of older, exposed strata through frost shattering and shearing.

3. Previous studies

3.1 Introduction

The implementation of the Geological Conservation Review (GCR) in 1977 gave scientific grounding to the identification of potential geological Sites of Special Scientific Interest (SSSIs). Since this time, statutory bodies including English Nature and other national conservation groups, along with academic and commercial organisations have strived to advance one of the core conservation objectives, which is to preserve nationally important geological sections for future scientific and educational study and public enjoyment. This objective has been progressed through a range of research studies, symposia, the publication of learned papers in geological journals and positive protection and management of interests on the ground.

At the same time, the science of soil mechanics has made advances, including the study of technical methods of assessing and maintaining the stability of slopes constructed through unconsolidated strata.

As part of the current project, carried out on behalf of English Nature, some of the products of these studies and academic literature have been reviewed. These have provided an insight into the background behind conservation objectives and methodologies, as well as an overview of technical advances in terms of conservation strategies.

In addition, a review has been made of the conservation efforts that have been made in continental Europe to generate permanently conserved sites that contain important geological/archaeological exposures.

Scope of literature review

∉ review of progress in geological conservation management;

∉ bridging the gap between geological conservation and engineering design;

∉ review of efforts made in Europe to conserve important geological sites.

3.2 Review of progress in geological conservation management

Important relevant symposia, technical studies and research projects have included:

Geological management studies	
Author	**Title**
Stevens and others (1992)	*Conserving our landscape. Proceedings of Conference Crewe, May 1992.*
Glaser and Lewis (1994)	*A report on recent excavation and conservation at Wolston Gravel Pit SSSI, Warwickshire.*
O'Halloran and others (1994)	*Geological and landscape conservation*.
Ellis and others (1996)	*An introduction to the Geological Conservation Review*.
Bennett and others (1997)	*An assessment of the 'Conservation Void' as a management technique for geological conservation in disused quarries.*
Bridgland D.R., and others (1997)	*Important faunal sites of the Pleistocene of Germany*.

Geological management studies	
Author	**Title**
Glaser (2001)	*Conservation and management of the earth heritage resources in Great Britain.*

Conserving our landscape*. *Proceedings of Conference, Crewe, May 1992

In May 1992, a conference entitled *Conserving our landscape*, was held in Crewe, Cheshire. The conference was sponsored by the Geologists' Association, Quaternary Research Association, British Geomorphological Research Group, Countryside Council for Wales, Scottish Natural Heritage and English Nature.

Papers presented at the conference included the following themes:

- general aspects of geological conservation;
- coastal conservation;
- river conservation;
- uplands conservation and man made excavations.

Many of the papers described the origins and status of SSSI and RIGS sites; and the needs for the development of site management strategies in order to preserve exposures and key sequences for future study. However, only the following few papers provided technical guidance on how this might be achieved in a physical sense:

- a paper by Addison and Campbell, which included six case studies drawn from sites in North Wales, where a range of conservation principals and requirements were described, along with a useful discussion on how these objectives might be achieved;
- in a paper by Bennett, the concept of the **"conservation void"** was introduced. This was described as a process where, through Planning Agreements, land can be set aside during the restoration of former quarrying operations to provide permanent space for geological exposure;
- finally, a contribution by Bridgland acknowledged the engineering problems associated with the maintenance of steep slopes in Quaternary strata. Suggestions were made for the generation of a number shallow exposures set at differing, albeit overlapping, levels in order to expose a full sequence of deposits. Among the case studies described by Bridgland was an example of the use of the **"conservation void"** concept at Wolston Pit in Warwickshire.

The works undertaken at Wolston Pit SSSI have also been described by Glaser and Lewis (1994) and by Bennett and Doyle (1996).

The implementation of the "Conservation Void" concept at Wolston Pit

The conservation measures taken at Wolston Pit SSSI in Warwickshire were considered as one of the Case Studies included in this report. The site is the type locality for the Wolstonian, a cold stage within the penultimate cold stage of the Pleistocene in the UK.

Former quarrying operations at Wolston Pit exposed a section of Pleistocene deposits, approximately 15 metres high. These deposits are part of a thick succession of beds associated with the glaciation of the English Midlands.

Since quarrying stopped, the former void has been backfilled with landfill. However, a planned exposure was retained at its southern end; an example of the conservation technique known as the 'conservation void'.

Unfortunately, the conservation methods have not worked well in this instance. This is believed to have stemmed mainly from a poor understanding of geotechnical issues at the time of its establishment that have led to its formation in an adverse location with respect to stability. In particular, issues regarding groundwater and surface water control were not addressed and as a consequence the void is subject to flooding and stability problems.

Recommendations have been included in the case study to indicate the types of drainage measures that would be required to establish a lasting conservation exposure.

An introduction to the Geological Conservation Review

The introduction to this volume describes its purpose as explaining the importance of Britain's geological heritage; how this heritage is defined by the Geological Conservation Review and how identified sites are protected by law.

Threats to conservation are described along with issues concerning their physical protection.

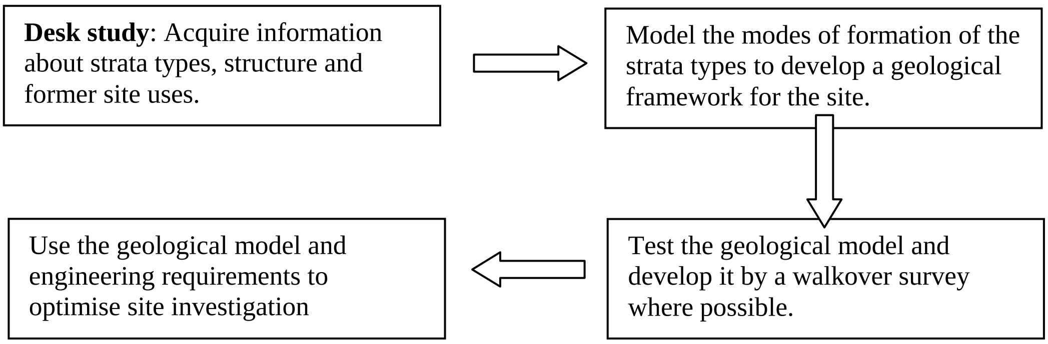
3.3 Geological conservation and engineering design

On 18 November 1997, Professor Peter G Fookes presented the Glossop Lecture to the Geological Society at Keyworth. The title of his address was

Geology for Engineers – the geological model, prediction and performance

Fookes, who is an accomplished civil engineer and geologist, describes the objective of his lecture as that of trying to bridge the gap between the two disciplines of geology and civil engineering "for the purpose of safe construction and excavation" in the ground.

Fookes described the proven principle stages of a site investigation which comprise: desk study, walkover, preliminary ground investigation, main ground investigation and supplementary investigation during construction. A method used in modelling known as the Geological Environmental Matrix (GEM) was explained whereby all details of a site investigation are arranged logically. This can be illustrated generally by the following Flow Chart, which has been adapted from the lecture notes.



A study of this paper, which includes detailed sketches of geological structures, is recommended to ground engineers and geologists alike.

Gibb (1988): ***Factors affecting the conservation of geological features in quarries and pits***

In December 1988, Sir Alexander Gibb and Partners reported on a commission from the Nature Conservancy Council (the predecessor to English Nature) to carry out a study into the factors affecting conservation of geological SSSIs located in active and disused quarries and pits.

The research project was based on visits to 111 SSSIs in quarries and pits. The sites were selected in order to provide a broad range of geological conditions and geographic location. The study included both hard rock quarries and pits exploiting unconsolidated deposits, notably sand and gravel. The remit of the Gibb study therefore extended through a greater range of ground conditions than in this current study, which is confined to unconsolidated strata only.

During the site visits, factors jeopardising the integrity of the SSSIs were found to include landfilling/backfilling, loss by flooding, weather and erosion, geological collection and excavation. However, poor access and visibility were found to be the most common factors affecting exposure utilisation, particularly by talus and vegetation.

Following the initial series of site visits, thirteen of these were selected for detailed review and analysis along with the design and costing of appropriate conservation measures. In slopes comprising unconsolidated sediments, the report recommends that preservation of sections is achieved by grading slopes to between 40 and 50^{o}, and a maximum height of 8 metres. This would be followed by periodic re-cleaning.

Consideration was not given however to any of the legal or safety aspects of conservation.

Symonds (2002): ***Geological conservation of unconsolidated sediments in quarry exposures***

This report was commissioned by the Department of Transport, Local Government and the Regions (DTLR) to examine the problems encountered by English Nature in conserving exposures of unconsolidated geological sediments within current and historic quarry exposures.

Key interest groups were consulted, including: quarry operators, developers, land use planners, English Nature, the Health and Safety Executive and engineering consultants.

The report discusses current planning guidance in respect of geological conservation issues, and is supplemented with reviews of relevant publications and correspondence with academic Quaternary geologists.

Section Two of the report discusses the potential conflicts of interest between stakeholders surrounding the conservation, management, development and utilisation of geological SSSIs. The findings are particularly relevant to this study and are summarised below.

Conclusions of report produced by Symonds, 2002	
Interest group	**Key issues**
Land Use Planners	∉ It is noted that Planning and Minerals Guidance Policy Statements make it clear that land instability is an issue that must be taken into consideration during the planning application process. ∉ Local Authority planners are directed to balance ‘alternative’ development options that may reduce pressure on conservation sites. ∉ Planning departments may not have the technical expertise, in house, to fully assess both latent stability conditions and alternative development options. ∉ Local authorities have increased responsibilities under the Countryside and Rights of Way Act.
Engineering Consultants	In respect of new planning applications, or mineral extraction assessments, specialist engineering consultants are most usually employed by the developer or mineral operator. Therefore, advice that is given by the specialist consultant, while technically competent, may be geared towards satisfying the interests and specifics of their clients over other stakeholder objectives.
Quarry Operators	∉ The report notes that in their normal course of operations, quarries inevitably create geological exposures that may be of interest to the scientific community. ∉ While most operators may allow access to these exposures by geologists for inspection, it is considered that they may be unwilling to conserve working faces, unless compensated for the resulting loss of mineral resources.

Conclusions of report produced by Symonds, 2002	
Interest group	**Key issues**
Health and Safety Executive	The role of the Health and Safety Executive (H+SE) is described with reference to supporting legislation including the Quarries Regulations (1999). ∉ The report notes that quarry faces that cut through unconsolidated sediments at sufficiently steep angles to prevent re-vegetation are likely to be categorised as 'significant hazards' under the Quarries Regulations. Such designated hazards within active quarries must be brought under control, either by engineering design or by the exclusion of people.

Section Three of this report provides an in-depth critique of legislation and guidance relevant to geological conservation, mineral extraction and the stability and safety of excavations and slopes.

It is concluded that while much planning legislation exists to ensure that investigation and design produce secure engineering construction, there is little effort being made to **integrate** these requirements with those of geological conservation. Symonds point out the legislative strength that English Nature now enjoy with regard to enforcing conservation measures, but recognise that English Nature also needs guidance on the suitability of design and management techniques.

Symonds conclude that the implementation of currently available rock slope stability measures to unconsolidated sediments has been largely unsuccessful. They also conclude that management techniques rather than engineering solutions offer the greatest immediate benefit to sediment conservation.

Goldberg (1974)

A method of exposure conservation is described by Goldberg (1974). This entails the formation of a **'sediment peel'** from profiles of unconsolidated deposits. The procedure of making the sediment peel consists of placing a layer of cheese cloth over a cleaned profile and then applying several coats of white polyvinyl acetate glue. Once the glue has dried, the cheese cloth is removed along with the surface layer of sediment from the exposure. The peel can then be mounted for display or stored.

The sediment peel is said to provide a means of retaining important stratigraphic features, which can then be studied in detail in a laboratory. However, it is a last resort and not a favoured substitute for insitu conservation.

3.4 A review of conservation measures adopted in Europe

Abbeville, North-Eastern France

In the Abbeville district of the Somme, north-eastern France, military and civilian earthworks carried out in Quaternary sands and gravels, brickearths and peats during the 19th century yielded a considerable number of archaeological and palaeontological (fossil) finds. The most notable were found at Moulin-Quignon, Fauberg St Gilles and Menchecourt.

Techniques used to preserve some of the World War 1 entrenchments have also been used to conserve some of the exposed geological and archaeological sites. For the most part these techniques are relatively rudimentary, and include the covering of relatively shallow exposures with timber and other materials to provide protection against erosion by wind, rain and frost. At Moulin-Quignon, a doorway has been constructed against a section of sands and gravels to provide viewing access as required.

St Acheul, Somme Northern France

At St Acheul, also in the Somme Valley 19th Century sand and gravel pits, notably Bultel and Tellier (now listed as historic monuments) yielded large quantities of palaeolithic hand tools and axes. Indeed a main sub-division of the Palaeolithic is known as the Acheulean after the sites.

The remaining exposures are now conserved within the St Acheul Archaeological Garden, which incorporates an education/classroom/exhibition area, a ‘Time Trail’ leading through the former gravel pits (with associated information signage), and an observation tower overlooking the Somme Valley.

Bilzingsleben, Thuringia, Eastern Germany

At Bilzingsleben in eastern Germany, an exposed section of Quaternary travertine displays a Lower Paleolithic human campsite that is believed to have been occupied 230,000 to 350,000 years BP. This was also the site of the discovery of skull fragments from both Homo erectus and Homo sapiens, along with numerous other mammal fossil remains.

The site has been preserved for inspection by the construction of a small wooden building which surrounds the principle section. This serves to protect the exposures from environmental factors. This site is currently maintained by the University of Jena and can be visited by arrangement with the custodian.

3.5 Key points

The key points that have come out of the literature and internet review can be summarised as follows:

Key points
Considerable efforts have been made through the Geological Conservation Review, The Joint Nature Conservation Committee, which oversees English Nature and other UK heritage organisations, to provide protection, in statute where possible, to geological exposures of international and national importance (SSSIs and RIGS). These bodies also expend considerable efforts in order to ensure that conservation issues remain in the public focus through the regular organisation of symposia and publication of geological studies in learned journals. ∉ At the same time legislation and guidance has been formulated that are directed at the development planning authorities to ensure that conservation issues are addressed appropriately at planning stages (Symonds, 2002). ∉ However, the literature review has yielded little by way of guidance to the geological community as to how management of available exposures is achieved in a physical sense. This seems to be particularly the case when dealing with unconsolidated deposits. ∉ Bridgland, Bennett and Glaser have all described the concept of the ‘Conservation Void’, which was put into effect at Wolston Pit SSSI in Warwickshire. However, recent examination of this site shows that the trial has so far failed, principally as a result of a poor understanding of geotechnical slope stability mechanisms. ∉ In 1988, Sir Alexander Gibb and Partners reported on a commission to review conservation strategies for geological structures. Many of the recommendations included in their reports were sound, particularly where these concerned hard rock exposures. However, suggestions for the conservation of unconsolidated deposits are considered to be simplistic. ∉ A serious attempt to bridge the gap between geology and civil engineering was made by Professor Fookes in 1997. Unfortunately, this was largely directed at trying bring an understanding of geological processes to construction/excavation engineers, and therefore perhaps missed a potential audience of educating geologists in the principles of geotechnical engineering. ∉ Finally, a thorough review of literature and the internet has found that the situation in continental Europe seems no better than in the UK with the only attempts at permanent conservation measures in difficult unconsolidated deposits being taken up at important archaeological sites, such as at Bilzingsleben and St Acheul.

4. The stakeholders

4.1 Introduction

In England, the maintenance of geological sites of international and national importance that have been designated as SSSIs is administered, under statute, by English Nature. However, in most cases the land in which the SSSI resides is not owned by English Nature. This may lead to potential conflicts of interest if the actual land owners' site management policies differ from those required for satisfactory geological exposure management.

Several scenarios exist with regard to the long-term geotechnical exposure management depending on landowner and site management issues as set out below.

Typical SSSI management scenarios

- sites that are in public ownership;
- sites in private ownership undergoing active development under extant planning permissions;
- sites in private ownership with potential for development or redevelopment;
- sites in private ownership with no short-tem potential for development or redevelopment.

Notwithstanding any geotechnical issues, sites that are under public ownership theoretically present the fewest potential conflicts of interest where geological conservation issues are concerned. Such landowners might include for example: Local Authorities, Crown Estates, Ministry of Defence, and English Nature. Such public bodies generally have 'codes of practice' that are sympathetic to conservation and environmental issues. However, the costs of the creation of geological exposure and their long-term maintenance have to be funded by public money or grants.

Privately owned lands that include geological sites of SSSI status, but which have extant planning permissions for ongoing development, are largely outside the control of English Nature although owner/occupier responsibilities should be observed (see Section 4.2). The new designation of an exposure as a SSSI does not overrule existing planning permissions.

One of the largest groups of landowners with the legacy of also being custodians of SSSIs are within the minerals and waste industries. During active quarrying geological conservation is largely incorporated into working practices. However, opportunities exist for English Nature and operators to direct conservation issues through appropriate site restoration strategies once extraction or filling ceases.

Many sites within the United Kingdom have development potential either from 'greenfield' land undergoing a first time change in use, or land that is derelict, for which an upgrading of land use options is proposed (brownfield). While opportunities are very frequently pursued by a wide range of parties from individual landowners to public bodies, the driving force for development is almost inevitably commercial; where the principal objective of new development opportunities is to maximise financial reward (or public benefit) from site development and/or sale. As such the 'movers' of these commercial interests can be grouped under the title of 'developers'.

Lands containing SSSIs that are located in rural areas can often have very different development values, (often very little) and there are therefore different incentives (also often little) for private landowners to manage such sites in ways that would be of benefit to English Nature or other geological experts. However, landowners are required to ensure that no harm comes to SSSIs for which they are responsible (See Section 4.2).

From the above it is clear that many parties are potential stakeholders with respect to the management of SSSIs on development land, and these stakeholders have differing objectives or roles. Theses can be summarised as shown in Table 4.1.

Table 4.1 Stakeholders in SSSI management

Stakeholder group	**Examples**	**Duties**
The Regulators	English Nature	To champion the conservation of English wildlife and geological exposures of national and international importance.
	Local Authorities	Bound by numerous guidance documents to assess planning applications and to ensure that new developments are in the public interest. Local Authorities are required, under statute, to consult with English Nature where any proposed development impacts on an SSSI.
	Health and Safety Executive	Responsible for the regulation of risks to health and safety arising from work activity in Britain. Includes responsibilities for the Quarries Regulations and Construction (Design and Management) Regulations.
The Landowners and Users	Public Landowner	Usually required to promote environmentally friendly practices with respect to conservation issues.
	Private Landowner	Required to ensure that harm is not caused to a SSSI.
	Minerals Industry	Generally able to continue permitted excavations, but maintain and enhance interest wherever possible eg: arranging conservation measures during restoration works.
Developers and Purchasers and Funders	Entrepreneurs	Looking to make the maximum return on investment placed in land development. Potential conflict of interest when required to conserve SSSI.
	Financial Institutions	Provide funding for development projects. Would generally be reluctant to accept any risk of future instability and would seek a conservative approach to conservation.

Stakeholder group	Examples	Duties
The Experts	Planning Consultants	Usually appointed by the developer to provide advice on the resolution of planning issues.
	Geotechnical Specialists	Usually appointed by the developer to provide advice and geotechnical design including: foundations, retaining structures, and slope stability. However, the range of duties may be restricted by the brief placed by the developer.
	Geological Specialists	Appointed to support and advise on the management of geological sites, and monitor impacts from development and other practices.

4.2 The Regulators

4.2.1 English Nature

English Nature are a Government Agency established under the Environmental Protection Act, 1990. They are funded by the Department of the Environment, Food and Rural Affairs.

English Nature are chartered to champion the cause of conservation of English wildlife and geological exposures of national and international importance. In Wales and Scotland these duties are performed by the Countryside Council for Wales and Scottish Natural Heritage respectively. These three agencies are overseen by the Joint Nature Conservation Committee, which reports directly to government.

English Nature has a prime duty to conserve the diverse English geological heritage for future generations. It is assisted by non-statutory groups, including academic geologists and amateur societies that administer ‘Regionally Important Geological and Geomorphological Sites’ (RIGS). One objective of both English Nature and the RIGS groups is to provide the initiative required to retain exposures of important geological sections for scientific interest and associated environmental and educational benefits.

Except by way of pressure that can be asserted on Local Authorities and their Planning Departments, the voluntary groups and academic communities currently have no statutory powers to conserve local geological sections. However, **Article 10 of the Town and Country Planning (General Development Procedures Order) (1995)** requires Local Authorities to consult with English Nature whenever proposed developments may impact on designated SSSIs.

The coming into force of the **Countryside and Rights of Way Act (CROW), 2000**, gives further protection to SSSIs. In the first instance, by 2005, a management statement must be prepared for all SSSIs within England and Wales. Protection will then be afforded to geological exposures through the preparation of **‘Management Agreements’**, where the landowners/users are co-operative or by the implementation of **‘Management Schemes’** where the agreement of the landowners/users cannot be secured. In extreme conditions the 2000 legislation gives English Nature the powers to purchase the site, either through voluntary agreement or the use of **Compulsory Purchase Orders** where necessary.

4.2.2 Local Authorities

The purpose of the town and country planning system in England and Wales is to ensure that applications for changes in land use, wherever and for whatever purpose these are proposed, are in the general public interest, meet long-term rural and urban planning objectives and comply with development guidance set in legislation and policy.

The body of guidance documents to which Local Planning Authorities have to take into consideration is considerable. These include **National Planning Guidance Notes (PPGs)** and **Mineral Planning Guidance Notes (MPGs).**

Minerals Policy Statements and Planning Policy Statements

Mineral Planning Guidance Notes (MPGs) and Planing Policy Guidance Notes (PPGs), are presently being reissued as Minerals Policy Statements (MPSs) and Planning Policy Statements (PPSs).

Clearly, MPGs relate principally to applications for new mineral extraction applications or changes to existing mineral extraction consents.

Many of these guidance documents relate to environmental and nature conservation issues and the background to these are described in detail in the Symonds report to English Nature of 2002. However, two of the PPGs are particularly worthy of note and are discussed below.

Planning Policy Guidance PPG 9 – Nature Conservation

Planning Policy Guidance Note 9 (PPG 9): Nature Conservation (1994), regulates planning policy with respect to nature conservation. The guidance recognises that:

"adequate provision for development and economic growth …….." is a crucial part of any regional development strategy and should therefore be promoted where appropriate. But that such development should only be permitted:

"…….whilst ensuring effective conservation of wildlife and natural features".

This guidance note gives support to conservation bodies such as English Nature, helps Planning Officers in their task of assessing new application and aims for sympathetic redevelopment of sites that contain designated SSSIs.

It has been noted earlier that Local Authorities are required by **Article 10 of the Town and Country Planning (General Development Procedures Order) (1995)** to refer any application that includes a SSSI to English Nature for their consideration.

Planning Policy Guidance Note 14 (PPG 14): Development on Unstable Ground

A second Planning Policy Guidance Note 14 (PPG 14): "Development on Unstable Ground" (1990) notes that it is the responsibility of a particular site developer or owner to ensure the stability of all lands within and adjacent to a proposed development. But it also requires that Planning Authorities be satisfied that development proposals have been investigated and designed in sufficient detail that conclusions regarding long-tem stability might be expected to be reliable.

As a consequence of PPG 14 it is unacceptable that a natural unconsolidated soil slope, whether of geological significance or not, is left as a result of site development or re-development, in a condition where it might be allowed to degrade, either by shallow depth erosion or by more deep seated slope failure. This is particularly the case where such slope degradation may result in interference to public footpaths or highways, or to neighbouring properties. As a result, Local Authority planners as well as developers are usually keen to provide for relatively shallow (conservative) slope angles through unconsolidated soils, backed up with measures to encourage the rapid development of vegetation cover. These provide the most secure and maintenance free solutions to long term site slope stabilisation.

The Planning Guidance notes make it clear that it is not the Local Authorities responsibility to call for radical slope design to enable the creation of stable, exposed geological sections. Rather, they are required to ensure that proposals made by planning applicants can lead to sustainable site development within the various planning frameworks.

The application of PPG 14 is demonstrated by reference to Case Study 1, Greenlands Quarry, Purfleet, Essex (included as an addendum to this report) and described below. This case study also reflects the difficult position for Local Authorities to balance stability and safety requirements with nature conservation responsibilities.

Greenlands Quarry, Purfleet, Essex

Chalk deposits exposed by historic quarrying operations at the former Greenlands Quarry at Purfleet, Essex are overlain by a sequence of Thames Terrace Gravels. Examination of these deposits has indicated that they represent one of the best defined sections of this geological period, containing a unique succession of fossils and structures.

Redevelopment of the site, entailing the construction of the Queen Elizabeth Development Park resulted in space constraints. The construction of a new access road and site boundary fence at the crest, have led to a need to re-grade the terrace deposits and apply surface treatment to promote re-vegetation. As a result visible access to the SSSI is now not available.

The English Nature requirement to be able to easily access the Thames Terrace deposits has not been met.

Health and Safety Executive

The Health and Safety Executive (HSE) are responsible for the regulation of almost all the risks to health and safety arising from work activity in Britain http://www.hse.gov.uk/pubns/ohsingb.pdf. In particular, The HSE administer two sets of regulations that are of significance to this study; these are the **Quarries Regulations (1999)** and the **Construction (Design and Management) Regulations (CDM), 1994.**

Since 1999, active quarries have been subject to the **Quarries Regulations**. These regulations have been brought in under the **Health and Safety at Work Act (1974)** and impose requirements with respect to health and safety in active quarries. The regulations supersede certain provisions formerly imposed by or under the **Mines and Quarries Act (1954)**, the **Mines and Quarries (Tips) Act 1969** and in certain other health and safety regulations. They apply to all quarries where persons work and impose duties on the operator with respect to persons at or in the area immediately surrounding the quarry.

Part VI, of the **Regulations** relates amongst other things, to excavations and tips and requires the operator to ensure that inspections and assessments are carried out on a regular basis. Some of the requirements are detailed below.

Some provisions of the 1999 Quarries Regulations

- excavations and tips should be designed, constructed, operated and maintained so as to ensure health and safety and to ensure that excavations and tips rules are prepared;
- proposed or existing excavations or tips are appraised by a competent person and, where required, subjected to a geotechnical assessment;
- excavations and tips are subject to further geotechnical assessments at specified intervals and in specified circumstances.

The requirements for safety in design can conflict with the aim of providing a steep section for geological conservation purposes. It should be noted that the **Quarries Regulations (1999)** do not apply to disused quarry sites.

The **CDM** Regulations place duties on the stakeholders involved in a construction project to plan, co-ordinate and manage health and safety at all stages. For a development site at a former quarry, for instance, the **CDM** regulations would typically be applicable. The design of a section of slope through unconsolidated strata would have to consider health and safety objectives. The regulations require that engineers minimise risk at the design stage, before construction begins so the site workers can manage health and safety risks during construction.

4.3 The Landowners and users

4.3.1 The minerals industry

The extraction of mineral resources is governed by the mineral planning process. Over recent years numerous **Minerals Planning Guidances (MPGs)** have been published and have become part of the Local and County Council Planning Authorities' armoury of regulations governing new development and minerals applications. These guidances have been succinctly described in the report to **English Nature** prepared by Symonds (2002) and will only be referred to, as necessary, in this study.

Many of these guidances, including **MPG 7, *The Reclamation of Mineral Workings (1996)*,** relate to conditions given in mineral extraction approvals for restoration requirements, once the quarry resources are exhausted.

In general **English Nature** monitors geological exposures that are uncovered during the active extraction of mineral resources. However, on an informal basis quarry operators are often amenable to make important geological exposures available for inspection by the scientific community.

Historically, consents for mineral extraction have been granted without conditions governing land restoration requirements or with any regard to the future management and conservation of nationally and internationally important geological sections. However, it is now often the case that mineral operators recognise the potential development value of the void space for future development purposes. In urban areas in particular, the void space generated by mineral extraction may be utilised as landfills for the disposal of domestic or inert wastes, thereby generating further income to site owners, followed by development for commercial or residential usage. In rural and semi-rural areas surrounding towns and cities exhausted quarries may also be utilised as landfill, followed by restoration to beneficial use for agriculture or forestation.

Where the prospect of refilling quarry void space with waste is not a viable option, for instance in rural areas or where the demand for filling space is absent, restoration will usually be carried out to minimise and control hazards and provide amenity space.

It has been the case recently, that former chalk pits in south-east Essex and northern Kent, are being used for housing, retail and industrial development without prior backfilling. This is of particular interest where River Terrace Gravels overly the chalk and commonly constitute designated SSSIs (see Case Studies).

Where circumstances for new planning applications are required for either the landfilling or recreational use of an exhausted quarry void, English Nature have the opportunity to intervene in the planning process and to direct requirements to conserve SSSIs if within the curtledge of the application boundaries. It is one of the objectives of this report to provide English Nature with the technical means of asking the necessary questions to ensure that the conservation of such sites is protected in the long-term. In particular, English Nature is encouraged to guide appropriate site investigations, analysis methods and slope (exposure) design that are appropriate for future scientific study.

This is well demonstrated by reference to a specific example of active quarry site in Telford, Shropshire where recent experience has shown that use of the Town and Country Planning process can be utilised to achieve SSSI conservation.

Application of a Section 106 Agreement

An active brick clay quarry in Shropshire, contains a currently designated SSSI that relates to exposed structures within the Hadley Formation of Upper Coal Measures (Carboniferous) age. Although the quarry has up to thirty years of future brick making clay/mudstone reserves, plans are already being prepared for its ultimate restoration.

The final land use is expected to comprise: backfilling with inert fill, the construction of a mix of housing and light industrial development areas, and the establishment of areas of public open space. In addition, there are plans to set aside an area of the exposed quarry highwall to conserve an exposure of the SSSI.

This exposure is being planned with due consideration to its long-term protection. In order to control land drainage at the foot of the exposure, which if unchecked could restrict access to the section as well as jeopardising long-term stability, it is proposed to create a 'wetland' area with an artificially 'depressed' groundwater level.

To ensure that the wetland area is not allowed to flood, pumped drainage measures are to be incorporated within the restoration scheme. The pumped drainage will be commissioned by the quarry owners, but its long-term maintenance will be assured through a **Section 106 (Town and Country Planning Act, 1990)** agreement with the Local Authority. Under this arrangement, the quarry owners will provide a lump-sum payment to the Local Authority. In theory the investment of this capital sum will provide sufficient revenue for the Local Authority to adopt and maintain the drainage scheme once the quarry operator has sold the land.

4.3.2 Private land owners

As discussed above, since the enactment of the **Countryside and Rights of Way Act (2000),** it has become a requirement to prepare a '**Management Statement'** for every SSSI in England and Wales by the Year 2005. These may subsequently be developed into '**Management Plans'** either with the agreement of the private landowner or, if necessary by imposition. In the extreme, the legislation allows for the voluntary or compulsory purchase of land containing the SSSI, if there is a danger of the notified interest suffering significant harm.

4.3.3 Public landowners

There is a requirement on all publicly owned or run organisations to conserve and enhance SSSIs. Any works that are to be carried out that may impact on the site may only be carried out after consultation with English Nature and in receipt of an approval for the works to go ahead.

4.4 The developers and funders

4.4.1 Developers

As discussed in Section 4.1, **'developers'** may include a wide range of entities from private landowners looking to make improvements to an existing property on a local scale, to the commercial developer seeking to invest money, usually in a construction project – for instance: new housing, industrial or retail units. In most cases however, the objectives of the development, whether on a small or large scale, is to **'add value'** to the land within the application boundary.

Applications to make significant development (or redevelopment) of land must pass through the Local Authority planning procedures. The proposals will be measured against the numerous **National Planning Policy Guidances (PPGs),** or if the proposal includes the excavation of mineral resources, to the **Mineral Planning Policy Guidance Notes (MPGs).** These guidances will include those (for instance PPG 9) that might alert the Planning Authorities to the presence of SSSIs within the curtilege of a planning proposal.

There is clear incentive for developers to prepare applications that meet the approval of Planning Authorities and where appropriate, Government Agencies including English Nature. However, they are unlikely to propose measures that incur significant extra cost or loss of development land. Furthermore, they will act on the advice of their retained experts in matters concerning planning issues and slope or retaining wall design.

4.4.2 Financial institutions

Financial institutions ('funders') include city and merchant bankers, controllers of investments, insurances and pension funds. They provide the capital used for the larger development projects with expectations of fixed financial returns. The first duty of the funders is to their own financial clients. Development investments are generally protected through a series of warranties that indemnify the funders against financial losses arising from negligent advice or design given by the developer or his specialist consultants.

4.5 The experts

4.5.1 Planning consultants

Planning Consultants are generally retained by the larger property developers to provide advice during the preparation of Planning Applications and to negotiate its satisfactory progress through the Local Authority planning process. The Planning Consultant can provide the guidance and expertise to broker the compromises that may sometimes be required to overcome impasses between the developer and planning authorities. The Planning Consultant would normally provide the expertise to negotiate, for instance, **Section 106 Agreements (Town and Country Planning act, 1990)** to provide the financial standing for the long-term maintenance of SSSIs.

4.5.2 Geotechnical specialists

Geotechnical specialists are retained by the developer to provide advice on ground engineering and also environmental matters (especially those related to ground and groundwater contamination) with respect to a proposed development. Ideally, their commission should follow the phased rationale as described by Fookes (1997) and in Section 6 of this report. This would ensure that the data collected during the desk study, reconnaissance, intrusive and laboratory testing phases are co-ordinated to provide data of appropriate quantity, quality and distribution to satisfy the requirements of the commission.

Well researched desk studies, carried out in advance of planning applications will identify in a preliminary way the constraints and 'abnormal' costs that must be accounted and provided for during project financing. Desk studies should include the identification of SSSIs when these are present within influencing distance of a proposed development, and highlight any requirement to initiate conservation measures.

However, it is often the case that the Geotechnical Specialist is only brought into the project team at a relatively late stage in the planning programme and has been commissioned to provide advice on a relatively narrow scope of works defined by the appointment brief. It is therefore possible that geotechnical advice can be given by professional consultants who may be unaware of the full site circumstances.

Geotechnical Specialists are protected against the risk of providing inappropriate geotechnical advice by retaining Professional Indemnity insurances. These back-up the warranties that specialists can be required to provide to employers as part of their professional services.

The eligibility of Geotechnical Specialists to be able to maintain insurance cover at a reasonable premium, requires a conscientious and quality controlled design process. Given the constraints of commission, this results in groundwork designs that are prepared in accordance with well established, safe, design codes. This circumstance provides little scope for geotechnical specialists, retained by the developer, to arrive at innovative geotechnical design, or to make recommendations that are not in keeping with best industry practice.

Notwithstanding the above, there is of course no reason why additional Geotechnical Specialists cannot be retained by Local Authority Planning Departments or even by English Nature. Under these circumstances, a technical assessment can be made of the advice provided by the developers specialists and, importantly, ensure that the data and methods used for geotechnical design are sufficient and appropriate, and that due consideration has been given to the conservation of geological sections.

4.5.3 Geological specialists

Geological specialists with expert knowledge of a particular type of deposit or landform, can be retained by either English Nature, Planning Authorities or the developer. Their role can be to advise on the impacts of proposed development options and can assist with the formulation and assessment of mitigation strategies.

During construction works, geological specialists can provide a monitoring function, recording newly opened, temporary geological sections. This is a similar role to the 'watching brief' that is frequently adopted by archaeologists.

4.6 Key points

Potential conflicts of interest between English Nature and other stakeholders in land ownership, mineral exploitation and development or re-development have been described. The Key Points that have arisen from this assessment are summarised below.

Key points

- quarry operators have to follow Health and Safety legislation and may batter slopes in unconsolidated sections above rockhead to shallow angles for safety reasons;
- quarry operators will ideally want to maximise extraction, leaving little room at site boundaries for future engineering of slope sections should the quarry be redeveloped after it has closed;
- in many cases, small slope sections in unconsolidated sediments located at the edge of development sites may be considered to be of a relatively low priority (compared with other ground engineering issues) and may not receive due attention at an early stage;
- developers will rely on advice from Geotechnical Specialists in the design of slopes, who in turn will have to design for safety (under CDM). Steep unprotected slopes do not readily comply with requirements for safety;
- Local Authorities are also required to take safety and stability into account when considering development applications (PPG 14), although they also have clear responsibilities with regard to Nature Conservation (PPG 9);
- the future owner of developed land containing conserved SSSIs may be reluctant to take on long-term maintenance responsibilities, although the planning process does allow provision to be made under, for instance Section 106 Agreements, for Local Authorities to adopt these responsibilities in exchange for up-front financial contributions.

In order to try to overcome some of the conflicts of interest described above English Nature should make use of the powers that they already have under statute. These might include for instance:

- ensuring that appropriate site investigations are carried out at the planning stage to enable informed decisions to be made with respect to geological section conservation;
- obtaining independent opinion on slope stabilisation options;
- establishing financial provisions for long-term maintenance where this is required to conserve SSSIs.

5. Engineering geology

5.1 Introduction

The classification system described in Section 2 of this report is used by geologists to describe the landforms that typically define Quaternary sediment accumulations. However, this system is of only limited assistance to the geotechnical engineer.

For instance, what geologists call ‘Quaternary sediments’ are ‘soils’ in engineering geological terms.

To enable judgements to be made concerning the short and long term stability of exposed soil sections (slope stability), the geotechnical engineer must be able to assign strength and other behavioural properties, including drainage characteristics and stress history, to geological formations. This has led to the development of an alternative means of describing unconsolidated soil deposits that is based not on their mode of formation, but on their physical characteristics. This system of soil description is described in Section 5.2. Stress history plays an important role in determining the engineering properties of soils, as outlined in Section 5.3.

The concept of effective stress also needs to be appreciated as it is fundamental to an understanding of slope stability and slope engineering. An overview of effective stress is given in Section 5.4. Sections 5.5 and 5.6 review the role that suction pressures and cementation have on determining the engineering behaviour of soils.

5.2 Engineering description of soils

The engineering description of non-indurated (unconsolidated) deposits, ie soils, is very different to the geological classification described in Section 2 of this report. It is based principally on:

- particle size and particle size distribution (grading);
- states of compaction (relative density and shear strength) and
- organic content.

Personnel preparing full engineering descriptions of soils require both training and experience. Comprehensive guidance for the description of soils in the United Kingdom is provided within British Standard BS5930:1999 *A code of practice for site investigation.* The simplified descriptions of the engineering soil description system given below are intended to illustrate the principles only, and to demonstrate the differences between geotechnical and geological soil description.

Table 5.1 shows the detailed soil description system in full, as defined in the British Standard.

The engineering description of soils should include:

∉ **mass characteristics**, such as: field strength, moisture content, bedding, discontinuities, fissures and weathering;

∉ **material characteristics** such as: colour, particle shape and composition, grading and plasticity.

British Standards relating to site investigation and soil description	
The following British Standards relate to site investigation and soil description	
BS 1377:1990	British Standard Methods of Test for Soils for Civil Engineering Purposes
BS 5930:1999	Code of Practice for Site Investigations
BS10175:2000	Code of Practice for the Investigation of Contaminated Sites

The engineering soil description system defines three basic soil types (granular soils, cohesive soils and organic soils) to which other descriptors including strength and grading can be added. Descriptions of each of these soil types are included below.

5.2.1 Granular soils

Granular soils are defined as soils that contain more than about 25% of its constituent grains with a nominal dimension greater than 65σm (micron).

Such deposits can generally be described as sands, gravels, cobbles or boulders. Each term represents a defined range of particle sizes as follows:

Particle size range for granular soils

∉ Sand has a range of particle sizes between 60σm and 2mm. It can be further subdivided into ‘fine sand’ (60σm to 200σm), ‘medium sand’ (200σm to 600σm) and ‘coarse sand’ (600σm to 2mm);

∉ Gravel has a range of particle sizes between 2mm and 60mm. As with sand, gravel can be further subdivided into ‘fine gravel’ (2mm to 6mm), ‘medium gravel’ (6mm to 20 mm), and ‘coarse gravel’ (20mm to 60mm);

∉ Cobbles have a range of particle sizes between 60mm and 200 mm. There is no further subdivision within the cobble range;

∉ Boulders have a range of particle sizes larger than 200 mm. There is no further subdivision within the boulder range.

In mixed granular soils, the principal soil description is based on its main constituent. For instance, where a soil is shown, either by inspection or laboratory testing, to comprise mainly sand, but with some gravel it could be described as ‘gravelly SAND’.

The range of particle sizes within a given soil mass is known as its **grading**. Soil grading curves are obtained by passing soils through a set of sieves with different mesh sizes (or by sedimentation for fine grained soils) eg: glacial till. Examples of typical soil grading curves are shown in Figure 5.1. A soil that contains a wide range of particle sizes, and is characterised by a flat grading curve, is said to be **'well graded'** (the equivalent geological term would be 'poorly sorted') eg: glacial till. Deposits that comprise only a small range of particle sizes, and therefore have steep grading curves are termed **'uniformly graded'** ('well sorted') eg: coarse sand eg course sand.

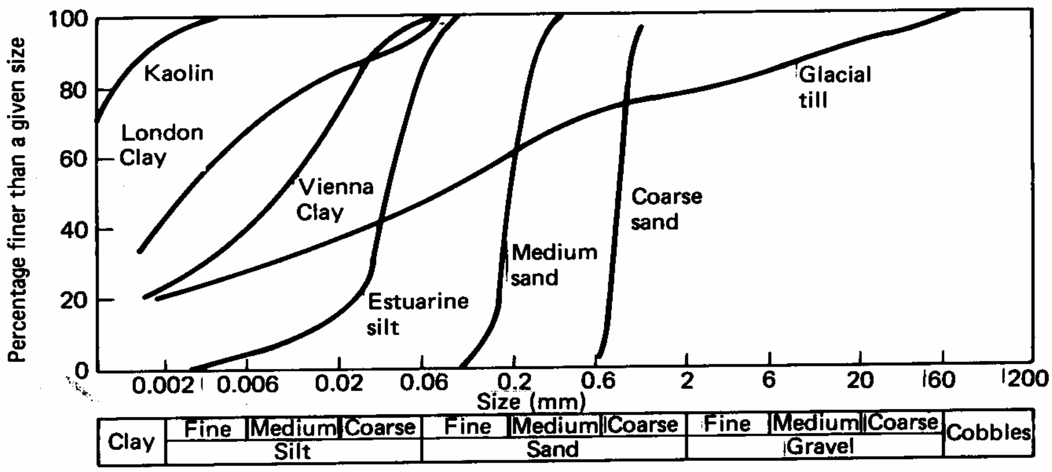


Figure 5.1: Examples of Soil Grading (After Bell, 1987)

The importance to the geotechnical engineer of being able to define a soil as **'granular'** is because it also provides information relating to two other important soil attributes. These are shear strength and drainage.

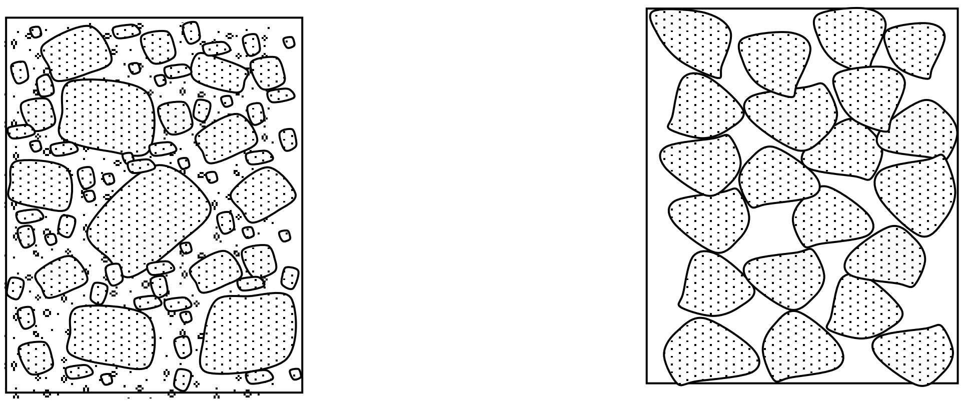
Attributes of granular soils	
Attribute	**Description**
Shear Strength	In granular soils, shear strength is defined by its **angle of friction** only (cohesion is absent). The angle of friction (designated by the symbol λ) of a granular soil will be the largest angle that a slope, of any height, will stand indefinitely. The angle of friction depends generally on: the **angularity** of the grains; the range of **particle sizes** present in the soil mass (grading) and the degree of **compaction**.
Drainage	Granular soils are generally considered to be **free draining**, ie they will allow the passage of water without the formation of **excess pore water pressures**. The drainage potential of a soil is defined by its **permeability**. For the most part coarse grained soils are more permeable than fine grained soils. Similarly, **uniformly graded** soils are more permeable than **well graded** soils.

The shear strength of granular soils is defined by inter-granular friction only, ie they possess no cohesion. This is important with respect to slope stability. The shear strength of granular soils is usually expressed in terms of an angle of friction ‘λ’. The greater the angle of friction then the greater is the shear strength of a given granular soil. In a general sense, the angle of friction of a granular soil will be the largest angle that a slope, of any height, will stand indefinitely.

The magnitude of the angle of friction is dependant upon a number of factors, but mainly on grain shape, grading and degree of compaction.

The shape of the constituent particles, and therefore to a degree also the mineralogy, defines the inter-granular friction. Clearly, angular particles will have higher inter-granular friction than rounded particles. The range of particle sizes within a given soil mass (grading) relates to the magnitude of inter-granular contact. For instance, in a **‘well graded’** soil, the gaps between the larger particles are filled with those of a smaller size. The greater the magnitude of inter-granular contact, the higher the friction angle.

The degree of compaction that has been applied to any soil type can be expressed as the amount of effort that has been available to push the constituent particles together. This compactive effort results in increased density as smaller particles are forced into the gaps between the larger ones, expressing air or water. To a degree, this is also dependant on the grading, described above, as no amount of compaction would increase the density of a granular deposit with a single particle size. This is illustrated in Figure 5.2.



Well Graded – Suitable for compaction

Uniformly Graded – Not suitable for compaction

Figure 5.2: Sketch of well graded and poorly graded soils

Of great importance to slope stability is soil drainage. The presence of excess pore water pressures and seepage forces are frequently the cause of slope instability. For the most part, granular soils can be considered to be ‘drained’, ie where present above the water table, moisture within the deposit will include only that which is retained by molecular attraction. Theoretically, water that enters a saturated granular deposit, for instance from surface run-off, will pass vertically through the soil by gravity. The drainage potential of a soil is defined by its permeability. This is expressed in units of metres per second (ms^{-1}), but can also be considered as the time required for a unit volume of water to pass through a unit area of soil. Most granular soils have a permeability in the range 10^{-5} to 10^{-1} ms^{-1}. Generally, uniformly graded soils are more permeable than well graded soils, where the packing of inter-granular pore space with finer particles reduces their drainage potential. However, more accurately, the permeability is dependent upon the grain size of the finest soil constituents. This is expressed in **Hazen's rule**, which allows soil permeability of sands to be estimated from soil grading as follows:

Hazen's rule for estimating permeability from soil grading

Permeability $= (d_{10})^2$ x constant.

Where d_{10} is the maximum particle size in metres of the finest 10% of the soil constituents and the constant varies between about 1.0 and 1.5 if permeability is expressed as metres per second.

It will be shown later that variations in permeability can be of great significance with respect to the objective of designing stable geological sections. Although, deposits may be made up of predominantly granular soils (for instance River Terrace gravels) the presence of even thin layers containing an abundance of fine grained soils, such as brickearths, can upset soil drainage, with implications to slope stability. Table 5.2 shows the typical range of permeability with respect to dominant particle size for granular and cohesive soils.

Table 5.2: Relationship between grain size and main descriptive divisions for soils with approximate permeability range

Grain size (mm) (log scale)	200	60	20	6	2	0.6	0.2	0.06	0.02	0.006	0.002	
Basic soil type	Boulders	Cobbles	Coarse	Medium	Fine	Coarse	Medium	Fine	Coarse	Medium	Fine	CLAYS
			GRAVELS			SANDS			SILTS			
	VERY COARSE SOILS		COARSE SOILS						FINE SOILS			
Drainage properties	High permeability generally $k>10^{-4}$ m/s (fine sands) Maximum can approach 1 m/s								Low permeability poor drainage $10^{-4}>k>10^{-8}$ m/s			Practically impervious $k<10^{-8}$ m/s

Cohesive soils

Cohesive soils are defined as soils that contain more than about 25% of constituent grains with a nominal dimension less than 60σm (micron).

Such deposits can generally be described as clays and silts. Each term represents a defined range of particle size as follows:

Particle size range for cohesive soils
Clays have a particle size of less than 2σm, they comprise a distinct mineralogical group of substances that display greater, or lesser degrees of cohesion;
Silts have a particle size of between 2σm and 60σm, while of a similar fine particle size, silts are generally formed from the degradation of quartz and other hard rock minerals. They display a measurable, apparent cohesion, but, in a pure form, ie where clay is absent, will crumble when dry.

From the definition given above, it can be seen that while a soil can contain less clay or silt than its coarser fractions (sand, gravel etc) by weight, it may still be termed cohesive. This is because only a relatively small proportion of clay and silt is required to affect the physical parameters that were shown above to define soil behaviour, ie, shear strength and drainage.

This also accounts for the rather imprecise particle size boundary between cohesive and granular soils. This is because it can be possible for soils to contain as much as 35% clay and silt, but still behave like a granular soil; for instance if the other 65% comprised coarse gravel and cobbles. Similarly, if a soil is predominantly made up of fine sand, it may need only as little as 15% clay/silt for the soil as a whole to behave as a cohesive material.

Attributes of cohesive soils	
Attribute	**Description**
Shear Strength	In cohesive soils, shear strength is defined by its **cohesion** and its **angle of friction**. In an undrained condition, the **angle of friction** is generally assumed to equal zero. However, when drained the angle of friction is positive. Its value usually depends on **mineralogy**, **plasticity** and state of **compaction**, the **angularity** of the grains; and the range of **particle sizes** present in the soil mass (grading). **Peak** shear strengths are applicable to soils that have not previously been sheared. However, if previous shear has occurred there is a marked reduction in strength to **residual** shear strength values.
Drainage	Cohesive soils are generally considered to be **slow draining**, ie they restrict the passage of water. When loaded, cohesive soils develop **excess pore water pressures** until these dissipate through slow drainage. The drainage potential of a soil is defined by its **permeability**. For the most part cohesive soils have a permeable that may be four or five orders of magnitude lower than granular soils. However, the mass permeability of clayey sediments may be increased by the presence of fissuring or thin sandy partings.

The shear strength of a clay soil is made up of two components. One of them is angle of friction and is similar to the property described above for granular soils. However, because of mineralogy, this is usually of a lower magnitude than for granular soils. The other component, which is not present in granular soils, is cohesion. Cohesion is derived from the mineralogy of the clay particles.

Collectively, clays are made up of alumino-silicate minerals including kaolin, illite and montmorillonite, and these minerals have a platy crystalline structure that has an affinity to retain molecular water between adjacent crystal plates. Water therefore provides a physio-chemical bond between adjacent particles. This means that even when apparently dry, clay will become hard and not easy to crumble. It is the cohesive component of clays that enable them to stand vertically when faces are first excavated.

However, clays also contain water that is not bound into the crystal structure. This free water does not contribute to soil cohesion, rather it can lubricate clay mineral surfaces, thereby reducing shear strength. In addition, because free water in cohesive soils can only drain very slowly, excess pore water pressures can develop in a newly excavated slope leading to varying stability conditions with time.

Once a cohesive soil has been sheared, the platy clay minerals are realigned parallel to the shear surface, giving rise to a lower shear strength. The pre-failure shear strength is the 'peak' value, whereas the ultimate post-failure value is termed the 'residual' value.

Because of the mineralogical differences between clays and silts, distinction between the dominant component of these soil types is necessary, since their derived soil mass will behave in different manners. A relatively small proportion of clay minerals within a composite soil can confer cohesive properties, and can be sufficient to warrant description as 'clay'. Mechanical behaviour is therefore used for the description of fine materials.

The property most indicative of the relative proportion of clay minerals in a fine grained soil is its 'plasticity'. This property is not readily determined insitu, being related to the proportion, by weight, that a soil can absorb water and yet still behave as a solid material; and the proportion of water that it can absorb before it acts as a liquid. The difference between these two values (referred to as the Atterberg Lilits), during which it acts as a plastic material, ie: neither as a solid nor as a liquid, is termed its 'plasticity', and is a measure of both the proportion and type of clay mineral within a soil aggregate.

With a knowledge of moisture content and plasticity, combined with an understanding of the geological stress history of a cohesive soil, it is possible to draw some inferences regarding the likely engineering behaviour of the soil. For example, in terms of slope stability, high plasticity soils will exhibit a greater loss of shear strength at failure than low plasticity soils.

Organic soils

These largely comprise peat deposits made up of decaying plant remains. Organic soils can hold prodigious quantities of water, often more by weight than solids, within the decayed plant structure. Unlike clays, this water is not bound molecularly, and peats are not slow draining. Peat is normally defined as having more than 35% organic matter. However, this category also includes many current alluvial deposits where periods of flood result in the deposition of sediments over the vegetated flood plain. The classification of organic soils is described in British Standard BS5930:1999 as follows:

Classification of Organic soils (after BS5930:1999)		
Term	**Organic Content (% by weight)**	**Typical Colour**
Slightly organic clay or silt Slightly organic sand	2 – 5 1 - 3	Grey As mineral
Organic clay Organic sand	5 – 10 3 - 5	Dark grey Dark grey
Very organic clay Very organic sand	> 10 > 5	Black Black

Small quantities of dispersed organic matter can have a marked effect on soil plasticity and therefore the engineering properties.

Attributes of Organic Soils	
Attribute	**Description**
Shear Strength	The shear strength of organic soils is generally considered to be too low to be relied upon in geotechnical analysis. They are characterised by unusually high compression when loaded, as water held within the organic structure is expelled.
Drainage	Organic soils are generally considered to be free draining, ie: they will allow the passage of water without the formation of excess pore water pressures. However, they are often associated in alluvial deposits, with cohesive soils. As a result mass permeability may be variable and be greater horizontally than vertically. The natural moisture content of organic soils is often very high, and in some peat deposits often exceeds (sometimes by several orders of magnitude) the weight of its solid component.

5.3 Stress history

In general, the source of the compactive effort comes from the weight of overlying deposits (termed the thickness of overburden). There is therefore a general increase in the amount of compaction (and angle of friction) with depth, other factors being equal. Such compaction can be retained when the weight of overburden is removed either by erosion or excavation. Where the thickness of eroded overlying deposits is considerable, say more than 30 metres, then the soils are described as **over-consolidated**.

Fissuring, which is often associated with over-consolidated soils, and is seen in older clays such as London and Oxford Clay, can play an important role in slope instability by allowing free moisture to enter the clays, promoting softening. For the most part, Quaternary deposits are **normally consolidated**, that is they have not undergone significant unloading as a result of erosion or uplift. However, exceptions to this include tills, that have been historically compressed by the weight of glacial ice.

5.4 Concept of effective stress

A complete description of the concept of effective stress is beyond the scope of this report. However, it is important for practising geologists to understand the basic concepts. Consider a unit of saturated soil that is confined such that the drainage of water is prevented. The application of load to the soil would be such that part of the imposed load would be carried by the soil particles, while another part of the load would be carried by the water between soil particles (water is incompressible).

In practice however, no soil system can be fully confined, and drainage of water, under load will always take place. The result of this drainage in the saturated system described above is that, with time, more and more load is taken by the soil particles and less by the interstitial water. When the load taken by the soil exceeds its shear strength then soil failure can occur.

The principle behind the concept is illustrated simplistically in Figure 5.3.

Failure of a unit of soil therefore, for instance within a slope, depends both on the magnitudes of stress and the state of drainage.

‘Total stress’ is the term used to describe the stress in the ground due to the weight of overlying soil and any external land. As the soil comprises a solid and vapour (usually waters) phase, ‘pore pressures’ are present. The term ‘effective stress is measured as the difference between total stress and pore pressure.

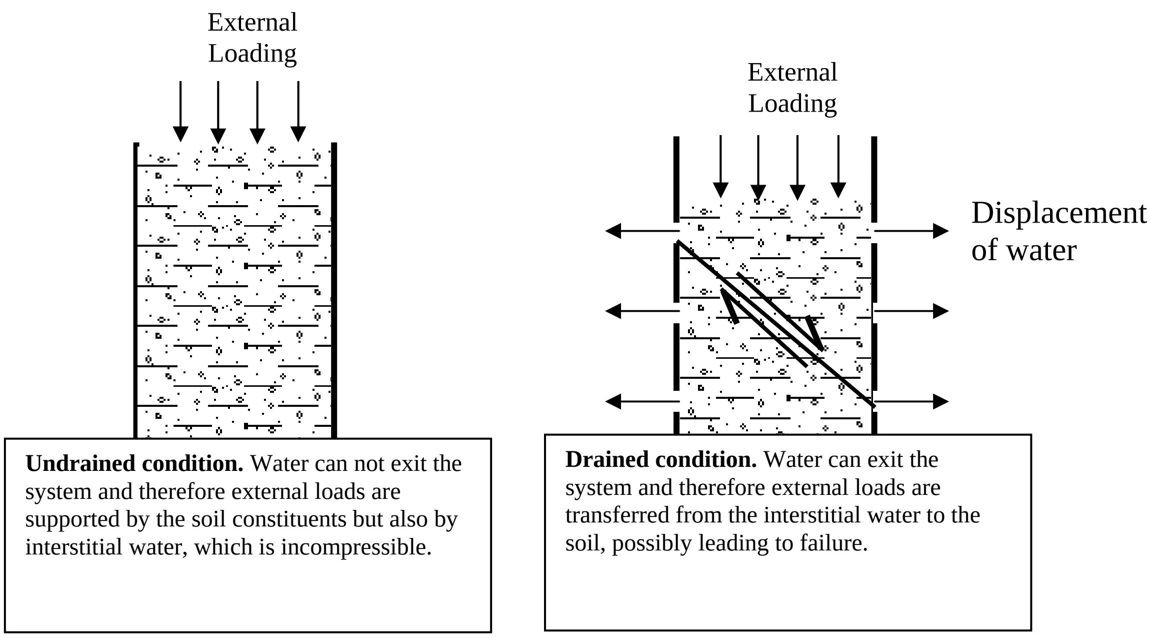


Figure 5.3: Concept of Drained and Undrained Loading

Effective stress concept
In terms of effective stress the stress causing deformation in a drained system (’) becomes the total stress ( ) (ie load divided by area over which the load is applied) minus the pore-water pressure (u).   **’ = - u**

‘Excess pore pressures’ are developed in response to undrained loading, and dissipate at a rate controlled by the permeability of the soil.

As described above, drainage of water from cohesive soils takes place very slowly. Therefore, shortly after excavation, a slope in a cohesive soil can stand at a relatively steep angle. However, as drainage occurs, its stability reduces. For this reason, stability analysis of soil slopes takes two forms. Immediately after excavation, slope stability is analysed in a ‘Total Stress’ condition – this means that the effects of soil drainage are ignored. The input values for such an analysis can be those obtained from quick insitu or relatively simple laboratory tests. These test methods are discussed in Section 6 of this report.

Relatively simple laboratory tests are appropriate when dealing with free draining granular soils, as excess water is allowed to drain quickly, during the test. However, this is not so with cohesive soils, in this case the critical ‘Effective Stress’ condition occurs in the long term. Analysis of this state is carried out using drained shear strength parameters. These can only be measured by performing sophisticated, and relatively expensive laboratory testing. Alternatively, these parameters can also be obtained from the back-analyses of case histories involving slope instability.

5.5 Soil suction

Preceding sections of this report have described the classical approach to soil mechanics and slope stability analysis where, under failure conditions, soils are assumed to be fully saturated. This is a convenient assumption and by and large, generates conservative designs. The principles of ‘effective stress’ were introduced in Section 5.4, which describes how in the short-term, an increase in stress on an element of saturated soil, resulting for instance from the excavation of a slope/cutting, initially results in the excess stress being carried by inter-particulate water – being incompressible. The initial loading condition is characterised by a positive pore-water pressure. With time, quickest in granular soils of high porosity and permeability, the is a gradual stress transfer to the soil particles as the ‘excess’ water migrates to areas of lower pore-water pressure. Positive pore-water pressures therefore gradually dissipate, resulting in an apparent decrease in soil shear strength.

However, except in some particular geological circumstances, such as inter-bedded sand and clay soils where sub-horizontal groundwater flow occurs, the soils located immediately behind cut slopes are rarely fully saturated. But nor are they completely dry. The remaining moisture within this **‘vadose zone’** as it is known, is held in place by surface tension to the solid particles surrounding it. This surface tension provides such soils with an apparent cohesion or ‘soil suction’, even in soils where conventional cohesion generated by the presence of clay minerals may be absent. Indeed, it is possible for slopes comprising purely unsaturated granular soils (sands and gravels) to stand at angles significantly steeper than their measured angle of friction. This is the so called ‘sand castle effect’. In terms of the effective stress principles of soil mechanics, the apparent cohesion (or soil suction) generated in unsaturated soils can be considered in terms of a **negative pore-water pressure**.

The **'Vadose Zone'** is the ground located above the level of the standing groundwater table, where the water pressure is zero, and includes an assemblage of mineral particles (solids), retained moisture (liquid) and air (gas). It therefore defines a soil state that includes materials in three phases.

The engineering applications of **soil suction** with respect to slope stability are a topic of active current research as is demonstrated by the Symposium in Print entitled Suction in Unsaturated Soils that is presently being published by the Institution of Civil Engineers in their geotechnical periodical *Geotechnique* (ICE, 2003).

For the most part, in normally draining natural soils, the apparent cohesion resulting from **soil suction** has been estimated as having a magnitude of between less than 1 kN/m^2 to about 30 kN/m^2 (Springman and others, 2003). However, soil suctions can reach much higher magnitudes, particularly in cohesive soils when these are affected by groundwater depletion caused by the growth of nearby trees of high water demand. This effect is of particular significance in plastic cohesive soils where this desiccation of the soils leads to volumetric shrinkage and to the potential for differential settlement beneath affected structural foundations. However, with regard to slope stability, it is generally considered that for low to medium sized slopes, a contribution of only about 4 kN/m^2, may provide a significant positive contribution to slope stability. Indeed, it is the contribution of **soil suction** which often results in observed slopes standing at significantly greater angles than would be expected on the basis of the results of conventional laboratory testing.

The above discussion suggests that under drained conditions, the shear strength of ground above the standing groundwater level benefits from a negative pore water pressure that equates to a soil suction. **Soil suction** can in theory, be utilised in slope stability analyses, However, the relatively small levels of apparent cohesion that are generated under normal soil draining conditions are vulnerable to change depending on the degree of soil saturation. Immediately behind slope faces, prolonged soaking by a period of intense rainfall can lead to large increases in the degree of saturation with consequential loss in soil suction. These conditions may lead to local sloughing of ground close to the slope face. This gradual slope face degradation may be repeated as the newly exposed ground also loses soil suction during further periods of soaking. Such occurrences are difficult to control using conventional design processes.

As a result of the risk of sudden loss of shear strength generated by **soil suction** following sudden increases in moisture content from environmental factors, there is reluctance among geotechnical designers to rely on soil suction as a means of providing reliable, long-term, shear strength parameters. It has been discussed earlier, that geotechnical specialists are, for the most part restricted from and/or reluctant to apply ‘novel’ or ‘risky’ technology in respect of geotechnical design. This reluctance is only increased under the present industry climate where professional designs by geotechnical specialists often have to be underwritten by indemnity insurances and collateral warranties.

Notwithstanding this, it is considered that circumstances may arise where, under closely controlled environmental conditions, utilisation of the additional soil shear strength afforded by a measure of **soil suction** could be applied to the long-term conservation of geologically important soil sections. These would require as a pre-requisite, the full environmental control of the section under consideration. Such controls would include:

Measures required to enable the utilisation of soil suction in design
∉ minimisation of horizontal groundwater recharge from behind the section;
∉ shielding of the section face from surface run-off and direct impact;
∉ shielding from excessive surface drying from wind and frost;
∉ prevention of excessive soil drying and shrinkage that may result in fissuring.

In many cases it is considered that the level of environmental control required to generate steep faced conserved geological sections, relying on soil suction for prolonged stability may not be justified. It would be necessary not only to incur capital costs while carrying out the requisite design and construction works, but also to have to rely on investment to be able to pay for the provision of long-term maintenance.

Utilisation of soil suction design methods at sites of national importance
Where the section is of such importance, for instance, those qualifying as National Nature Reserves, such as Swanscombe, north Kent (see Case Study No. 3) then it is believed that the case could be made to make such sections the focal point of publicly accessible heritage centres. It is envisaged that environmental control (including soil suction groundwater recharge and evaporation from the excavated face) required for long-term section conservation could be achieved by the construction of a secure shelter around the exposed ground, providing space for public display and educational facilities.

Under these conditions it would be possible to control the environment at and behind the slope face and to monitor the available soil suction from probes embedded within the slope. The latter would also be of long-term benefit to the geotechnical profession by providing information on the variations in soil suction in sheltered environments.

5.6 Soil cementation

For the most part, unconsolidated deposits can be considered as being un-cemented. Indeed the age of these materials is such that they have not undergone the diagenetic processes that result in the transformation of soft/loose sediments to rock.

However, there are some instances where geological conditions have resulted in sediments being deposited and remaining underwater for sufficiently long periods for weak cementation to occur, or being subject to fluctuating groundwater levels where materials are precipitated out of a solution and cement the soil particles.

The most common cementation process is the dissolution of amorphous quartz and/or calcium carbonate/sulphate from the solid phase into pore water. Changing water and gas pressures can result in the re-precipitation of these minerals, which can form bonds between adjacent soil particles. Because calcium carbonate is several times more soluble than amorphous quartz, this is the most commonly observed form of soil cementation.

The effect of soil cementation is to increase the shear strength of a soil by increasing its effective cohesion. Effective angle of friction can remain unchanged.

There are dangers in placing undue reliance on soil cementation of geologically young soils. On exposure, the cementation process can reverse, with a breakdown of cementation caused by circulating groundwater. In the case of calcium carbonate cementation, this breakdown can be accelerated by the infiltration of mildly acidic rainwater.

Notable examples include the raised beach deposits at Black Rock, Brighton and other locations in the south of England.

5.7 Key points

The engineering description of soils is superficially similar to geological ones, but follow rigorous protocols where sediments, such as ‘sand’ or ‘clay’, have a specific definition based on material properties and engineering behaviour.

In terms of the objectives of the current project the following Key Issues were highlighted.

Key issues with respect to soil behaviour	
Key Point	**Description**
Soil type	Three soil types have been described that have particular characteristics that affect performance in geological sections: ∉ **Granular soils –** mainly sands and gravels, containing less than about 25% fine grained materials. Shear strength is defined by angle of friction only. ∉ **Cohesive soils –** mainly clays and silts, although they may still contain up to 75% sand and clay. Shear strength is defined by cohesion and angle of friction. ∉ **Organic soils –** contains more than between 5 and 10% organic matter (more than 35% to be classified as a peat). Organic soils are characterised by very low strength and very high compressibility.
Soil Drainage	The importance of soil drainage was described with respect to geological conservation. The parameter that most describes a soil’s drainage potential is permeability. Permeability is generally dependent upon the soil grading. However, in practice the drainage of sand and gravel deposits can be restricted by the presence of low permeability layers such as brickearth.
Stress History	It is important to understand the stress history of a soil to be able to predict behaviour characteristics and be able to assess the state of stresses in the ground so as to understand present and future stability conditions of exposed geological sections. This is particularly the case with regard to pore water pressures that can have significant slope stability issues over time.
Effective Stress	The principle of effective stress is introduced. When preparing a design for a geological section it will be necessary to consider the drainage conditions prevailing at the time of construction and in the long term.
Soil suction	The concept of soil suction is defined. Soil suction can be a significant slope stabilising influence in unsaturated soils. However, the negative pore water pressures that generate soil suctions can be lost following re-saturation, for instance following heavy rainfall. Consequently, soil suction is rarely relied on in geotechnical slope stability design.
Cementation	Under some environmental circumstances, sedimentation conditions has led to the development of weak cementation in some Quaternary deposits (eg Black Rock, Brighton). The effect of this cementation is to increase the shear strength of the soil through an increase in effective cohesion. However, it has been recommended that a cautious approach be taken to placing reliance on cementation, particularly where slope instability can have serious consequences. This is because re-exposure can result in a breakdown in the cementation with time under that action of migrating groundwater or rainwater infiltration.

6. Slope stability

6.1 Introduction

Section 6.2 of this report sets out to describe some of the typical modes of instability that occur in slopes formed from unconsolidated soil deposits. Some of these modes take the form of gradual slope surface degradation, while others can result in the sudden displacement of large quantities of ground. The circumstances that result in these types of slope instability are described.

Sections 6.3 and 6.4 describe, in principle only, some of the classical methods of slope stability analysis. The purpose of this is to provide a background, for non-specialists, to the basic mechanics of stability calculations, and give guidance on the types of analyses that are appropriate in given situations. Slope stability analyses require a thorough understanding of soil mechanics principles and should be undertaken by specialists.

Finally, Section 6.5 explains to the concept of ‘Factor of Safety’, which is critical to the use of many of the traditional slope stability calculation methods.

6.2 Typical modes of failure

For the purposes of analysis, slope failure may be divided into a number of categories based on the failure mode. The more common failure modes are listed below.

Common modes of slope instability
∉ surface erosion;
∉ groundwater erosion;
∉ shallow subsurface sliding;
∉ rotational failure;
∉ irregular surface failure;
∉ wedge failure.

The geotechnical and environmental factors that govern these types of instability condition are described in the following sub-sections.

6.2.1 Surface erosion

Surface erosion occurs in granular soils and is commonly seen as slow but progressive surface degradation. Such erosion occurs under the action of rainfall (and surface run-off) and wind, although it can be exacerbated by extremes of ground temperature caused by direct heat and frost. Surface erosion involves a process of both soil particle detachment and transport.

The impact of raindrops on the surface of a soil slope can break down soil aggregates and disperse particles. Soil movement by rainfall (raindrop splash) is usually greatest and most noticeable during short-duration, high-intensity storms, although the erosion caused by long-lasting and less-intense rainfall can be significant, especially when compounded over time.

At the onset of run-off, water collects into small rivulets, which may erode small channels called ‘rills’. These rills may coalesce into larger and deeper channels called ‘gullies'. Run-off can occur whenever there is excess water on a slope that cannot be absorbed into the soil structure or trapped by artificial drainage measures. The amount of runoff can be increased if infiltration is reduced due to soil compaction, crusting or freezing.

Rainfall/run-off erosion is controlled by the following four basic factors:

Factors affecting slope surface rainfall and run-off	
Climate	Storm intensity, duration and frequency.
Soil Type	Inherent erodibility – dependent on size distribution and state of compaction.
Topography	The length and gradient of slope.
Vegetation	Type and extent of cover

Clearly, local climate will have an influence on the duration, frequency and intensity of rainfall. Information relating to these variations is available from the Meteorological Office and from publications relating to the prediction of urban surface water drainage requirements, for instance BRE Digest 365 (1991) “Soakaway Design”.

The susceptibility of a soil to erosion is known as its erodibility. While there is no basic test procedure to determine erodibility, the following trends have been established from observation:

Factors determining erodibility

- erodibility is low in well graded gravels;
- is high in uniform silts and fine sands;
- decreases with increasing clay and organic content;
- reduces with low void ratios and high antecedent moisture content.

Vegetation is often seen as a means of reducing or controlling rainfall/run-off erosion by binding the surface layer with roots and reducing the effects of raindrop impact.

Wind erosion is controlled by the same basic factors that control rainfall/run-off erosion. Fine particles, usually less than 0.1 mm (ie fine sand and non-cohesive silt) can be suspended by the wind and then transported great distances. Coarser particles can be lifted and deposited or blown along the surface, this is commonly known as the **‘saltation effect’**.

The speed and duration of the wind have a direct impact on the extent of soil erosion, as does the orientation of a given slope to the prevailing wind direction. Soil moisture levels can be very low at the surface of excessively drained soils or during periods of drought, releasing the particles for transport by wind. This effect also occurs in freeze drying of the surface during winter months. Notwithstanding this, there are also stabilising effects that come from the evaporation of water within soil slopes that results from the establishment of negative pore-water pressures – also known as ‘**soil suction’** – see Section 5.5.

Methods of control of wind erosion include the use of windbreaks or the establishment of vegetation cover. However, as can be seen in many coastal dune systems, the permanent establishment of these controls can be problematic.

The effects of frost action is to cause loosening of surface soils by the formation of ice crystals between individual soil particles with resultant expansion. On thawing, the loosened surface soils become more amenable to other forms of erosion. The materials most susceptible to frost damage are silts and fine to medium sands.

Summary of factors affecting surface erosion

The effects of surface erosion can be summarised as follows:

- it occurs primarily in granular soils;
- it is caused by environmental factors, including: rainfall (and surface run-off) and wind, but can be exacerbated by temperature extremes including heat (from exposure to sunshine) and frost;
- direct rainfall (and splash), especially during heavy storms can dislodge soil particles on a slope face;
- surface run-off, which flows down unprotected slopes generates erosion features including ‘rills’ that may coalesce to form larger features, such as ‘gullies’;
- wind erosion results from the dislodgement of silt and fine sand size particles from exposed faces;
- temperature effects both from exposed sunshine or frost can loosen soil surfaces exacerbating erosion later by other means.

6.2.2 Groundwater erosion

Groundwater erosion is the removal of soil as a result of groundwater seepage from a soil face. Such erosion is commonly called piping. Piping occurs when seepage forces exceed inter-granular friction.

The sketch included as Figure 6.1 demonstrates how rainfall and surface run-off may collect behind a soil slope by infiltration into a permeable granular soil. If a layer of lower permeability is present within the body of the slope, this can restrict the vertical movement of groundwater, initiating flow towards the face and the development of a hydraulic gradient. The transport of fine particles from within the slope may result in loosening of these soils leading to enlargement of depressions behind the crest.

In the same way, dislodgement and transport of soils at the line of seepage results in slow but progressive steepening of the slope face above. Ultimately this will lead to shallow surface soil movement extending to the slope crest.

The example shown in Figure 6.1 represents the mode of slow, progressive slope failure observed in glacio-fluvial sands and gravels at a quarry site in North Wales.

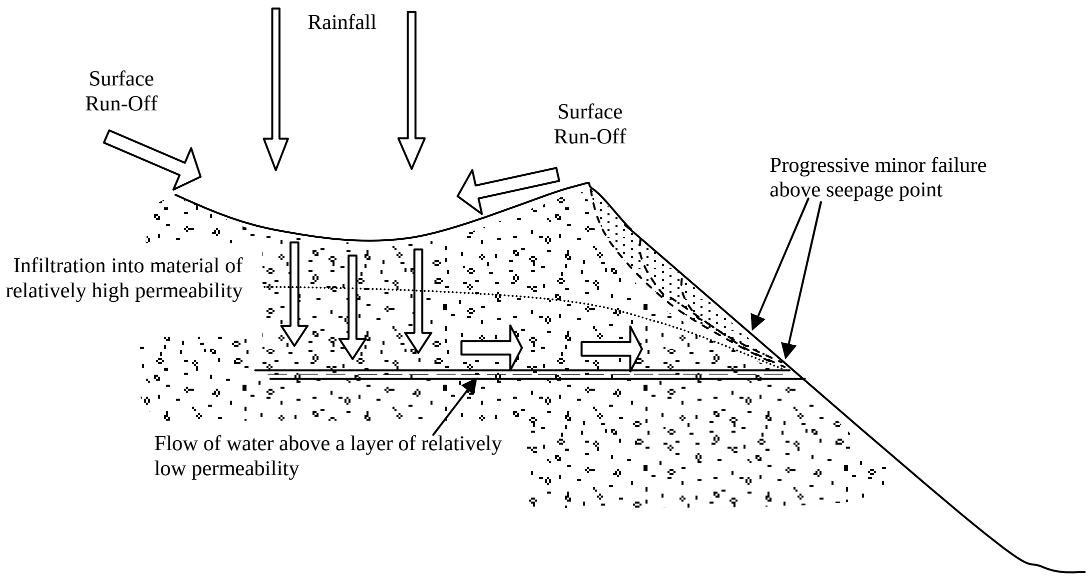


Figure 6.1: Instability caused by groundwater erosion

Summary of factors causing groundwater erosion

The factors causing groundwater erosion can be summarised as follows:

- occurs primarily in granular soils, but especially where the vertical movement of infiltrating water is prevented by the presence of low permeability layers;
- seepage from a slope face can result in the erosion of material from within the slope, this is known as internal erosion or 'piping';
- in addition, water seepage from the face can result in surface erosion leading to over steepening and surface slumping.

6.2.3 Shallow surface slides

As a distinction to the type of slope instability described above, the shallow surface slide represents the translational movement of surface materials during a single sliding event, albeit these may be of long or short-term duration. They are distinguished from the deep seated type of rotational failure described later in this section, in that these slides are generally confined to weakened near surface materials in natural or older man made slopes. The failure surface is prevented from extending deeper into the ground mass because of increasing soil strength. Such slides often extend through colluvial deposits overlying solid strata on natural hillsides (see Figure 6.2); but can also include softened cohesive and loosed granular soils.

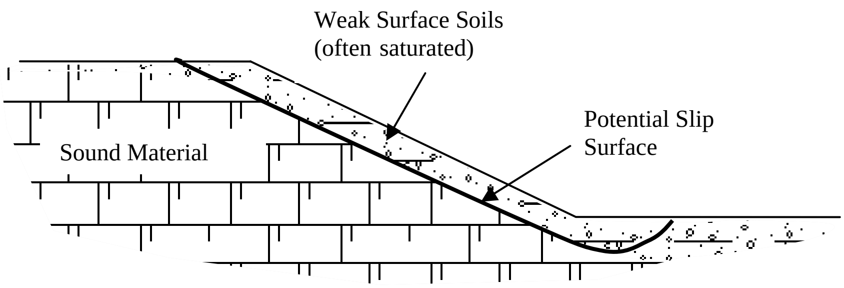


Figure 6.2: Shallow Surface Slide Through Weak Surface Soils

Two categories of surface sliding can be distinguished depending on the rate of movement. Firstly, slow soil creep, including the transport of objects supported by the soil, such as vegetation, is often termed solifluxion.

Secondly the process can be accelerated by the presence of water within the surface materials or by sudden a change in water state, such as thawing after a winter freeze. These rapid movements, including mud flows, occur in the same types of ground as surface creep – solifluxion, but can result in the destruction of surface structures and the uprooting of vegetation.

Summary of factors causing shallow surface slides

The factors causing shallow depth surface slides can be summarised as follows:

- ∉ the transport of a body of soil under a single event, although this may occur very slowly, in the case of surface creep (or solifluxion) or rapidly, in the case of mud flows;
- ∉ instability of this type is common in natural or older man-made slopes. Instability is prevented from becoming more deep-seated by increasing shear strength with depth below slope surface.

6.2.4 Rotational failure

Rotational failure occurs predominantly in cohesive soils when the resolved stresses from the weight of soil and groundwater exceed the shear strength of the soil. Theory, as well as observation, has shown that in many cases the shape of the failure surface when seen in a cross-section, approximates to an arc of a circle as shown in Figure 6.3.

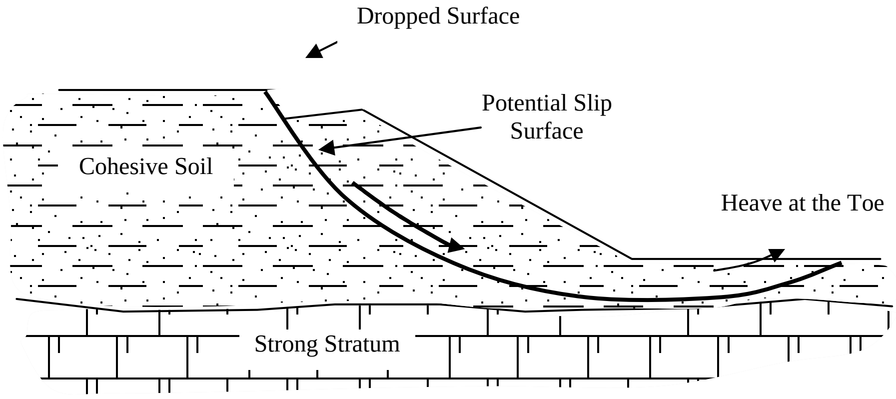


Figure 6.3: Rotational Failure in Cohesive Soils

When failure is initiated as a result of soil weight and groundwater pressures, it is often the case that failure surfaces are deep seated, and this distinguishes them from the shallow sliding mechanisms described above. The limit to the slip surface depth often results from the presence of stronger ground beneath the base of the slope. However, case histories have shown that failure surfaces often extend below the slope toe, resulting in ground heave in this area.

Rotational failures are of great significance because their potential to be deep seated can result in their generation of prodigious volumes of failed ground.

Groundwater pressure usually plays a significant role in the onset of this type of failure. However, unlike the shallow depth slides described in earlier in this section, rotational failures do not necessarily depend on sudden changes in groundwater volumes or levels, but can simply result from the gradual, long-term change from undrained to drained soil conditions.

Summary of factors causing rotational slope failure

The factors causing rotational slope failure can be summarised as follows:

∉ rotational failure occurs predominantly in cohesive soils when the resolved stresses from the weight of soil and groundwater exceed the shear strength of the soil;

∉

∉ when failure is initiated as a result of soil weight and groundwater pressures, it is often the case that failure surfaces are deep seated. This distinguishes them from the shallow failure mechanisms described previously;

∉

∉ rotational failures are of great significance because their potential to be deep seated can result in the generation of prodigious volumes of failed ground.

6.2.5 Irregular surface failure

Although, the assumption of a slip surface forming along an arc of a circle is convenient for analytical reasons, it is sometimes the case that the real, non-circular shape of a slip surface must be taken into consideration. In general, failure along a non-circular surface can be anticipated if the soil deposit is non-homogeneous, or contains fissures or discontinuities as shown in Figure 6.4.

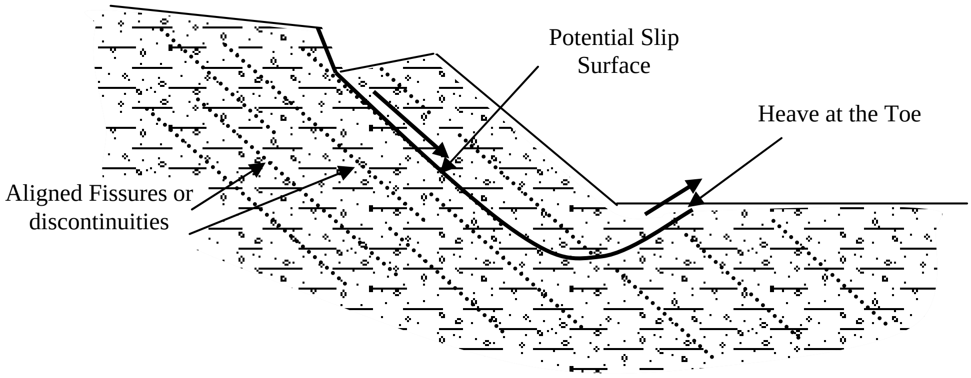


Figure 6.4: Irregular Failure Surface Development in Fissured Soils

An overall non-circular failure surface can also result when progressive circular type failures occur as a series, advancing up a hillside – see Figure 6.5.

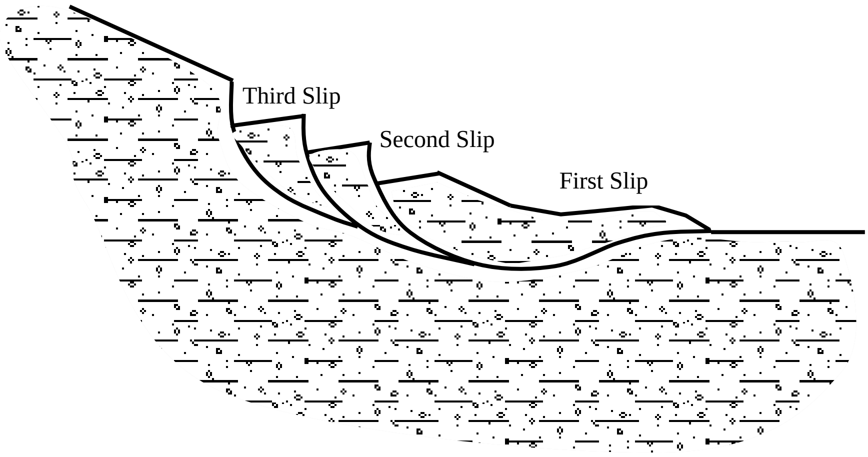


Figure 6.5: Irregular Slip Surface Development due to Progressive Failure

Summary of factors causing non-circular rotational slope failure

The factors causing non-circular rotational slope failure can be summarised as follows:

- ∉ it is sometimes the case that the real, non-circular shape of a slip surface must be taken into consideration. In general, failure along a non-circular surface can be anticipated if the soil deposit is non-homogeneous, or contains fissures or discontinuities;
- ∉ an overall non-circular failure surface can also result when progressive circular type failures occur as a series, advancing up a hillside.

6.2.6 Wedge failure

In weak layered strata, failures can take place along defined planes. These often include a sub-horizontal sliding surface, with a steep back face as shown in Figure 6.6. Often, the disturbing force in this type of failure comes from high water levels within the ground behind the steep back face.

Figure 6.6 shows two variations of wedge (or block) failure. The upper wedge has the sliding plane day-lighting in the slope face and is known as a ‘two-part wedge’. The lower block has the sliding plane running beneath toe of the slope. In this case, the passive resistance of the ground sitting above the plane must be overcome. This case is known as the ‘three-part wedge’.

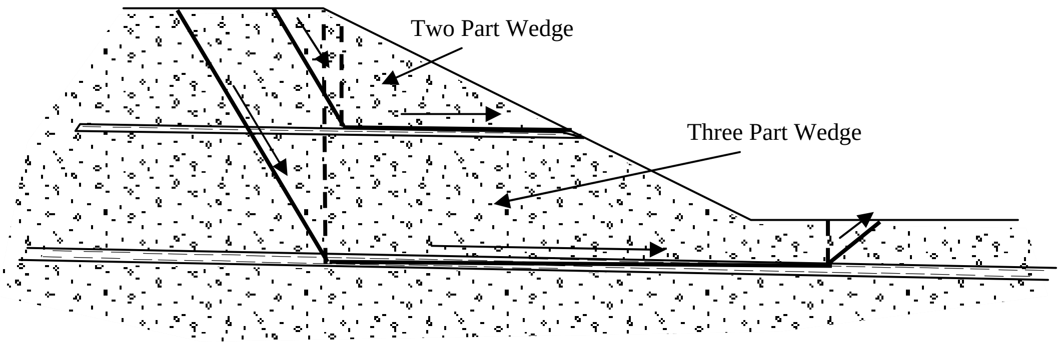


Figure 6.6: Typical Block or Wedge Failure along Sub-Horizontal Planes of Weakness

Summary of factors causing wedge failure

The factors causing another non-circular wedge shaped slope failure can be summarised as follows:

- ∉ in weak layered strata, failures can take place along defined planes. These often include a sub-horizontal sliding surface, with a steep back face;
- ∉ the driving force of this type of slope instability is often high level groundwater pressures acting against the steep back face of the wedge.

6.3 Methods of stability analyses

A detailed treatment of the theory behind slope stability analyses is outside the scope of this report, however, reference may be made to standard geotechnical texts on this subject, including: Lambe and Whitman (1969); Morgenstern and Sangrey (1978) and Chowdhury (1979). The intent here is to review the concepts of stability analysis and to describe some of the methods that are regularly used in practice. This review will serve to explain the combinations of conditions that lead to slope movement and to identify the relative importance to stability of different soil, slope and drainage conditions.

The analytical methods presented are also relatively easy to apply. They make it possible to answer the following questions about the stability of proposed geological sections:

Critical factors relating to the stability of unconsolidated soil sections

- ∉ the rate of soil erosion from exposed soil slopes;
- ∉ the critical height for stable cut slopes;
- ∉ the critical piezometric level in a slope;
- ∉ the effectiveness of drainage, buttressing and other remedial measures.

6.3.1 Prediction of soil erosion rates

Many empirical models have been developed to predict the rate of erosion at any specific location. Most are based on the Universal Soil Loss Equation (USLE) developed in the USA by Wischmeier and Smith (1965). The USLE takes account of the factors listed above by assigning empirical factors based on statistical analysis of erosion measured under natural and simulated conditions. The average annual soil loss per unit area (X) from a site is predicted according to the following relationship:

Estimation of the rate of soil erosion (after Wishchmeir and Smith, 1965)
X = RKSLCP
Where: X = the average annual soil loss per unit area; R = the rainfall erosion index for a given storm period; K = the soil erodibility factor; S = the slope gradient factor; L = the slope length factor; C = vegetation factor; P = erosion control factor.

Methods for the assessment of each of these factors have been published by Gray and Leister (1982). Indeed, the calculations can be performed, interactively, on the internet (http://www.iwr.msu.edu/cgi-bin/rusle/rusle_constr.cgi) although only for counties in the USA.

6.3.2 Limit equilibrium analysis

Limit equilibrium analysis is used to determine the factor of safety for a given slope.

Solutions are available for the analysis of forces along any given surface to determine its factor of safety. However, calculation is easier when the failure surface is an arc of a circle. The derivation of circular failure analyses have most usually been attributed to Fellenius (1936) and to Bishop (1955). The term '**factor of safety'** is expressed as the ratio between the forces available to resist failure (usually the shear strength multiplied by the area of the failure surface) and the sum of forces acting to cause failure, resolved along the failure surface.

Limit equilibrium analysis
$$\text{Factor of safety} = \frac{\text{Sum of restoring forces}}{\text{Sum of disturbing forces}}$$
Limit equilibrium occurs when the factor of safety = unity

When these forces balance, ie generate a factor of safety equal to 1.0, then this condition is known as 'limiting equilibrium'. Calculations of this type are therefore often known as **limit equilibrium analyses**.

Although the calculation methods are relatively simple for any given failure surface, it is often the case that in order to find the most critical surface, the one with the lowest factor of safety, it is necessary to carry out the calculations on a large number of trial surfaces. For this reason it is normal for slope stability analyses to be carried out by computer, using proprietary software.

The analysis of failure along an irregular surface is usually carried out using the method developed by Janbu (1954). The method has the advantage that it allows a great deal of flexibility in the choice of the failure surfaces that can be considered. But this flexibility means that large numbers of slip surfaces cannot as easily be analysed as when circular forms are selected. This makes data preparation for multiple computer analyses tedious.

Limit equilibrium type analyses have some drawbacks, which should be kept in mind. Some of the analytical methods simplify the resolution of disturbing and restoring forces and are therefore not entirely accurate, and different computational procedures tend to give different results. However, this is not considered to be as onerous as, say, the uncertainties which surround shear strength and piezometric level information, which are used as input data for analyses.

Finally, soil slope failures may occur in a progressive manner, often with the initial formation of a tension crack behind the slope crest. Failure then occurs as stresses in different sections of the slope exceed shear strengths at different times. This is particularly common in over-consolidated soils such as London or Oxford Clay.

In spite of these drawbacks, limit equilibrium analyses remain a powerful tool for stability assessment and provides a rational basis for the design of slopes and remedial measures.

6.3.3 Finite element analysis

Many geotechnical problems, including slope stability, can be analysed by proprietary **finite element analysis** computer software. These allow the designer to examine the distribution of stresses and strains in the ground. Complex ground and groundwater modelling can be achieved along with interactions between the ground and built structures (such as a retaining wall). **Finite element analysis** is a powerful design tool, but should only be utilised by operators with a sound soil mechanics background.

In **finite element analysis**, failure conditions are defined by levels of unacceptable soil deformation. For convenience, some software packages will compare acceptability limits with predicted deformation to derive contours of Factor of Safety, which can be used for comparison with the output from limit equilibrium analysis.

6.4 Analytical considerations

6.4.1 Shear strength parameters

Determination of the factor of safety by limit equilibrium methods requires an estimate to be made of the shear resistance that can be mobilised along an assumed failure surface. The shear strength of unconsolidated strata is given by the Coulomb criterion:

Coulomb criterion for soil shear strength

$$\mathbf{s = c + \omega \tan \lambda}\#$$

where:

s = soil shear strength;

c = cohesion;

ω = the stress acting perpendicular to the failure surface (normal stress);

λ = angle of friction.

The cohesion and angle of friction of a soil are known as the shear strength characteristics, and can be determined from suitable laboratory tests carried out on representative soil samples. Alternatively, they can be found by back-analysis based on the geometry of failed slopes, assuming a factor of safety of unity.

Finite element analyses allow the designer to use soil shear strength/deformation models other than Coulomb's. Some of these are more appropriate to soils of different strengths and stress history.

An important consideration in slope stability analyses is whether to employ a **total** or an **effective stress analysis**. This decision will govern the type of shear strength parameters that are used in analysis.

6.4.2 Total stress analysis

In the short term, following construction, saturated clay soils are in an undrained condition (see Section 5.4). Stress induced by the formation of the slope will be taken in part by the soil structure and in part by the interstitial water between clay particles. Theory and observation have shown that under these conditions, soils behave as purely cohesive materials, ie: they have an angle of friction, $\lambda = 0$. The shear strength, defined above is therefore equal to the cohesion (termed 'undrained' shear strength, Cu, in this circumstance), and is independent of normal stress. Undrained shear strength can be determined readily from a number of laboratory and in situ methods.

Total stress analyses are applicable where insufficient time is allowed for pore pressure dissipation to occur. A temporary excavation in a clay soil can be analysed in this manner.

A total stress analysis does not require a determination of groundwater pressures within the slope.

6.4.3 Effective stress analysis

When groundwater pressures are governed by steady state seepage conditions, or where long-term stability is a consideration, then stability analysis should be performed in terms of effective stresses. All permanent excavations should be analysed for the long term conditions, and it is often the case that these describe the critical stability condition.

An effective stress analysis requires that drained shear strength parameters be used and that the pore water pressure distribution be known from groundwater studies. Effective strength parameters (c' and λ') may be obtained from the results of specialist laboratory testing. The use of 'peak' shear strength parameters in any analysis will give rise to better (often considerably so) factors of safety than 'residual' shear strength parameters.

In the context of this study, it would be expected that effective stress analyses would be carried out, as the slopes in question would be required to remain stable in the long-term, with little or no provision for maintenance regimes.

6.4.4 Translational slope failure

The stability of simple natural slopes, where all boundaries (including ground and groundwater surfaces) are approximately parallel to each other, can be modelled and analysed by the so-called, **‘infinite slope’** equations. In this analysis, the slip surface is assumed to be a plane, roughly parallel to the ground surface. The following types of slopes or slope conditions meet these criteria:

Ground conditions where the infinite slope solution may be used

∉ loose products of weathering (residual soils) overlying an inclined stronger sub-stratum;

∉ bedrock slopes mantled with glacial or colluvial deposits;

∉ homogeneous slopes of coarse textured, cohesionless soils.

In this way, the analysis is applicable to the shallow subsurface sliding mode of slope instability discussed in Section 6.2. Because of the geometry of an infinite slope, overall stability can be determined by analysing the stability of a single, vertical element in the slope as shown in Figure 6.7.

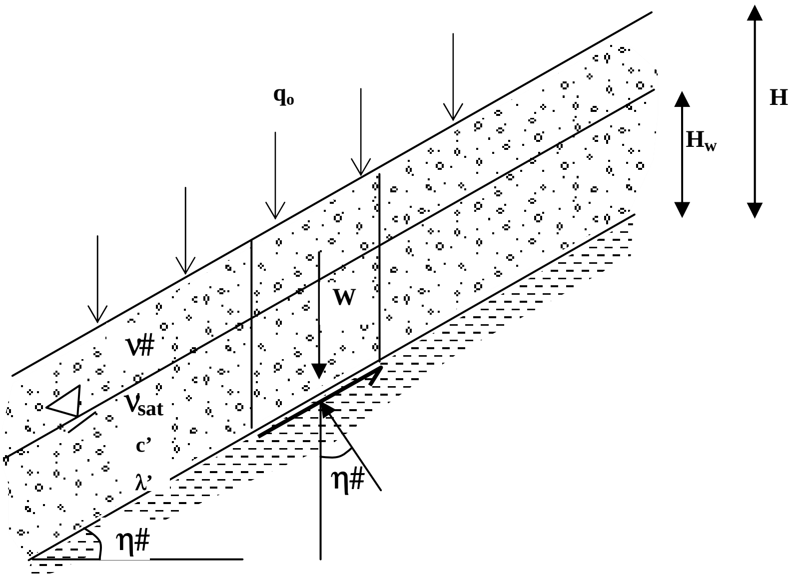


Figure 6.7: Sliding Along an Infinite Slope

End effects in the sliding mass can be neglected, as can lateral forces on either side of the vertical element which are assumed to be opposite and equal. The factor of safety, based on an infinite slope analyses is given by the following equation:

$$FOS = \frac{\{[c'/(\cos^2 \eta \times \tan\lambda')] + (q_o + \nu H) + (\nu_{sub} + \nu) \times H_w\} \times (\tan \lambda' / \tan \eta)}{[(q_o + \nu H) + (\nu sat - \nu) \times H_w]}$$

Where:

λ' = drained angle of friction
c' = drained cohesion
η = slope angle
ν = moist soil density
ν_{sat} = saturated soil density
ν_{sub} = submerged soil density
ν = density of water
H = thickness of sliding mass
H_w = height of groundwater above plane of sliding
q_o = uniform surcharge load on slope

The equation is completely general, and takes into account the influence of surcharge, groundwater and soil cohesion. However, the expression can be simplified in certain, more specific cases as follows:

∉ Cohesionless soil, no surcharge

$$FOS = \frac{[\nu \times (H - H_w) + \nu_{sub} \times H_w] \times \tan \lambda'}{[\nu \times (H - H_w) + \nu_{sat} \times H_w] \times \tan \eta}$$

∉ Cohesionless soil, fully saturated, no surcharge

$$FOS = \frac{\nu_{sub} \times \tan \lambda'}{\nu_{sat} \times \tan \eta\#}$$

Where ν_{sub} = half ν_{sat} (approx) then

$$FOS = \frac{0.5 \times \tan \lambda'}{\tan \eta}$$

∉ Cohesionless soil, dry, no surcharge

$$\text{FOS} = \frac{\tan \lambda'}{\tan \eta\#}$$

This shows that for a dry, granular material, the critical slope angle is equal to the angle of friction for the soil deposit; however the factor of safety is halved if fully saturated. This demonstrates the considerable influence of slope drainage on slope stability.

∉ Cohesive soil, no surcharge (FOS = 1)

Dry slope ($H_w = 0$): $c'_d/\nu H = \cos^2\eta \times (\tan\eta - \tan\lambda')$

Saturated slope ($H_w = H$): $c'_d/\nu H = \cos^2\eta \times (\tan\eta - (\nu_{sub}/\nu_{sat}) \times \tan\lambda')$

These equations are useful for determining the amount of cohesion required to maintain minimum stability requirements.

∉ Rotational failure

Deep seated, rotational failures occur in many types of soils, but most commonly in cohesive deposits, and especially in cut slopes. As described in Section 6.2,, solutions exist for almost any irregular shaped failure surface; however, relatively simple solutions exist for failure surfaces that approximate to an arc of a circle in cross-section. For this reason, these are most often applied to slope stability design problems. It is therefore most likely that the following methods would be used for the design of new, temporary and permanent geological sections.

Where the effects of groundwater can be ignored, it is possible to carry out **'total stress'** analyses, ie: those for which it can be assumed that $\lambda = 0$, it is possible to use stability charts, for instance those developed by Taylor (1948). Stability charts are useful for determining the critical height of cut excavation for specific combinations of soil properties and slope geometry. The critical slope height (H_c) is maximum height at which the slope will remain stable. This height is found from a so-called **'Stability Number' (N_s),** using the Taylor's chart, shown in Figure 6.8 The Stability Number is related to the critical height by the following expression:

Taylor's stability equation

$$N_s = H_c \times (\nu/C_u).$$

Where:

H_c = Critical (maximum stable) slope height
ν### #Soil Density
C_u = Undrained Shear Strength

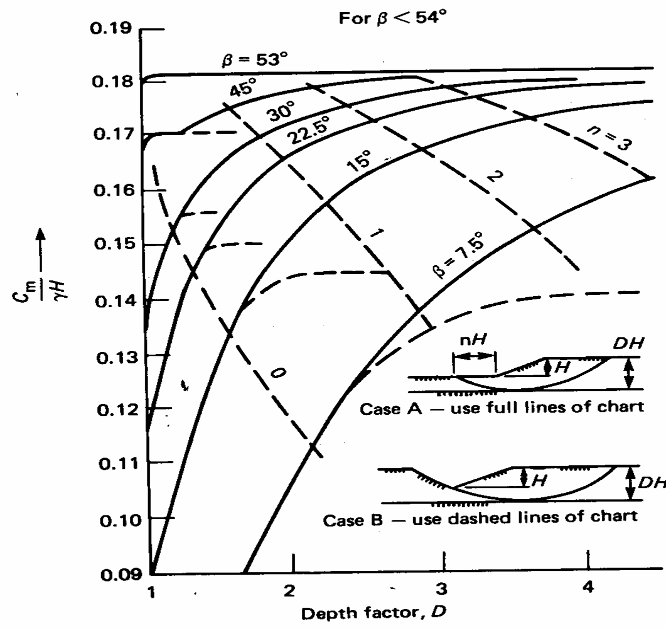
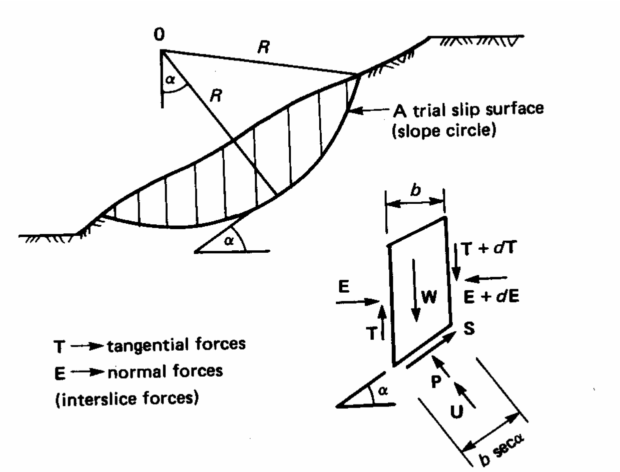


Figure 6.2: Taylor's Stability Chart for λ = 0 analysis

The estimation of critical height of cut slope and comparison with design cut heights from stability charts is useful for preliminary planning and siting. However, because these charts are derived in terms of **total stresses**, they are of limited value when seepage is present at the slope face, or where groundwater is present within the soil mass above the critical failure surface.

Where groundwater and seepage characteristics must be taken into consideration, then the most common analysis method is that known as the 'method of slices' (although it is also valid for total stress slope stability analysis).

The complete theoretical derivation of the procedure is beyond the scope of this report, however, the general principle is illustrated in Figure 6.9.



$$F = \frac{\sum \frac{cl\cos\alpha + (W - ul\cos\alpha)\tan\phi}{m_\alpha}}{\sum W\sin\alpha}$$

in which

$$m_\alpha = \left(1 + \frac{\tan\alpha\tan\phi}{F}\right)\cos\alpha$$

Figure 6.9: Method of Slices – Typical Force Distribution

In what is usually termed the simplified Bishop method, force equilibrium of a slice in the vertical direction is taken, and the variation in the horizontal across the slice is ignored. The stability equations shown in Figure 6.3 are solved by an iterative process until the Factor of safety (F) assumed on the right hand side, for a given failure surface equals the value computed on the left hand side. This procedure must be repeated for several trial surfaces until the lowest factor of safety, and hence the critical surface, is determined. The method is readily adaptable for computerised analysis, and many proprietary sloped stability software packages are available.

6.5 Factor of safety

This section provides guidance on the selection of appropriate factors of safety for temporary and permanent stability of exposed geological sections.

The selection of an appropriate design **Factor of safety** requires consideration of all relevant performance requirements, as well as an assessment of the reliability of input data. These decisions should be made using sound engineering judgement, by an experienced geotechnical engineer.

Relatively high **Factors of safety** (more than 1.5) are required where site investigation data is of either limited quality or quantity, or where the consequences of failure could be life threatening or endanger the property of third parties.

Where the consequences of failure are less drastic, or where there is better confidence in design data then a lower factor of safety may be applied. It is common to find designers aiming for a **Factor of safety** of 1.3 using peak shear strength parameters.

The onus here on **English Nature** is to ensure that when required to oversee the design of a conservation section, it will be essential to ensure the highest quality of design data, in order to optimise the required **Factor of safety**.

It has been shown that conservative designs are often produced by engineering consultants as a result of potential for future litigation in the event of failure, and even under instruction from developers and their funders who require a **'no risk'** solution to stability issues. Conservative design may arise where high **Factors of safety** are adopted, using lower bound shear strength values and pessimistic groundwater assumptions for example. It is equally possible, that conservatism in design is fully justified where the consequences of failure are very serious. With limit equilibrium design methods, the **Factor of safety** is an average for the slip surface analysed. By contrast, **finite element analysis** identifies stress and strain concentrations, and allows the slope to be contoured in terms of stress strain or factor of safety.

The design process should be geared to providing a safe structure for the duration of its intended use. During the early to mid 1990's, attempts were made to standardise geotechnical design procedures throughout the European Union. These culminated in the publication in 1994 and 1997 by **Eurocode 7: Geotechnical Design**. These were published in three parts as follows:

∉ Part 1: General Rules (1994);

∉ Part 2: Design Assisted by Laboratory Testing (1997);

∉ Part 3: Design Assisted by Field Testing (1997).

These Eurocodes describe the processes that include appropriate analyses as well as adequate data collection and definition of construction objectives and can be summarised as follows:

Processes of appropriate data collection and analysis (Eurocodes)

∉ data required for design are collected, recorded and interpreted;

∉ structures are designed by appropriately qualified and experienced personnel;

∉ adequate continuity and communication exist between personnel involved in data collection, design and construction;

∉ adequate supervision and quality control is provided;

∉ execution is carried out according to relevant standards and specifications;

∉ construction materials and products are used as specified.

The **Eurocodes** formalise the use of partial **Factors of Safety** and **limit state design** in geotechnical engineering. Calculations are carried out using **characteristic values** (these are values that are sufficiently conservative as to provide confidence that actual or in situ properties exceed design values) of the material parameters with partial factors applied, depending on data reliability and consequences of failure, to obtain **design values**. Following the application of partial factors, the calculations are performed in the same manner as for the traditional approach. However, in this case a **Factor of Safety** of greater than 1.0 only is required.

6.6 Risk assessment

Risk assessments are now part of the everyday engineering decision making process in the mining and construction industry sectors. Risks should first be understood and recognised, and then evaluated, before they can be managed. In the UK, professional practice requires that engineers should:

∉ 'Give due attention to risk analysis evaluation, decision making, implantation and monitoring during all phases of an engineering project to ensure effective management of risk;

∉ 'seek to ensure that management systems do not allow risk issues to be ignored, subverted or delegated to levels which have no control'.

Most risk assessment methodologies follow the **Source** → **Pathway** → **Target**

Model, established by the Environment Agency, whereby the **Hazard** is a combination of the source and the potential pathways via which a release or effect will travel or migrate, and the **consequence** is the effect that this has on the **target** or ultimate interest on which an adverse impact is manifested. A particular source may have more than one pathway, or combination of pathways, affecting more than one target group.

In the case of slope stability **'Hazard'** can be considered to be slope instability where the consequences can be:

∉ risk to human safety;

∉ risk to property at the foot of the slope;

∉ risk to property beyond the crest of the slope.

6.7 Key points

Section 6 of this report has attempted to provide an overview of the subject of slope stability as it relates to the formation of stable excavated slopes in unconsolidated sediments.

It should be noted that it has not been the intention to prepare a manual for the report users to carry out slope design. Rather, the objective has been to demonstrate the various **mechanisms** of instability that can arise from different ground conditions; and to point towards the **type** of stability analysis that are most appropriate under these conditions.

In Section 6.2, the report described six of the most common types of slope instability that occur in unconsolidated sediments. These can be summarised as follows:

Common mechanisms of slope instability	
surface erosion:	Slow surface degradation under the action of direct rainfall (and run-off) and wind. Most pronounced in granular soils.
groundwater erosion:	Slow surface degradation caused by seepage out the slope face. Occurs in bedded soils with layers of varying permeability.
shallow subsurface sliding:	Movement of weak surface soils down a slope (often natural or aged cut slopes), usually when they become saturated. Can be slow, when it is known as surface creep (soliflixion) or rapid when they are often referred to as mudslides.
rotational failure:	Failure of potentially large quantities of ground along a surface that may approximate to an arc of a circle. Often occurs in homogeneous cohesive soils. Can develop over relatively short period of time.
irregular surface failure:	Similar form as the rotational failure except that the failure surface is not circular. The irregularity may result from non-homogeneous ground conditions, or as a result of progressive failures that migrate up-slope.
wedge failure:	This mechanism can also result in large scale slope instability, with large ground movements occurring relatively quickly. Sliding usually occurs along a preferred plane of weakness, with groundwater providing the driving force.

The general theory behind the various methods of stability analysis that can be used to investigate the potential influence of each of the described failure mechanisms has been described.

Finally, guidance has been given on the selection of shear strength parameters and the use of appropriate factors of safety.

7. Site investigation and data collection

7.1 Background

There are many published guidance documents for carrying out site investigations. Authoritative works include those by the Association of Geotechnical Specialists (1991), the British Drilling Association (1992), Craig (1995) and Weltman and Head (1983). However, site investigation works are also covered in the United Kingdom by British Standard BS5930:1999, "A Code of Practice for Site Investigation".

This section of the report draws from the British Standard, and discusses aspects of site investigation as they relate specifically to obtaining information that can be used for the design of stable slopes in unconsolidated soils. However, it is recommended that anyone preparing a site investigation for the design and management of geological sections should refer to the British Standard document directly.

The main objectives of a site investigation with respect to the formation of stable geological sections are to provide the information required to arrive at rational and economic designs. Without the results of a well structured site investigation, designers are forced to make design assumptions on visual inspection or available published values. This inevitably results in either conservative design, or worse, slope failure.

The scope of an investigation will depend on the nature of the expected ground conditions, the height of the proposed conservation section and whether information is required for other purposes, for instance the design of permanent structures. An investigation must also work within constraints of available time and budget. The investigation should also be planned to ensure that more than one conservation option can be prepared. These options can later be evaluated to determine the most appropriate solution to any given situation.

Site investigation works often fall into four stages, which can be summarised as shown in Table 7.1.

The reconnaissance and desk study stages may be undertaken collectively. These four stages can be considered to be the data gathering stages, to be followed by interpretation and design.

Table 7.1: Typical stages of site investigation	
Work stage	**Comments**
Site reconnaissance	Based on a walk-over inspection of the site; identification to physical constraints to the investigation works.
Desk study	Based on documented information relating to the site. In particular, identifies historical or buried constraints to the investigation works.
Intrusive investigation	Physical inspection of the ground conditions through the excavation of trial pits, boreholes or other means, and the collection of representative samples.
Laboratory testing	Geotechnical and chemical testing of representative samples to obtain appropriate slope stability design data.

As work proceeds through any of these stages it is necessary to review the information as it becomes available and, if necessary, modify the planned scope of works.

Some of the procedures used during these investigation stages are described in the following sections.

7.2 Reconnaissance

Much useful information can be obtained simply by visiting and walking over a site. Important features to be noted include: topography, drainage, vegetation and available geological exposures. Local land use can be assessed and routes for investigation plant can be planned. Such access can be problematic, particularly where sites have rapidly varying topography and property boundary constraints. The latter frequently places limits on the positions and type of investigations that can safely be performed.

Typically, the observations made during a site reconnaissance visit will include some or all of those shown in Table 7.2, although this list should not be considered to be exhaustive. All sites should be considered as being unique and will present individual challenges with respect to access and data collection.

Table 7.2: Typical observations made during a site reconnaissance visit

- additions and omissions on available plans, such as new or demolished structures, roads and power lines;
- topography;
- orientation, height and angle of slopes;
- details of existing structures;
- details of obstructions such as power lines, trees, ancient monuments and pipelines;
- signs of surface instability;
- water levels, direction and rate of flow of rivers, streams and canals;
- extent and status of mine or quarry workings;
- observe and record any obvious hazards to public health and safety;
- observe and record any areas of discoloured soil, polluted water, distressed vegetation or significant odours.

7.3 Desk study

The principal aims of the desk study are essentially the same as the site reconnaissance, in that information is collected that may be of value to the design of intrusive site investigation works as well as the design of the proposed project. Where the two differ is that the site reconnaissance is based on a visual inspection and a site visit, while the desk study uses published and unpublished records in the public domain. Sources of such information include: local authorities, libraries and County Records Offices. Geological data are also available from the British Geological Survey and from published maps and memoirs. There is also much information available on the internet, which can be downloaded in digital formats.

Copies of old maps, plans and photographs are often available from libraries and public records offices. These can often provide information relating to the historical usage of a site. Such maps have been produced for the whole of the United Kingdom at 1:2,500 scale by the Ordnance Survey since the late 19th century, but older maps may also be available for some areas, for instance local tithe maps were produced by some administrations for the purpose of tax collection.

In an area that may have been subject to historic and/or current coal mining it is recommended that a Coal Mining Report is obtained. These are prepared by the Coal Authority, which is the body responsible for administering the inherited liabilities from former coal mining activities. The Coal Mining Report will provide details on the presence and depths of recorded underground mine workings and the approximate date that these ceased. The extent and proximity of past, present and future opencast coal operations will also be reported. Often of great value are records of abandoned mine entries (shafts or adits) that may be within the site, or within a distance of 20 metres from the site boundary. Such entries may not have been adequately stabilised at the time of abandonment, and therefore present a risk to future site users.

Historically, the Health & Safety Executive Inspectorate of Mines was the repository for abandonment plans of mines other than coal. In the early 1990’s, the plans held were disseminated to their relevant county records office where, coupled with other information already held, they form a source of primary importance.

In addition to obtaining geological and topographical data, information relating to statutory services and utilities that may be buried beneath the ground surface in the vicinity of a site is ideally collected at desk study stage. Information relating to major trunk and supply routes will be available from the various utility or service providers such as Transco, BT and the various power and water supply companies (a list that has lengthened considerably in recent years). However, these providers may not be able to provide information on local supply services passing beneath private land.

Large amounts of information are now contained on commercially available databases. Several companies (EnviroCheck, Sitesearch) administer and update these databases, and can provide information in digital formats within a few days or less, by electronic transfer. Information held on these databases includes, among others, those listed in Table 7.3.

Table 7.3: Types of information held on commercially available site databases
∉ historical mapping; ∉ positions of nearby licensed water abstraction and discharge consents; ∉ positions of nearby licensed landfill and waste transfer sites; ∉ records of reported pollution incidents; ∉ groundwater vulnerability; ∉ surface water quality; ∉ river flood data; ∉ recorded previous contaminative site uses; ∉ a register of local borehole records held by the BGS; ∉ geological environment; ∉ coal mining affected areas; ∉ sensitive land uses.

An important output from the desk study and site reconnaissance work is the Designer Risk Assessment. This is now a requisite under The Construction (Design and Management) Regulations (1994), usually referred to as CDM. Dangers presented by steep slopes, buried or overhead services, and hazardous substances within the ground can, through careful planning, be avoided. However, where this is not possible, the purpose of the Designer Risk Assessment is to ensure that subsequent site operations are undertaken in the full knowledge of site conditions, and that where possible, risks to site personnel and the general public are identified and minimised. CDM regulations also place duties on clients, employees and contractors. In particular, companies contracted to carry out site investigation works should prepare a site safety plan before commencing their works.

In summary, the typical sources of information that may be consulted during the desk study phase of a site investigation are summarised as in Table 7.4:

Table 7.4: Typical sources of information that may be consulted during the desk study phase of a site investigation

Information source	**Type of information supplied**
Local Authorities	Planning history, details of recent, nearby construction.
Libraries and County Records Offices	Historical maps, local historical records and manuscripts.
British Geological Survey	Geological maps, regional geological guides and memoirs. Historical borehole records.
Ordnance Survey	Historical and current topographic maps, aerial photographs (since 1980).
English Heritage	Aerial photographs (pre 1980).
The Coal Authority	Coal Mining Report.
Health and Safety Executive and County Record Offices	Non-coal mining records and other hazards.
Utility and Service Providers	Records of buried and overhead services and utilities.
Commercial Databases	Catalogued public records – See Table 7.3.

7.4 Intrusive site investigation

7.4.1 Introduction

The locations of points of exploration, such as boreholes, probes or trial pits should be selected so that they provide a complete geological and hydrogeological perspective of the site. At least one exploration point should be located behind each section where a conservation exposure is planned. Although there are no defined rules, a minimum of three exploration points should ideally be selected at each site at spacings of between about 10 and 30 metres, although this may vary depending on the size and accessibility of the exposure site.

The depth of the exploration points should be sufficient to permit the assessment of the stability of the future exposure slope. This may mean penetrating the full depth of weak strata, when present at and immediately below the toe of the proposed exposure. British Standard BS8002:1994 “Code of Practice for Earth Retaining Structures” recommends that borehole depth, below excavation level, should be at least three times the proposed retained height.

The method of investigation must be able to provide information on the physical characteristics of the ground as well as groundwater conditions. This may include provision to investigate for artesian water if appropriate.

Although thorough ground characterisation is the most important aspect of the selection of exploration point location and method, this may also be influenced by other site features. The topography, access restrictions or structures may have an influence on the method of investigation in these cases. Where the working position is on steeply sloping ground, it may be necessary to form horizontal working platforms, using scaffolding, or by the importation of fill to create access routes. These will have an influence on site investigation costs.

In view of the importance of ground investigation in providing data on which engineering design will later rely, site works should only be carried out by experienced personnel. This will normally be by a specialist site investigation contractor. The works should be supervised by a suitably qualified geotechnical engineer. Depending on the nature of the deposits being investigated it may also be appropriate for a specialist geologist and/or archaeologist to be present during intrusive works in order to maintain a watching brief of materials uncovered.

Methods of intrusive exploration most commonly comprise trial pits, boreholes and probes. The relative merits of these, with particular reference to the establishment of geological sections are discussed below.

7.4.2 Trial pits

Shallow trial pits are usually excavated using an hydraulic back-acting excavator. Unless shoring is used to maintain the excavation sides, this method of exploration is restricted to ground that can stand unsupported for the period of time required to complete soil inspections. In most cases, it is only practical to excavate trial pits to a maximum depth of between 4 and 5 metres, but even at these depths, the quantity of ground that is disturbed may be considerable (between 5 and 10 cubic metres). For this reason, the use of trial pits in areas of geological conservation may be inappropriate. However, this method of exploration has advantages over others in that it can indicate variations in ground conditions that occur laterally as well as vertically. While it may be unsafe for engineers to enter even shallow trial pits (it is prohibited to enter an unsupported excavation deeper than 1.2 metres for health and safety reasons) ground conditions can be recorded from inspection of materials brought to the surface in the excavator bucket. Disturbed samples can be collected for later laboratory testing, and simple index tests can be carried out. The most common of these is the shear vane test. This provides a measure of the soil shear strength and can be carried out on blocks of cohesive soils brought to the ground surface.

A trial pit record (a typical example is shown in Figure 7.1) should include its position, orientation and ground surface elevation. The record should also provide a visual representation of the strata encountered together with detailed written descriptions, prepared in accordance with the guidance given in BS5930:1999. The descriptions should also refer to any lateral variations in ground conditions. Notes should include:

- the orientation of the face(s) that have been described;
- the stability of the excavation sides;
- the results of insitu testing;
- the depth and type of any retained soil or groundwater samples;
- the depth and approximate flow rate of any groundwater seepages;
- the presence of any unusual odours.

7.4.3 Boreholes

Boreholes are used extensively in site investigation. They enable exploration to greater depths than is possible with trial pits, while generating considerably less disturbance to valuable geological strata. For instance, a 150 mm diameter borehole, 10 metres deep, would generate less than 0.25 cubic metres of disturbed ground.

Boreholes can be progressed through almost any type of strata. The two most common methods in use within the United Kingdom are percussion boring and rotary drilling. While other methods, such as wash boring and power augering, are rarely used, or used more extensively overseas, these are considered inappropriate for geotechnical design work and will not be discussed further.

CONTRACT NO.: NL05839

EXCAVATED FOR: Anyclient Ltd
LOCATION: Anyplace, Anytown

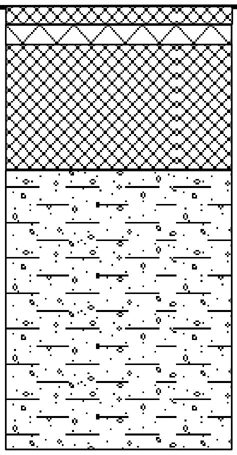
NATIONAL GRID REF.: SJ 527 856
GROUND SURFACE LEVEL: 87 m aOD
DATE EXCAVATED: 21 April 1999
DATE BACKFILLED: 21 April 1999

Sheet 1 of 1

TRIAL PIT NO.: 3

WIDTH: 0.60 m
LENGTH: 2.00 m
EXCAVATED USING A JCB 3CX
VERTICAL SCALE 1:50
ORIENTATION OF DESCRIBED FACE : North

DEPTH	LEGEND	STRATA DESCRIPTION
Ground Level – 0.05 m		MADE GROUND: Thin black tarmac
0.15 m		MADE GROUND: Red brown ash.
1.20 m		MADE GROUND: Soft to firm grey and orange brown sandy clay with grey sandy partings, small black carbonaceous partings and rare sub-rounded sandstone gravel.
3.30 m		Stiff dark grey sandy CLAY with much fine and medium sub-rounded sandstone gravel with rare cobbles. (Glacial Till)



Depth scale: Ground Level, 1.00m, 2.00m, 3.00m, 4.00m, 5.00m

SHEAR STRENGTH (HAND VANE)
kN/m^2

0.50	41, 38, 38
1.00	36, 42, 40
1.50	80, 98, 90

REMARKS

1/ Trial Pit stable.
2/ Trial Pit dry
3/ No unusual odours
4/ Trial pit terminated at 3.30 m in natural strata

SAMPLES

0.40	BB	Made Ground
0.45	SB	Made Ground
1.50	SB	Sandy Clay
2.50	SB	Sandy Clay

Drawn By	Checked By

Appendix
1

Figure 7.1: Example Trial Pit Record

7.4.4 Cable percussion boring

For site investigations in soils, the most commonly seen tool is the light cable percussion boring rig, Figure 7.2. The rig comprises a wire rope winch of 1 or 2 tonne capacity, which is driven by a diesel engine, and a derrick of about 6 metres height, over which the boring tools are controlled. The rig can be folded down to form a trailer that can be towed behind a light vehicle.

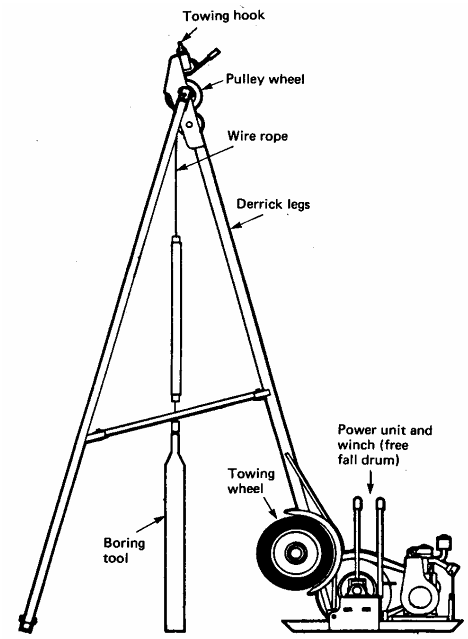


Figure 7.2: Cable Percussion Boring Rig

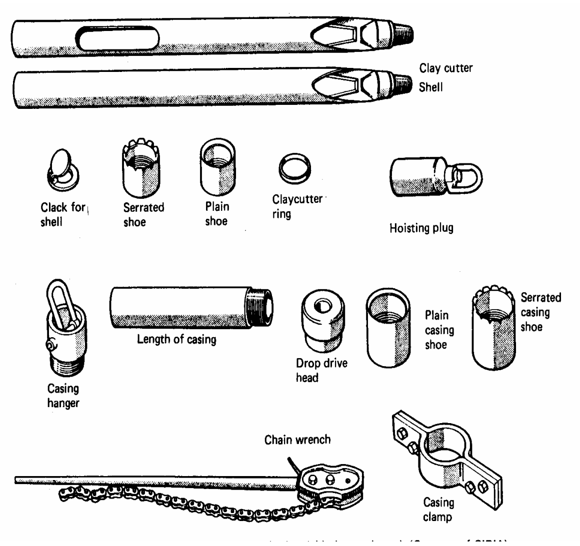


Figure 7.3: Typical tools used during Cable Percussion Boring

Boreholes are advanced by percussion using weights to drive a sampler, usually 150mm diameter (although hole diameters of up to 300mm can be achieved) a short distance into the ground. By returning the sampler to the ground surface the sample can be retrieved. The sides of the borehole can be maintained by driving steel casing with a diameter that is slightly larger than the sampling tool. The latter casing can be omitted when boring through self-supporting cohesive strata, but must be introduced when encountering granular soils. Two tools are used to advance the borehole.

In cohesive soils, a ‘clay cutter’ is used and relies on the soil being retained within an open steel tube or cruciform. However in granular (cohesionless) soils such as sand and gravels, a bailer is used (also known as a ‘shell’). This method relies on there being sufficient water in the borehole to cover the non-return valve located near the base of the bailer, and therefore in dry granular strata this may require the introduction of water. Typical boring tools are shown in Figure 7.3.

Whenever obstructions, such as cobbles or boulders are encountered, these must be dislodged or smashed using a chisel tool, basically a heavy steel bar which is attached to the winch rope and allowed to freefall to the base of the hole.

The boring method allows several different sampling methods to be used. These produce samples of varying quality, which dictates the use to which the sample can be put, particularly in respect of laboratory testing. These sampling methods are described in Table 7.5.

Table 7.5: Sampling methods used during cable percussion boring

Sample type	**Method of collection**	**Suitability for geotechnical testing**
Disturbed Samples (Cohesive)	The collection of disturbed samples of cohesive soil from the clay cutter borehole advancing tool. The sample is usually of between 1 and 10kg weight and is preserved inside a plastic or glass container, or heavy-duty polythene bag.	The sample is suitable for inspection and for preparing engineering soil descriptions. Most soil classification tests can be carried out on such samples, although care should be taken when scheduling soil moisture content tests on samples collected underwater.
Disturbed Samples (Granular)	Disturbed samples of granular soils can be collected from inside the bailer.	These generally provide samples that are suitable for engineering soil description, however, the sampling method may result in the loss of fine particle fractions, making them inappropriate for classification testing.
U100	Open drive (U100) samples of cohesive soils are obtained by driving a 100 mm diameter by 450 mm long steel or alloy walled tube into the base of the borehole using a trip hammer. The samples are retained within the tube during transit to the laboratory where they can be extruded for examination and testing. This type of sample is often called ‘undisturbed’, and while this may be the case when the soils are firm or stiff, softer, sensitive soils (especially silts) may undergo extensive disturbance.	Where only minor disturbance has occurred, the samples may be used to carry out shear strength and compressibility testing, as well as classification tests. Alternatively, the samples can be split to allow examination of strata bedding or other structures.

Table 7.5: Sampling methods used during cable percussion boring		
Sample type	**Method of collection**	**Suitability for geotechnical testing**
Piston Samples	In soft and/or sensitive soils (soils that lose strength on disturbance), where shear strength and compressibility testing is required, piston samples can be recovered. These are collected within thin-walled sampling tubes to reduce sample disturbance. The sample tube is progressed into the soil by static pressure rather than dynamic impact; and retained within the tube during withdrawal by the vacuum generated between the piston and the top of the sample.	Samples may be used to carry out shear strength and compressibility testing, as well as classification tests. Alternatively, the samples can be split to allow examination of strata bedding or other structures.
Split Barrel Sampler	The split-barrel sampler is used in the borehole standard penetration test (described below). It recovers disturbed samples of 35 mm diameter. The sampler can be used in both cohesive and granular soils, although its use in gravelly soils should be avoided.	The sample is suitable for inspection and for preparing engineering soil descriptions. Most soil classification tests can be carried out on such samples.

Insitu tests of various types can be carried out within boreholes formed by cable percussion methods (Table.7.6).

Table 7.6 Typical borehole tests	
Type of test	**Purpose of test**
Standard Penetration Test (BS 1377)	∉ Sample collection; ∉ assessment of relative density; ∉ results can be correlated with a wide range of shear strength, compressibility and deformability characteristics.
Borehole Vane Test (BS 5930)	∉ Determination of insitu shear strength.
Falling/Rising Head Test	∉ Determination of soil permeability.

The most commonly used borehole test is the **standard penetration test (SPT).** This is a dynamic penetration test carried out in accordance with the methods described in BS1377-Section 9:1990. The test uses a 50 mm (outer diameter) thick walled split-barrel sample tube which is driven into the ground at the base of the borehole by blows from a standard weight falling through a standard distance. The number of hammer blows to drive the sampler a distance of 300 mm into the ground (after an initial seating drive) is known as the ‘N’ value. In gravelly soils, the split sampler can be replaced with a solid cone of similar length and diameter and a 60^o apex angle. The test is empirical, but is supported by a considerable body of published data accumulated over many years and which relates the SPT ‘N’ value to other soil properties. The great merit of the test, and the main reason for its widespread use is that it is simple to carry out and relatively inexpensive. The soil parameters that can be inferred from the results of SPTs are approximate, but can give a useful guide to ground conditions.

In soft and firm soils (and especially sensitive soils) the **borehole vane test** can be used to measure the insitu shear strength. The test is carried out by introducing a cruciform vane, attached to a solid rod, into the soil at the base of a borehole and rotating it until the soil shears. The torque required to shear the soil can be related to the peak shear strength. The general arrangement of the test set up is shown in Figure 7.4 and full details describing the test procedure are included in BS1377-Section 9:1990. The results of this type of test are questionable in stiff or fissured soils, or where soils tend to dilate on shearing.

Soil permeability can be determined within boreholes. In simple terms, the test must be carried out below the groundwater, and is performed by either introducing or extracting water from within the borehole (hence they are known as falling or rising head tests) and measuring the response time for the water level to return to its rest level. Care must be taken to ensure that both borehole and surrounding ground and groundwater conditions are fully understood in order to apply appropriate correction factors.

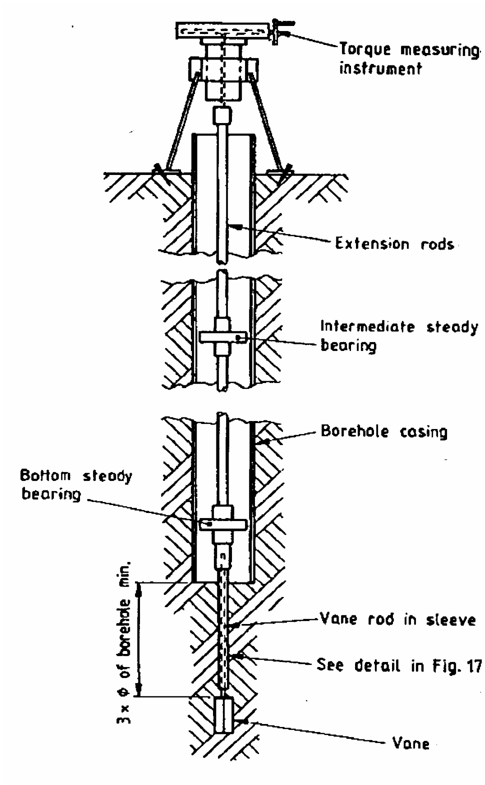


Figure 7.4: Test Arrangement for Borehole Vane Test

Boreholes produced by cable percussion methods are suitable for the installation of groundwater monitoring instrumentation. Groundwater observations made during the drilling works provide valuable information, but may not accurately reflect pore water pressure conditions in the ground. Monitoring instrumentation varies from the use of standpipes, comprising a slotted plastic pipe and surrounded with sand, to porous ceramic piezometers, that can be set at a particular level and provide information on groundwater pressures.

A borehole record (a typical example is shown in Figure 7.5) should include its position, orientation and ground surface elevation. The record should also provide a visual representation of the strata encountered together with detailed written descriptions, prepared in accordance with the guidance given in BS5930:1999.

Start date	17 October 1989	Casing diameter	200 mm to 8.00 m 150 mm to 12.00 m 100 mm to 14.00 m
End date	21 October 1989		
Drilling method	Cable percussion to 12.00 m	Borehole diameter	200 mm to 8.50 m 150 mm to 12.00 m 100 mm to 21.50 m
Equipment	Rotary coring to 21.50 m TH6 core barrel, water flush		

BOREHOLE No.	1
National grid Coordinates	5423.0 E 4256.0 N
Ground Level	33.68 m OD

Date and time	Casing depth (m)	Depth to water (m)	Sample details: Depth (m) from to	Type	No.	U 100 Blows / SPT Blows/N	U 100 Rec. / SPT Drive	Description of strata	Depth (thickness) (m)	Level m OD	Legend
17/10 15.00	NIL	DRY	0.50	D	1		-	Friable brown gravelly TOPSOIL	(0.40) 0.40	33.28	
	NIL	DRY	0.50 - 1.45	U	2	40	450	Stiff fissured brown mottled yellow and light grey CLAY. Frequent rootlets. Fissures are very closely spaced, subvertical, rough. (ESTUARINE DEPOSITS - DESSICATED CRUST)			
			1.50	D	3						
			2.00 - 2.50	B	4				(2.70)		
	NIL	DRY	2.90 - 3.35	U	5	32	450				
			3.40	D	6				3.10	30.58	
18.00 17/10	2.50	DRY	4.00	D	7			Firm brown and dark grey mottled CLAY. Occasional rootlets. (ESTUARINE DEPOSITS)	(0.80)		
18/10 08.00	2.50	DAMP DRY	4.50 - 5.50	D	8	-	1000		3.90	29.78	
								Soft grey and dark grey CLAY with closely spaced sub-horizontal partings and thin laminate of light grey fine sand. (ESTUARINE DEPOSITS)			
			5.40			HV	(22/6)				
	5.50	DRY	6.00	D	9	FV	(25/6)	Occasional shell debris, from 6.00 m to 8.50 m			
	6.00	DRY	6.50 - 7.50	NR	-				(4.60)		
			8.00 - 8.45	D	10	-	450				
								Possibly medium dense, light brown slightly gravelly fine and medium SAND. gravel is fine and medium of rounded quartz and sub-angular limestone (ALLUVIAL DEPOSITS)	8.50	25.18	
	8.00	6.50	8.70 8.70	D D	11 12						
	9.00	0.00	9.00	D	13				(1.50)		
	10.00	0.00	10.00	D	14	C 30			10.00	23.68	

Remarks
1. See key sheets for explanation of abbreviations and symbols
2. An inspection pit was excavated by hand to 0.6 m
3. Small amounts of water were added to assist boring from 0.6 m to 3.50 m
4. Ground water was encountered at 8.50 m rising to 6.50 m to 3.50 m
5. SPT blows were at 9.00 m 1.1.0.1.3.5.5.6.9.7. Test was extended due to initial blows. Final 300 m was used to derive N value

Logged by	DRN 23/10/89
Compiled by	ANO 23/10/89
Checked by	VIP 23/10/89

Project	CATCAIRN BUSHES, HAMPSHIRE Notable Developments Limited
Contract No.	5903
Sheet No.	Sheet 1 of 3

Figure 7.5: Typical Borehole Record (after BS5930:1999)

The light cable percussion boring method is the stalwart of site investigation operations. Its basic design has altered little over the past 50 years, although methods of interpretation of test data and of the results of testing of the samples recovered have advanced with the discipline of geotechnical engineering generally.

For many applications in civil engineering, the product and eventual interpretation of data arising from the results of insitu and laboratory testing based on the product of light cable percussion boreholes is adequate. However, the boring method does not produce a continuous record of ground conditions, rather, it provides 'snap-shots' of the strata encountered at discrete sampling or testing locations. So, while macro structures (larger than say 500mm) are revealed and their properties recorded, it is not always possible to define smaller scale structures, for instance thin bands of cohesive material within otherwise, free draining granular strata. From inspection of some important geological sections, described in the case histories (see for instance Wolston Pit SSSI), it is considered that these small scale structures can be of great significance with respect to their conservation and exposure.

7.4.5 Rotary drilling

The other common method of borehole production is by rotary drilling, in which a drill bit is rotated at the bottom of a borehole. This method is most commonly used for sampling rock formations, but with care and the selection of an appropriate drilling method, can be used to sample soils. A drilling fluid, usually air or water, is used to cool and lubricate the drill bit, and to remove drill cuttings. The drilling fluid is commonly circulated through hollow drill rods to the bit then returns to the ground surface between the rods and the borehole sides.

There are two basic types of rotary drilling as described in Table 7.5.

Table 7.5 Types of rotary drilling	
Open Hole Drilling	The drill bit cuts the full face within the borehole diameter. Usually only used for general geological correlation or for looking for voids, such as solution holes or mine shafts. No usable samples recovered for geotechnical testing.
Core Drilling	An annular bit is fitted to the base of a rotating core barrel. The core is retained inside the core barrel, which can be returned to the ground surface and recovered. May be used for site investigation. Samples may be usable for geotechnical testing, depending on the state of disturbance.

Rotary drilling for site investigation is usually carried out using core drilling methods. The objective of core drilling is to achieve optimum core recovery and core quality. Unfortunately, the Quaternary deposits that are the subject of this study are the most challenging in this respect as there is a tendency for the unconsolidated materials to get washed away with drill fluids. However, with care an experienced driller using a large diameter core barrel and probably by using a polymer drilling fluid in place of air or water, may be able to achieve good sample recovery in these deposits. However, such rotary core drilling may be several times more expensive than cable percussion boring. Drilling for site investigation is normally carried out using a triple tube core barrel, which has a removable inner liner. The recovered ground can be retained inside this liner for transport before inspection.

Where successful, the production of a section of continuous core for detailed geotechnical inspection and sub-sampling for later laboratory testing overcomes the deficiencies inherent in the use of cable percussion boring methods, and provides superior information for geotechnical design.

7.4.6 Difficult access boring/drilling rigs

Where there is a problem of access to ideal exploration locations, it may be necessary to resort to smaller, more portable, drilling equipment. Examples of such equipment are described below.

Archway competitor rig (see Figure 7.6), this is a self contained percussion boring rig which can be either trailer or track-mounted. Even when fully equipped, this rig can be manoeuvred through gaps no wider than a normal domestic doorway. The competitor rig can recover continuous samples within a rigid plastic liner of diameters of up to 100 mm. However, because of the percussion method, sample disturbance is often great. It is also possible to carry out standard penetration tests. However, the rig does have several disadvantages. The boring method does not use casing, and therefore, it is often not possible to maintain borehole side stability in granular soils, particularly below the water table. In order to achieve hole depths of more than a few metres, it is necessary to successively reduce the diameter of the sampling tool. This clearly results in poorer sample quality; in most cases, it is not possible to progress this type of borehole more than about 6 metres. If obstructions, such as cobbles, are encountered, then the hole normally has to be terminated.

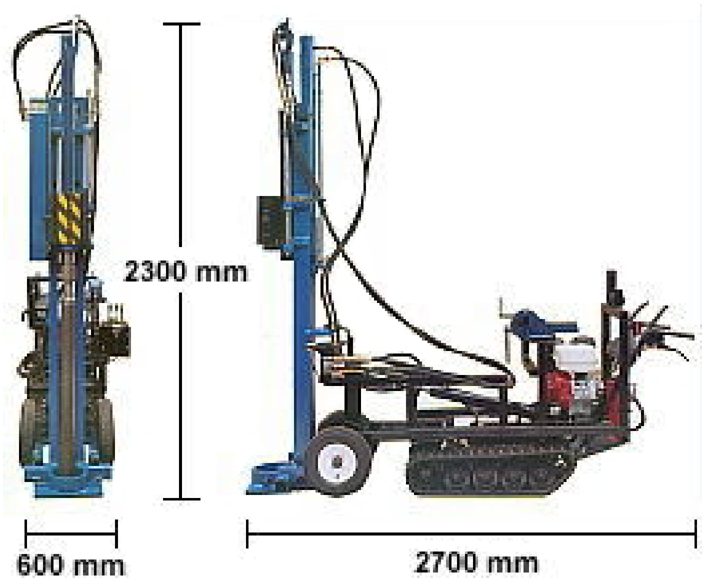


Figure 7.6: Archway Competitor Boring Rig (after Archway Engineering Ltd)

The pioneer rig was developed to overcome the disadvantages of the Competitor rig, while being of comparable size. It also recovers soil samples by dynamic percussion (or carry out SPTs), but can introduce casing to enable sampling to continue in wet granular soils. In addition, the rig is fitted with a rotary drilling head, so that obstructions can be removed but also allow core drilling methods to be used. Drilling depths of more than 50 metres can be achieved with the Pioneer rig.

Custom made cable percussion rigs are available that can be broken down into portable sections (the largest is usually the diesel engine). These can be carried, if necessary, and erected at any exploration point. However, sufficient operating room is still required.

7.4.7 Probing

Two types of probing are in common use in site investigation works; these are the 'dynamic' and 'static' cone penetration methods.

Dynamic probing

The apparatus for dynamic probing comprises a sectional rod with a cone fitted to its base. The cone is of a slightly greater diameter than the rods. The assembly is driven into the ground using a constant mass that applies a constant force, by dropping through a standard distance on to an anvil fixed to the top of the rods.

The number of blows to drive the cone 100 mm is recorded. In this way the test is similar to the standard penetration test (SPT) described earlier. However, its development, which has always been aimed at being portable and quick to set up and operate and yet powerful enough to penetrate dense strata and obstructions, has resulted in non-standardisation of equipment. There are therefore several types of dynamic probe available. This has hindered the development of accepted relationships between the driving resistance of the dynamic probe with geotechnical soil properties.

The main use of dynamic probes has therefore become one of providing the means of determining the thickness and distribution of weak (or strong) strata at a large number of locations, relatively cheaply. However, the results of dynamic probing do not enable the identification of the composition of the soils under test, and it is not possible (usually) for samples to be recovered.

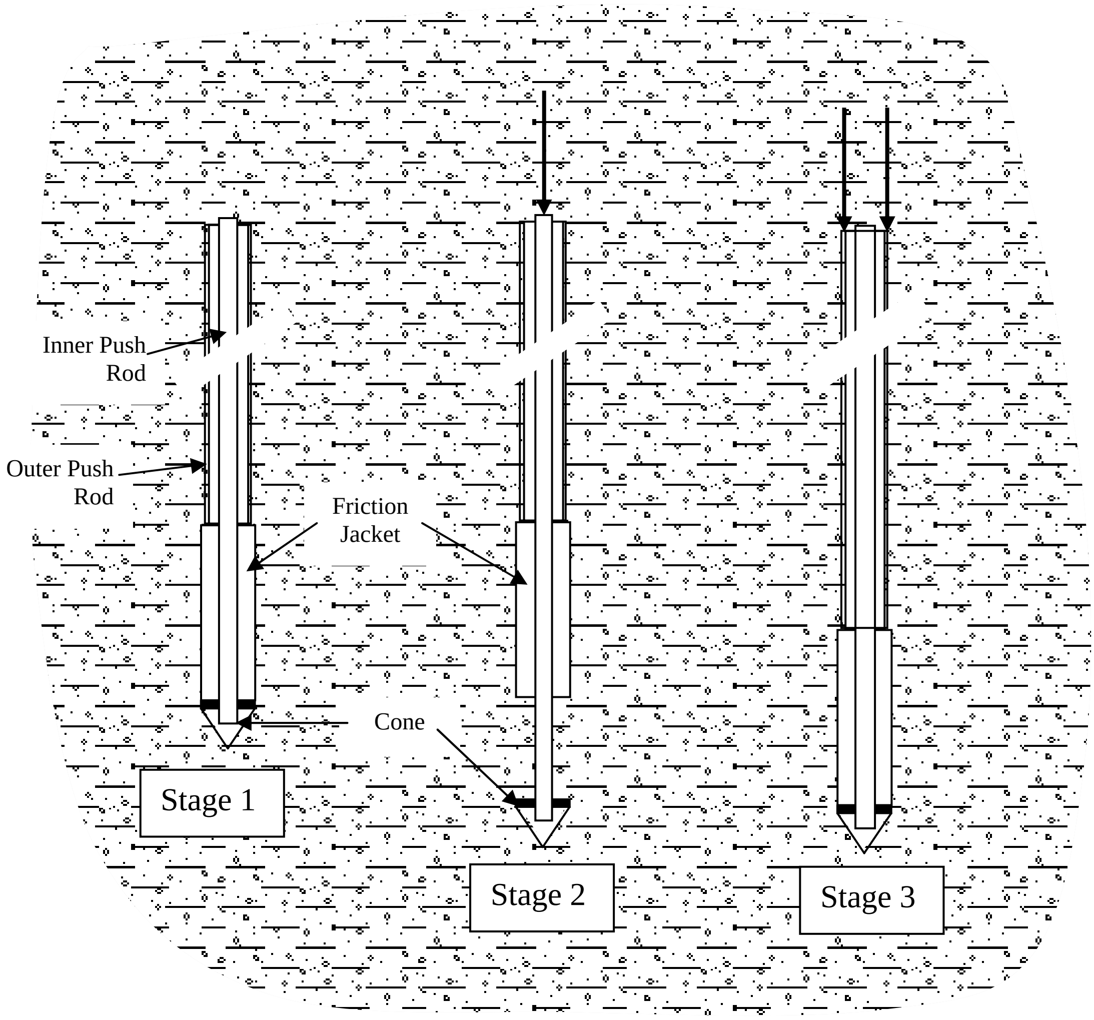
It is not believed that the use of dynamic probing would be of great value to the design of geological conservation sections.

Static probing

Static probing, which was developed initially for the weak soils present in the Low Countries of continental Europe, relies like dynamic probing, on the penetration of soils with a solid steel cone of slightly larger diameter than the 'push rods'. However, instead of using dynamic energy from a drop weight to drive the cone, this is advanced by static push. This generally requires equipment of high mass, (trucks loaded with concrete kentledge) to achieve high penetration pressures through dense soils, although the original concept used portable equipment, sometimes secured with temporary soil anchors. The principle is shown in Figure 7.7. However, several sophistications have been added to the basic concept. The head of the cone is fitted with a sensor that electronically records driving resistance. This is possible because the driving force and rate (about 20 mms^{-1}) are held constant during the test. Behind the cone, the driving assembly is fitted with a 'friction jacket'. The test procedure enables the front cone and friction jacket to be advanced independently. Relationships have been developed between measures of cone resistance, and sleeve friction and laboratory derived geotechnical parameters. However, most importantly, it has also been found that the ratio between side friction and cone resistance is dependent on the mineralogy of the soil being penetrated. This allows soil types to be determined without inspection.

A third attachment to the driving assembly that has now become almost standard is an electronic piezometer (water pressure transducer). This system is generally known as a piezocone. The measurements of all of the above parameters are generally recorded electronically, and are therefore conducive to continuous data recording and subsequent analysis.

This system now becomes a very powerful geotechnical tool, capable of recording small scale (about 100 mm resolution) variations in soil type (granular or cohesive) and soil strength, as well as being able to measure groundwater pressures. Dissipation tests, which monitor the short-term reduction in water pressures with time, allow estimations of soil permeability. Above the groundwater table, the negative porewater pressures recorded by piezocones are related to ‘soil suction’.



Stage 1: Rest position, cone and friction jacket together.

Stage 2: Force on central push-rod advances the cone.
Force divided by cone area, equals soil cone resistance.

Stage 3: Force on outer push-rod.
Force divided by friction jacket area, equals soil frictional resistance.

Figure 7.7: Principle of Static Cone Test Method

Typical output from static cone (piezocone) tests are shown in Figure 7.8 (after Bell, 1987).

The system has been in steady development over recent years. This has included the development of tools to enable soil samples to be recovered for inspection, and classification testing, along with others that can be used to establish load – deformation characteristics (cone pressuremeter).

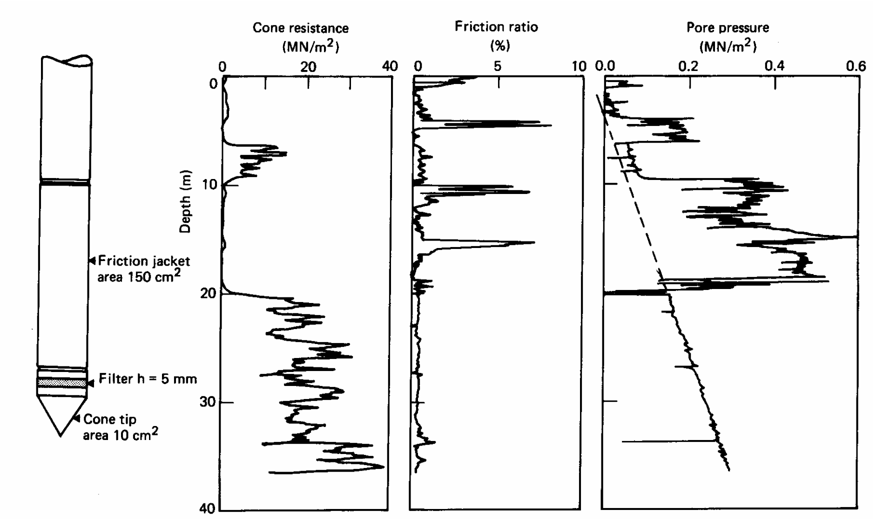


Figure 7.8: Typical Output from Piezocone (after Bell 1987)

It is considered that the static cone site exploration tool has many features that may be of great assistance to the design of stable geological sections. In particular, it is believed that the close resolution of insitu properties has advantages over the more traditional reliance on the results of SPT test, available from the more traditional cable percussion investigation techniques.

7.4.8 Summary

A summary of frequently encountered intrusive site investigation methods is included in Table 7.7, along with some of their advantages and disadvantages.

Table 7.7: Frequently encountered intrusive site investigation methods	
Investigation Method	**Comments**
Trial Pitting	**Advantages** ∉ relatively inexpensive; ∉ provides opportunity to inspect the ground at close quarters and identify lateral variations. **Disadvantages** ∉ very disruptive of ground under investigation; ∉ limited to a maximum depth of about 5.0 metres.
Boreholes Cable Percussion	**Advantages** ∉ relatively inexpensive; ∉ the most common type of site investigation boring; ∉ less disruptive than trial pits; ∉ enables most common forms of insitu testing to be carried out; ∉ provides samples for most types of geotechnical laboratory testing; **Disadvantages** ∉ does not provide a continuous record of strata encountered.
Rotary Drilling (Open Hole)	**Disadvantages** ∉ not usually used for geotechnical site investigation, except for geological correlation and void searching; ∉ does not provide samples suitable for geotechnical testing.
Rotary Drilling (core)	**Advantages** ∉ when carried out successfully, by experienced operators, core drilling can provide high quality, continuous samples; ∉ depending on the natural state of ground disturbance, samples can be used for most types of geotechnical laboratory testing. **Disadvantages** ∉ relatively expensive ∉ unconsolidated deposits are the most difficult to core satisfactorily. Use must often be made of large core diameters and polymer drilling fluids.
Difficult Access Rigs	∉ used where access is difficult or impossible for conventional trailer or truck mounted boring/drilling rigs. Types include: ∉ Archway Competitor (percussion); ∉ Pioneer (percussion/rotary); ∉ Cut-down cable percussion.
Probe Testing Dynamic Probe Testing	**Disadvantages** ∉ not used for geotechnical site investigation, except to provide general comparative information over a wide area;
Static Probe Testing	∉ provides high quality information on ground conditions, including a continuous soil strength profile along with groundwater pressures; ∉ identifies small changes in vertical sequence, such as sandy partings in cohesive deposits; ∉ relatively inexpensive. **Disadvantages** ∉ may require at least one cable percussion borehole to provide sufficient sample for geotechnical testing.

7.5 Laboratory testing

7.5.1 Background

Laboratory testing of samples of soils recovered during the intrusive phase of site investigation is required to classify and characterise the materials encountered and to provide geotechnical parameters for engineering design.

Laboratory testing of soils in the UK is covered by a specific British Standard BS 1377:1991 “Methods of Tests of Soils for Engineering Purposes”. While this guidance sets out the procedures to be adopted by laboratory managers and technicians, a much more thorough description of the specification and interpretation of these tests is included in three volumes prepared by Head (1986).

While designing a laboratory testing schedule, consideration needs to be given to the purpose of the investigation. In particular, attention should be given to ensure that sufficient information is provided to allow the design of a range of conservation options, in order that these can be evaluated on an equal footing.

7.5.2 Classification testing

Classification tests can be carried out on both disturbed and undisturbed soil samples, and, for the present purposes should be used to categorise each of the strata present within (as well as above and below) a particular section.

In the case of a granular soil, the property most important in term of design is its particle size distribution. The results of grading tests are usually displayed as particle size distribution curves, examples of which are shown as Figure 7.9. The curves are produced by plotting the percentage, by weight, of materials passing standard sieve sizes down to 65σm, and by sedimentation for finer fractions.

As discussed in Section 5, the particle size distribution can be of assistance in determining the drainage characteristics of soils. Figure 5.2 shows the relationship between the dominant soil type and approximate permeability range.

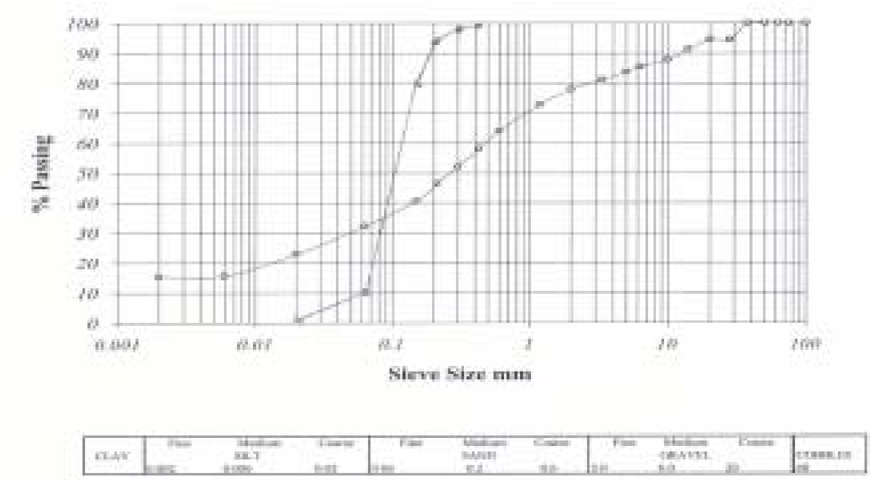


Figure 7.9: Examples of Soil Grading Curves

In the case of fine grained (cohesive) soils, the most common classification tests are those used to determine their insitu moisture content and plasticity. The suite of plasticity tests are usually known as the Atterberg Limits, and determine the proportion of water, by weight, that a soil can absorb and yet still behave as a solid material (known as its plastic limit); and the proportion of water that it can absorb before it acts as a liquid (known as its liquid limit). The difference between these two values, during which it acts as a plastic material, ie: neither a solid or a liquid, is termed its ‘plasticity index’, and is a measure of both the proportion and type of clay mineral within a soil aggregate. From a knowledge of the natural moisture content and plasticity limits, insitu soil behaviour can be predicted in a general way.

Figure 7.10 Illustrates the classification of fine grained soils based on the results of plasticity testing.

Where sufficient samples are available, a profile of moisture content vs depth can be helpful in indicating zones of softening or groundwater seepage.

Slope stability calculations require a knowledge of soil bulk density. This can be determined from direct measurement of undisturbed samples, if available.

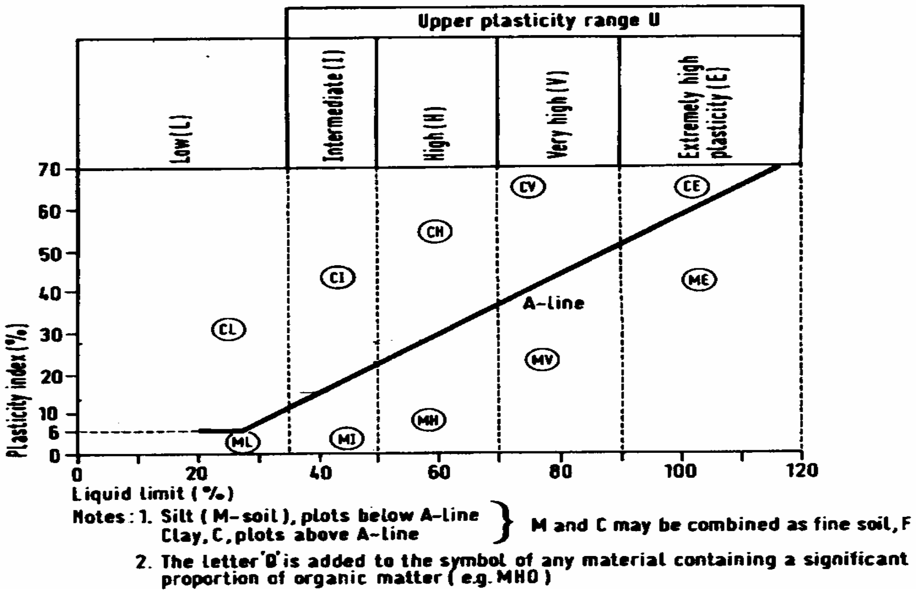


Figure 7.10: Plasticity Classification Chart (BS 5930:1999)

7.5.3 Shear strength testing

In the case of the construction of specific geological sections, the type of testing should be chosen to provide information which can be used for both short term and long term slope stability analyses (see Section 5.4).

Summary of undrained/drained stability	
Undrained Stability:	∉ evaluates the short-term stability of cuttings and excavations; ∉ uses undrained (total stress) shear strength parameters of cohesive soils (λ = 0); ∉ does not consider effects of groundwater fluctuations.
Drained Stability	∉ evaluates the long-term stability of cuttings and excavations ∉ uses drained (effective stress) shear strength parameters of cohesive and granular soils; ∉ groundwater conditions must also be modelled.

Short term analyses are carried out using the results of 'quick' undrained triaxial tests. Such tests can be carried out quickly, as their name implies and are relatively cheap. Several tests can therefore be carried out. It is often useful to be able to correlate laboratory strength data with those determined from insitu testing. The results of quick undrained triaxial testing are usually expressed as 'undrained shear strength' (Cu), measured in kNm^{-2}.

Long-term stability analyses have to be carried out using consolidated and drained tests. These take much longer to complete as the test first requires the sample to consolidate under a selected confining pressure, and must then be stressed slowly to allow dissipation of excess pore water pressures during the test. As a consequence, these are expensive and normally kept to a minimum. The results of consolidated drained tests are usually expressed as 'effective cohesion (c'), measured in kNm^{-2} and effective friction angle (λ'), measured in degrees. Both types of shear strength test will also provide information on the bulk density and natural moisture content of the soils.

It is not possible to obtain undisturbed samples of granular soils. Strength properties of these are therefore determined using recompacted samples inside a shear box. The friction angle of the soil is then determined by direct shear. The tests are normally carried out under saturated conditions, but because granular soils are generally free draining, the results always give the drained strength properties (effective friction angle λ'). In the case of granular soils, both short and long term stability analyses are carried out using drained parameters.

7.5.4 Summary

Numerous other types of laboratory tests can be carried out, but those described above are the most common where information is required for slope stability design. Where other soil properties are referred to elsewhere in the report, then reference is made to the appropriate test method.

A summary of the types of tests described in this section is shown in Table 7.8.

Table 7.8: Summary of common test methods for slope stability design	
Type of Test	Comments
Classification	Cohesive soils: ∉ moisture content; ∉ plasticity indices; ∉ particle size distribution; ∉ bulk density Granular Soils: ∉ particle size distribution; ∉ bulk density.
Shear strength	Cohesive soils ∉ quick undrained triaxial strength (short-term stability); ∉ consolidated undrained triaxial testing (long term stability analysis). ∉ shear box test to determine residual shear strength properties. Granular Soils ∉ shear box tests (both short and long-term analysis)

7.6 Derived soil design data

There will always be circumstances where, because of topography or other access restrictions, it is not possible to perform the scope of site investigation required to produce the quality of design data required for rational slope stability analysis. In these cases, it may be necessary to derive geotechnical design data by empirical means.

Several means are available to derive design parameters, including visual inspection (although these may require at least some basic test data) or published data.

British Standard BS8002:1994 "Code of Practice for Earth Retaining Structures", includes tables that enable the estimation of basic soil properties, that may be regarded as conservative enough for use in the absence of sophisticated laboratory test data. These tables are included and described below.

Values for the unit weight of soils are required for slope stability analyses, as these generate the principal driving forces controlling stability. These may be measured insitu or in the case of cohesive soils, by laboratory testing. However, the values included in Table 7.9 may be used with care in the absence of such test data, provided the limitations of using indicative parameters are appreciated.

Table 7.9: Unit Weights of Soils and Similar Materials (BS8002:1994)				
Material	**Moist Unit Wight** $_{m}$ **kN/m^3**		**Saturated Unit Wight** $_{s}$ **kN/m^3**	
	Loose	**Dense**	**Loose**	**Dense**
A – Granular Soils				
Gravel	16.0	18.0	20.0	21.0
Well graded sand and gravel	19.0	21.0	21.5	23.0
Medium or coarse sand	16.5	18.5	20.0	21.5
Well graded sand	18.0	21.0	20.5	22.5
Fine or silty sand	17.0	19.0	20.0	21.5
Rock fill	15.0	17.5	19.5	21.0
B – Cohesive Soils				
Peat (very variable)	12.0		12.0	
Organic clay	15.0		15.0	
Soft clay	17.0		17.0	
Firm clay	18.0		18.0	
Stiff clay	19.0		19.0	
Hard clay	20.0		20.0	
Stiff or hard glacial till	21.0		21.0	

Back analyses of first time slope failures in cohesive soils (ie: not along pre-existing shear planes) have found drained shear strengths no lower than a critical drained (effective) angle of friction λ'_{crit}. In the absence of reliable test data, the conservative values of λ'_{crit} listed in Table 7.9 may be used, with effective cohesion c' equal to zero, provided the limitations of using generalised parameters are appreciated.

Table 7.10: Critical effective friction angle λ'_{crit} for clay soils	
Plasticity Index %	**λ'_{crit} Degrees**
15	30
30	25
50	20
80	15

The shear strength of granular soils can only be determined directly by laboratory methods using a shear box. However, where the results of shear box tests are not available, or where the soil particle sizes are too coarse to fit inside conventional shear boxes, then it has been shown that the peak effective angle of shearing resistance λ'_{peak} of a granular soil in degrees can be estimated by:

$$\lambda'_{peak} = 30 + A + B + C$$

Where A, B and C are variables depending on particle angularity, particle size distribution and relative density, respectively. Typical values for A, B and C are given in Table 7.11.

Table 7.11: Peak Effective Friction Angle λ'$_{peak}$ for Granular Soils (BS8002,1994)	
A – Angularity[1]	**A (Degrees)**
Rounded	0
Sub-angular	2
Angular	4
B – Particle Size Distribution[2]	**B (Degrees)**
Uniform (U < 2)	0
Moderate Grading (2 < U < 6)	2
Well Graded (U > 6)	4
C – Relative Density (SPT 'N' Value[3])	**C (Degrees)**
Loose (N < 10)	0
Medium Dense (10 < N < 30)	2
Dense (30 < N < 50)	6
Very Dense (N > 50)	9

1. Angularity is estimated from visual inspection of the soil
2. Grading can be determined from particle size distribution curves by use of the Uniformity Coefficient U = D_{60}/D_{10}
   Where D_{10} and D_{60} are particle sizes such that in the sample, 10% of the material is finer than D_{10} and ^6% is finer than D_{60}.
   A step-graded soil should be treated as uniform or moderately graded soil according to the grading of the finer fraction.
3. The results of standard penetration tests ('N' value)

Intermediate values of A, B and C by interpolation.

Alternatively, use can be made of published data relating to geotechnical properties obtained during the investigation of nearby sites in similar strata. These are often the property of site developers or owners, who may or may not be amenable to the dissemination of their data.

However, other publicly financed projects have yielded data that are available for use in preliminary geotechnical design. Much of this information is held by the British Geological Survey (BGS). These data are periodically collated and published. Of particular interest in regard to the Terrace Gravel deposits, which have been examined during some of the case studies that accompany this report, is the publication of a summary of the geotechnical properties determined during site investigations associated with the construction of the M25 motorway (Technical Report WN/90/2 'SW Essex – M25 Corridor: Engineering Geology', 1991). These were produced as part of the Engineering Geology Series. The report includes useful information including: particle size distribution, bulk density and shear strength, of many geological deposits such as Cretaceous Chalk, Tertiary Woolwich and Reading Beds, London Clay and Thanet Sands, as well as Quaternary glacio-fluvial deposits, Terrace deposits and head. Average geotechnical properties for a number of the deposits occurring in the M25 corridor in SW Essex are included in Table 7.12.

Table 7.12: Average geotechnical properties for unconsolidated deposits in SW Essex (BGS, 1991)					
Geotechnical Parameter	**Thanet Sand**	**Woolwich and Reading Beds**	**Glacio-fluvial sand and gravel**	**Terrace Gravel**	**Head**
Moisture content %	29	23	15	21	26
Liquid Limit %	53	28			
Plastic Limit %	21	20			
Plasticity Index %	30	13			
Bulk Density Mg/m^3	1.90	1.94	2.10		1.92
Dry Density Mg/m^3	1.50	1.58	1.88		
SPT 'N' value	58	35	29	30	24
Permeability m/s	27.6 x 10-6				
Clay Content %	5	5	0	0	0
Silt Content %	13	12	0	0	0
Sand Content %	80	74	69	26	72
Gravel Content %	0	7	5	71	10
Undrained Shear Strength Cu kN/m^2	74	61			

Reports are available from the BGS for other areas in England and Wales including:

BGS Engineering Geology Reports	
∉ Stoke-on-Trent	Report Ref WN/90/11
∉ Birmingham West (Black Country)	Report Ref WN/91/15
∉ South Essex	Report Ref WN/EG/75/20
∉ West Wiltshire and SE Avon	Report Ref WA/VG/85/8
∉ Southampton	Report Ref WO/87/2 & WO/87/4
∉ Wrexham	Report Ref WN/90/10

Data collections are also available from learned papers and text books. These include Stroud and Butler (1975), which relates the results of standard penetration tests to undrained shear strength in cohesive soils. Others include: Sladen and Wrigley (1983); Bell (1994); Northmore and others (1996) and Terzaghi (1955).

It should be recognised that the use of data supplied by a third party or taken from databases may be 'without ownership'. This is important where this data is relied upon in geotechnical design.

7.7 Key points

The summary of site investigation practices has indicated the following with particular reference to the methods that might be of most benefit when designing stable geological sections:

∉ desk studies and visual reconnaissance of sites for proposed geological sections are very important, and form the basis for the subsequent intrusive investigation and design;

∉ in order to achieve the most reliable design data it is necessary to carry out the most appropriate types of intrusive investigation. The relative merits have been described, and recommendations made with respect to the use of the commonly used method of static cone testing;

∉ emphasis has been placed on the importance of monitoring groundwater levels and the preference for obtaining them either from monitoring standpipes, piezometers or by using the piezocone attachment to static cone testing;

∉ typical laboratory testing options have been described. While classification tests do not generate design data directly, they are relatively cheap to carry out and may help to produce ‘derived’ design data. By contrast, the long term drained tests required for effective stress analysis are expensive, but often the only viable alternative.

∉ options are provided whereby geotechnical design data can be estimated on the basis of visual inspection along with limited laboratory testing or from published sources.

8. Generic stabilisation methods

8.1 Introduction

Where a geological section cannot be maintained in a stable condition at an appropriate slope angle, then it may be necessary to apply artificial measures to prevent instability. Common measures that can be applied can be divided into the four generic types listed in Table 8.1.

Table 8.1: Generic Types Of Soil Stabilisation Methods
∉ physical support (buttress); ∉ soil reinforcement; ∉ artificial slope drainage; ∉ slope protection (re-vegetation); ∉ reprofiling.

Most of these methods, with the possible exception of artificial slope drainage, will to a greater of lesser extent, conceal at least part of the geological section that is to be conserved. However, it is believed that when applied with sensitivity, it may be possible to achieve an acceptable balance between 'preserved' sections, ie: those that are concealed but still available for future study, while still providing representative exposures or partial exposures that can be uncovered readily as and when required.

The principles behind each of the generic methods of slope stabilisation, listed in Table 8.1 along with examples, are described in the remainder of this section. Detailed design procedures are not included, however, basic principles and their range of application are discussed along with reference to design guides. Applications are noted where the partial use of stabilisation measures could lead to a sensible compromise between conservation and stabilisation requirements.

These stabilisation methods are examples of physical actions that may be taken to improve slope stability. However, an alternative might be the removal or limitation of the consequences of failure using a risk analysis bases assessment.

8.2 Physical support (buttress)

The principle behind the physical support of a slope is to provide sufficient mass at its toe to prevent ground movement – see Figure 8.1.

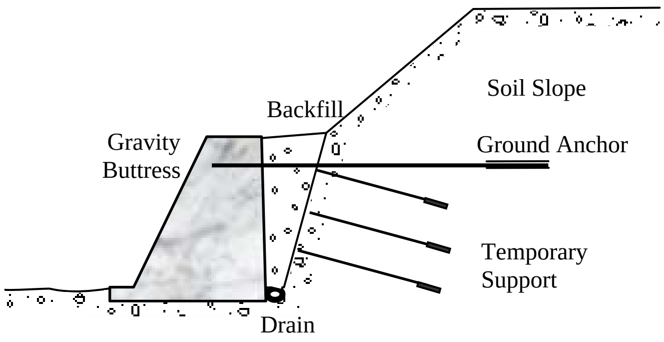


Figure 8.1: Typical Gravity Type Buttress

The engineering design principles behind gravity buttresses can be summarised in Table 8.2.

Table 8.2: Principles behind the design of gravity retaining structures
∉ the buttress should not overturn about its outside toe; ∉ it should resist sliding along its base; ∉ it should not overstress the ground beneath the buttress or induce excessive amounts of settlement.

Figure 8.1 shows several important constructional features about the gravity buttress. In practice it is often not possible to construct the buttress hard against the natural ground, and nor is this always desirable. It is normal practice to leave a suitable gap behind the buttress which is filled with gruanular fill. This should be free draining (uniformly graded) stone, which needs a minimum of compaction. A drain is often set at the base of the fill to prevent the build up of water behind the buttress. Where stability analyses indicate that there is a risk of either overturning or basal sliding, then the stability can be improved by the incorporation of ground anchors fixed between the buttress and the natural ground.

In order to provide stability during construction, it may be necessary to provide temporary support to retained ground in order to ensure the safety of construction workers. It is a requirement of designers, under CDM regulations (see Section 4.2), to ensure that construction is able to proceed in a safe manner.

In general terms, the method of construction and the types of materials used mean that these should be viewed as permanant structures, which will effectively conceal geological exposures for the long-term. Sediments preserved behind such structures will be available for future generations to decide when re-exposure is appropriate, but there are risks and costs associated with this re-exposure. In the short to medium term, physical and visual access is lost.

Gravity walls can be of a variety of types including mass concrete, stone (gabions) or timber (crib).

Several varieties of concrete retaining wall are in common usage. These structures include the gravity wall shown in Figure 8.1, but also include reinforced concrete cantilever walls with counterforts or external buttresses as shown in Figure 8.2.

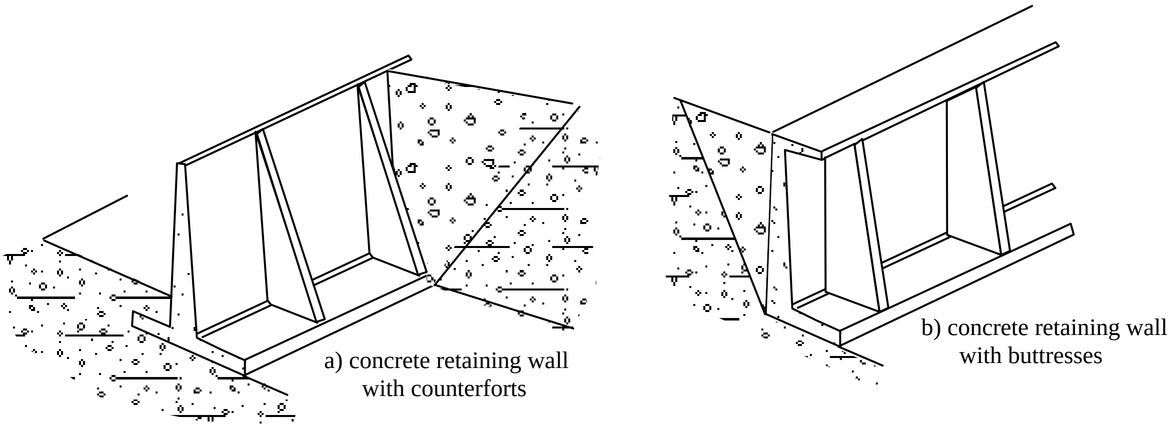


Figure 8.2: Forms of Reinforced Concrete Cantilever Retaining wall

Alternatives to concrete retaining structures exist which still provide a ‘gravity’ based containment. Two common example of these are stone gabions and timber crib walls.

8.2.1 Gabions

Gabions comprise cages or baskets, usually of steel wire or weld mesh, cuboid in shape and filled with stone. They are commonly used for the construction of retaining walls, revetments or anti-erosion works.

The permeability and flexibility of gabions make them suitable where the retained material is likely to be saturated or where the bearing quality of the sub-soil is poor. As such, if used with sensitivity, gabions are particularly appropriate for low cost effective ground support in the vicinity of geological conservation sites. An example of their possble use is illustrated in Figure 8.3.

In this example, it is envisaged that three (or more or less) overlaping conservation sections are established within a slope that would otherwise be inappropriate for full height conservation. While parts of the section may be hidden, for the medium to long-term, some conservation and management objectives can be met.

Box gabions are available in modules from a basic 1 x 1 x 1 metre cuboid, to blocks up to 6 x 2 x 1 metre deep. A starter layer can be created using a shallow mat either 300 or 500 mm thick, to ‘smooth’ irregularities in the ground surface and known as a ‘Reno Mattress’.

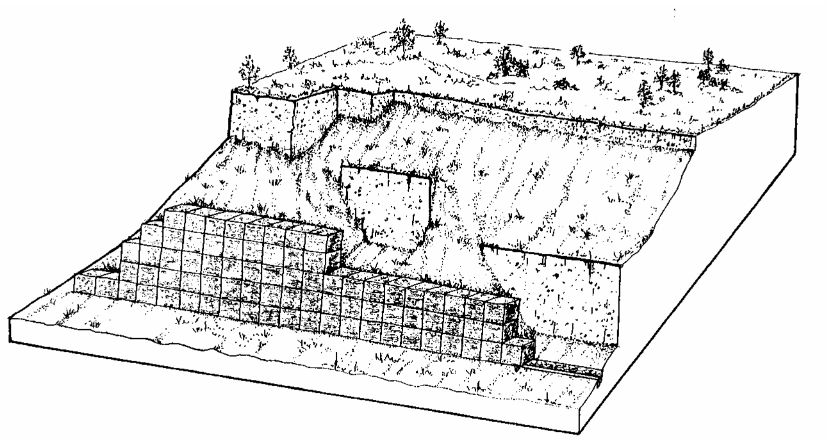


Figure 8.3: Sketch of Proposed Conservation Sections Supported by Gabion Baskets

Specifications for mesh diameter, corrosion protection, stone sizes and general construction are included in British Standard BS8002:1994 ‘Code of Practice for Earth Retaining Structures’.

8.2.2 Crib walls

Crib walls are another alternative to concrete gravity walls. They are built of individual units assembled to create a series of box-like structures containing suitable free draining granular fill to form a retaining wall structure.

Crib walls are usually built to a batter which should not be steeper than 1 horizontal to 4 vertical. An example of a timber crib wall is shown in Figure 8.4.

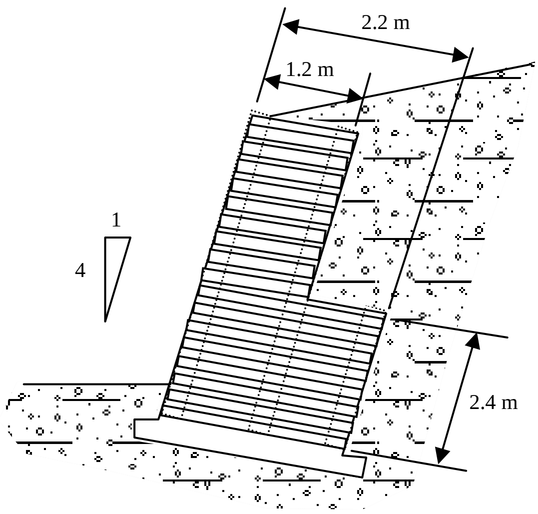


Figure 8.4: Timber Crib Wall – With Typical Dimensions

8.3 Soil reinforcement

The following section addresses the reinforcement of existing ground (which may be natural or made ground) using inclined soil nails. The method is usually used in granular soils and relies on grout penetration into the ground around the steel ‘nail’ to achieve the frictional resistance to pull-out forces.

Reinforcement spacings of between 0.75 and 3.00 metres are common, and often rely on the incorporation of a geotextile between nails to resist surface erosion. Depending on the nature of the retained ground, this can vary from wire mesh to 100% coverage using, for instance a coir matting. A typical arrangement is shown in Figure 8.5.

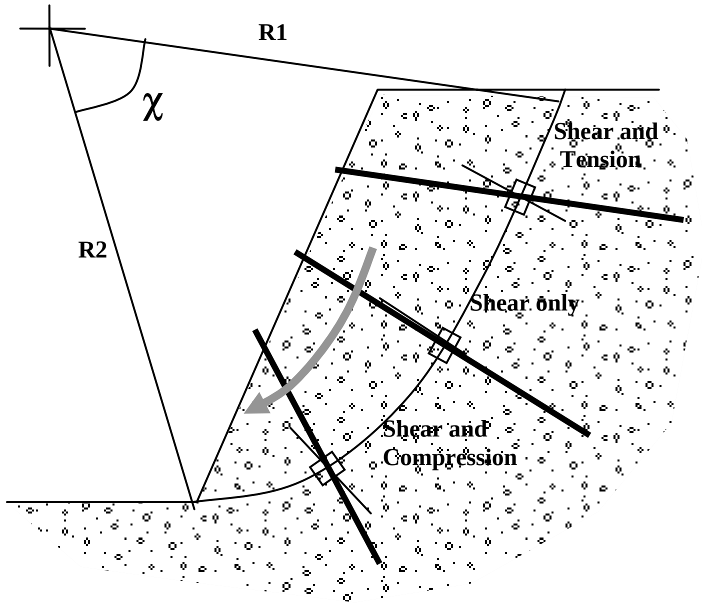


Figure 8.5: Typical Arrangement of Soil Nailing in a Natural Cutting (after Soil Nailing Ltd)

Soil nailing is appropriate for use as a means of reinforcing soil behind a proposed conservation section, in much the same way as the buttress support described above. However, it may also be used either within a conservation section or to support ground above a conservation section. The latter is possible because unlike the gravity structures, soil reinforcement does not require a bearing stratum (see Figure 8.6)

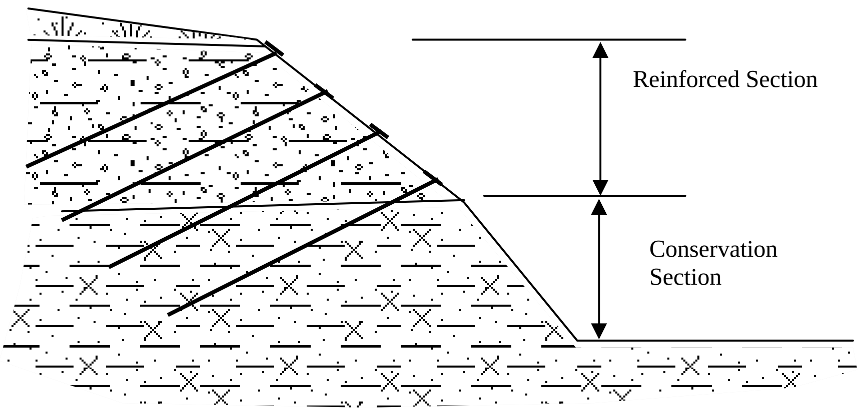


Figure 8.6: Soil Reinforcement Above a Geological Conservation Section

When used within a conservation exposure, some visual accessibility to the face would be lost, both behind the bearing plates of the soil nails and behind surface coverings. The impact of this is dependent upon the extent of soil nailing and associated works in relation to the location and extent of the interest. The most common facing materials used in the UK are geogrids. Typically, these are formed from high tensile polymers and may result in a lose of exposure of between about 15 and 40 % of the soil face, depending on grid dimensions. It is normal for geogrids to contain ‘carbon black’ (usually about 2%) in order to provide stability against ultra violet light. Theoretically, it might be possible to provide this protection by other means, and utilise a transparent geogrid that would provide greater visibility to the geological section.

It is frequently the case that after construction, reinforced slopes are hydro-seeded to restrict erosion from surface run-off, although, this is not essential if other forms of physical protection can be afforded. Once a geogrid has been placed, however, it is quite likely that localised vegetation will appear on the slope where moisture can be retained in pockets of loose soil retained by the grid.

The historical background to soil nailing is described by Bruce and Jewell (1985), while design methods are included in Highways Agency Report Ref HA 68/94 (1994).

Other common forms of soil reinforcement include ‘reinforced earth’ whereby an embankment of made ground is constructed within horizontal mats of geogrid. However, this has fewer applications to the conservation of geological sections.

8.4 Artificial slope drainage

Poor drainage is one of the major causes of slope instability. In Sections 5 and 6, it has been shown how seepage of groundwater from a face, or even high groundwater, within a body of a slope can have lead to degredation. The consequences of poor drainage can be clearly observed at the Wolston SSSI, where attempts have been made to conserve a geological section using the conservation void process (Glasser and Lewis, 1994) and is described in the case study accompanying this report.

Artificial drainage can be applied in a number of ways as shown in Table 8.3.

Table 8.3: Methods of artificial drainage
∉ cut-off drains behind a slope face; ∉ counterfort drains within a slope face; ∉ drainage relief boreholes.

8.4.1 Cut-off drainage

Figure 8.7 shows a typical application of the use of cut-off drainage behind a slope. In this case the drain is set within a trench excavated parallel to the slope crest. The cut-off drain is used when high groundwater levels are present leading to a risk of deep-seated rotational failure. High level seepage can be localised and seasonal, and difficult to identify and predict with confidence. A cut off drain may be specified even though there might be little indication of such seepages by the site investigations, because it is a sensible precautionary measure.

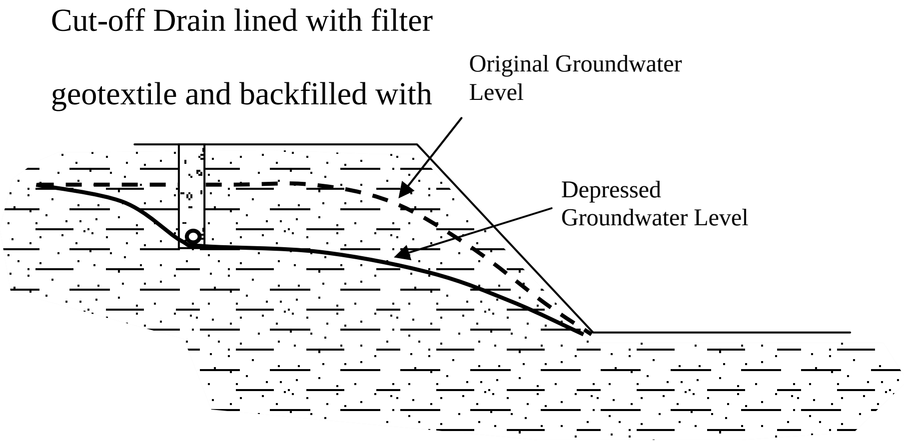
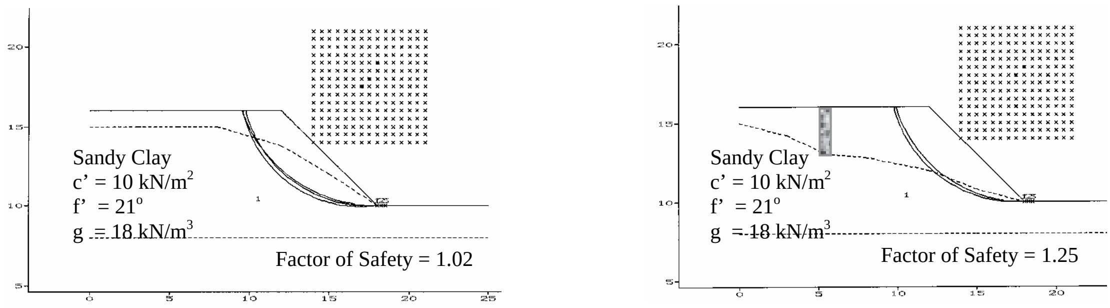


Figure 8.7: Application of cut-off drainage

Figure 8.7 shows the results of a drained stability analysis on the above section with and without drainage and using notational effective stress shear strength properties. The calculations show that the provision of drainage increases the factor of safety from 1.02 (Figure 8.8a) to 1.25 (Figure 8.8b).



(a) Without Drainage (b) With Drainage

Figure 8.8: Improvement in Factor of Safety as a Result of Soil Drainage

Cut-off trenches are usually backfilled with granular, single sized gravel, often inside a geotextile filter membrane. The single sized stone reduces long-term settlement of the drain backfill. In practice, the limiting depth of cut-off drainage is about 3.0 metres and is of greatest use in cohesive soils.

8.4.2 Counterfort drainage

Counterfort drainage is installed into the slope face, and therefore results in some loss of exposure as shown in the sketch Figure 8.9. Counterfort drainage can be especially useful for controlling groundwater seepages from the excavation face. This occurs where layers of varying vertical permeability results in sub-horizontal groundwater flow. In Section 6 of this report it has been shown that this type of occurrence can lead to shallow surface erosion and face degridation.

Counterfort drains are most effective if targetted at specific areas of seepage, that might become apparent after construction. In severe circumstances, thses can be constructed on a 'herring bone' pattern, in order to intercept more seepage points. These are frequently seen in highway cuttings throughout the UK. However, this clearly leads to further loss of exposure. Counterfort drains must be connected to adequate toe drainage in order to carry away collected water.

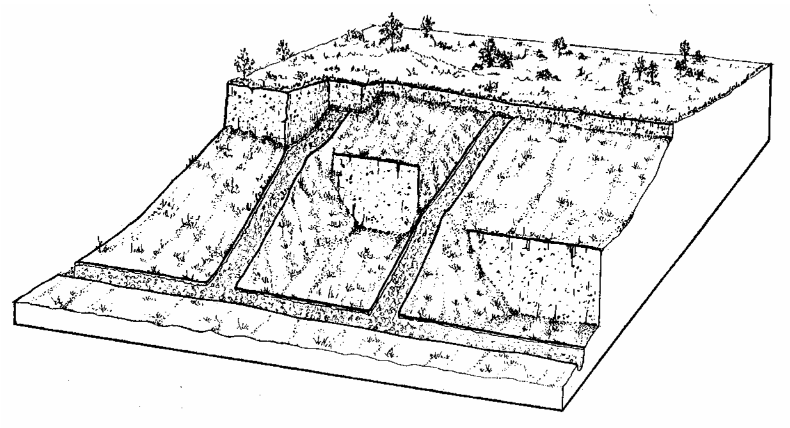


Figure 8.9: Schematic View of Counterfort Drainage

8.4.3 Drainage relief boreholes

Where a particular impermeable (cohesive) stratum inhibits downward migration and disippation of groundwater leading to an elevated, perched groundwater level, it may be possible to introduce inclined boreholes to generate artificial drainage (Figure 8.10). Care must be taken to ensure that drainage boreholes do not result in loss of significant slope material through internal erosion. They should therefore be lined with slotted plastic casing and possibly wrapped with a filter membrane.

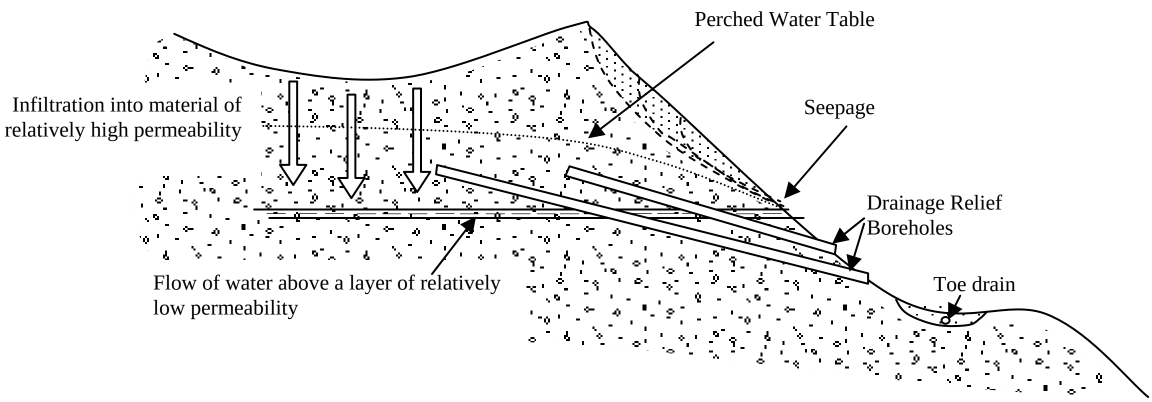


Figure 8.10: Drainage Relief Boreholes

In extreme circumstances, groundwater lowering can be undertaken using trenchless construction methods (including the use of drainage adits), or directional drilling.

8.5 Reprofiling

Perhaps the simplest of slope stabilisation methods is slope reprofiling, although this is not always as staight forward as it may seem. Clearly, if the face angle of a slope is slackened, then its stability against the types of rotational failure described in Sections 6.2 (rotational failure) and illustrated in Figure 8.8, should increase. This can be demonstrated by carrying out a series of stability calculations with constant soil properties, but as varying slope angle.

Figure 8.11 shows the variation between Factor of Safety against non-trivial failure (ie a failure that involves a significant proportion of the slope) and slope angle for the soil conditions shown in Figure 8.8b.

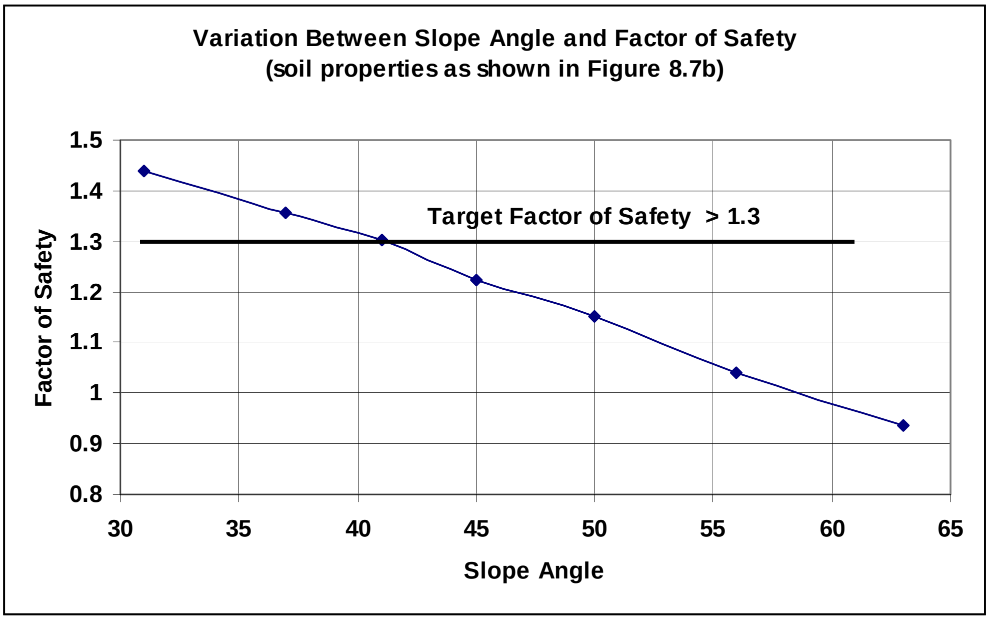


Figure8.11: Variation between slope angle and factor of safety

This graph shows a more or less linear relationship between Factor of Safety and slope angle, although it is anticipated that the graph would become non-linear at very steep or very flat slopes.

With reprofilling, complications can arise where the load on a potential slip surface is redistributed. If load is added to the disturbing force rather than the restoring force, then the slope will be less stable than reprofiling.

Furthermore, as the slope is slackened, the quantity of incident rainfall on the slope will also increase. This is likely to increase the rate of face erosion and the formation of erosion rills and gullies. In slopes that have relied on the effects of soil suctions and sementation to stand at steeper angles than would normall be predicted, reducing the slope angle could reslult in the saturation of the near surface zone, and subsequent instability.

In addition, flatter slope angles will encourage revegetation, which may or may not be desirable. Of course revegetation itself will increase stability against erosion and shallow depth surface sliding, but result in loss of visibility of the soil section. Notwithstanding this, of all of the hard engineering solutions, the removal of top soil and vegetation to allow periodic inspection, is probably the easiest.

As with many of the support/reinforcement options, a balance has to be achieved between stability (and safety) and the geological impact of concealment. It may be that compromises can be reached whereby sections are left exposed, but have fences at the toe to catch any eroding or unstable ground.

8.6 Key points

The section describes some of the commonly employed slope stabilisation strategies or treatments. These vary from 100% ‘hard’ cover, which should be treated as concealing any geological section for the ‘long-term’, to re-profiling and drainage control, which would entail only minimal concealment of a soil slope.

The merits of any of the proposed techniques must be assessed on the basis of the requirements of any particular geological conservation or management. For instance, the use of ‘hard’ structural restraint, if applied at selected areas, may enable the establishment of conservation exposures in a slope that would otherwise degrade naturally.

The effects of slope drainage and re-profiling on slope Factor of Safety have been demonstrated, and some of the drawbacks of re-profiling have been highlighted, notably increased erosion and likelihood of re-vegetation, but also the potential for reducing stability of used without regard for the consequent redistribution of forces and the potential for soil saturation.

9. Alternative stabilisation methods

9.1 Introduction

The procedures described in Section 8 “Generic Stabilisation Methods” represent methods of slope stability control that are amenable to engineering design. This concept is important. The requirement of CDM regulations necessitates that site construction works (in this case in the vicinity of soil slopes) are carried out in a safe manner. In the same way, when commissioned to carry out design works, for instance by a developer, geotechnical specialists have a duty of care to ensure that solutions are safe and appropriate to the proposed development. It would not be acceptable for instance to design unconsolidated slopes to a lower **Factor of Safety** than would normally be used, just because of pressures from English Nature or other interesting parties.

However, when considering the management of existing SSSIs located in ‘uncontroversial’ locations (ie where the consequences of failure are of low risk), it may be possible to look at some of the alternative stabilisation methods or analysis techniques. These could of themselves form research projects to measure the performance of novel stabilisation methods.

As described in Section 5.5, the concept of **soil suction** is particularly applicable to the stability of cuttings and excavations. It is widely believed that the soil moisture deficits present in excavations made in unsaturated strata, result in these sections standing at angles that are steeper than can be accounted for by traditional stability calculations using measured drained shear strength properties.

The application of **soil suction** to stability calculations is to apply a negative pore water pressure in a conventional effective stress analysis. Alternatively, the soil suction can be considered to generate an additional **‘apparent cohesion’**. The magnitude of the additional cohesion has been reported to be typically between 1 and 30 kN/m^2, but can be much larger.

One of the reasons why the **apparent cohesion** provided by soil suctions are generally not applied in conventional slope stability calculations is because sudden rises in soil moisture content can result in a loss of the suction effect, which may result in slope instability. Such a rise in moisture content can occur after prolonged heavy rainfall. Unfortunately, over recently decades, the incidence of extreme high intensity storms appears to have increased.

Therefore, in order to be able to rely on soil suction for the stability of conservation sections, provision would also have to be made to protect the slope face from extreme weather conditions, and to prevent build up of water behind the slope.

Several methods lend themselves to provide surface protection. Some of these are listed in Table 9.1.

Table 9.1
∉ face protection; ∉ enclosure; ∉ face coating; ∉ soil grouting.

9.2 Face protection

There are a number of anecdotal reports of fixing perspex sheeting against slope faces in order to deflect incident rainfall. In principle, it could be expected that this might successfully protect exposed soil slopes from heavy rainfall, while maintaining surface evaporation provided a suitable gap is maintained between the soil face and the perspex cover to allow free air passage see Figure 9.1.

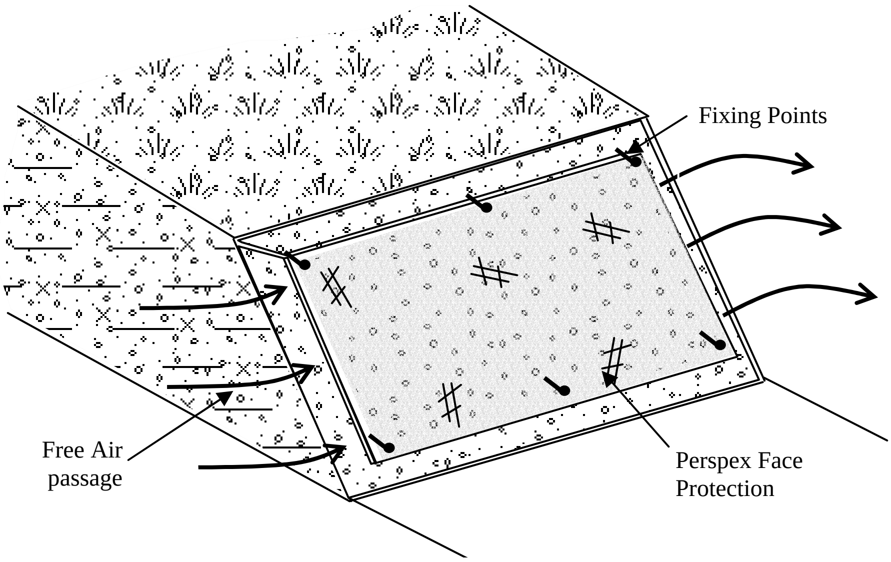


Figure 9.1: Face Protection Using Perspex Sheeting

However, practical difficulties are envisaged with this approach.

∉ it is likely that the build-up moisture on the rear face of the perspex sheet combined with incident sunlight could result in a rapid build up of algae and/or bacteria forming unsightly residues which would ultimately restrict visibility/visual access to geology;

∉ there is a likelihood that unavoidable slow soil erosion, possibly accentuated by increased air velocity behind the perspex sheet, would accumulate at the base of the section. This could result in the build-up of pore-water pressures when evaporation is suspected and consequent loss of the **soil suction** effect.

These problems can be overcome, but only by the implementation of a regular maintenance program.

Other forms of face protection may also be used. There is documentation (see Section 3.4) of an archaeological site in northern France, where an exposure has been conserved by the formation of a wooden frame and door over the most important section. It is understood that this has been in place for many years with some success. It is presumed that the problems noted above are overcome by periodically opening the door to allow collected minor erosion deposits to be removed. Access can be obtained by simply opening the door and therefore the build up of algae/bacterial deposits would not affect visibility.

However, this approach would only be feasible for relatively small exposures or localised features of interest.

9.3 Face coating

Advances within the chemicals industry have resulted in the generation of polymer based hydro-phobic compounds. It is conceivable that the application of appropriate materials to the face of a soil section could offer protection against incident rainfall by preventing the absorption of moisture during storm conditions. However, the success of such treatment would also depend on ensuring that under normal environmental circumstances, the evaporation of moisture from the slope face could continue. This might be achievable by leaving sufficient untreated areas to allow free outward passage of moisture. It is believed that this approach might be amenable for future research and field trials.

9.4 Grouting

As an extension to the concept described above, it is believed that consideration might be given to the application of low viscosity silicate base grouts to reinforce the surface layers of granular unconsolidated sediments. In this case its unlikely that the reinforcement principal of maintaining **soil suction** would be applicable. The use of these grouts would rely on filling all void space and providing an artificial soil cementation. Slope design would therefore benefit from increased apparent cohesion. However, the permeability of grouted strata would be significantly reduced and it is possible that ground water pressure would build up behind the treated area. Such an application would therefore, have to be supplemented with adequate drainage provision to prevent layer scale formation as a result of these ground water pressures (Figure 9.2).

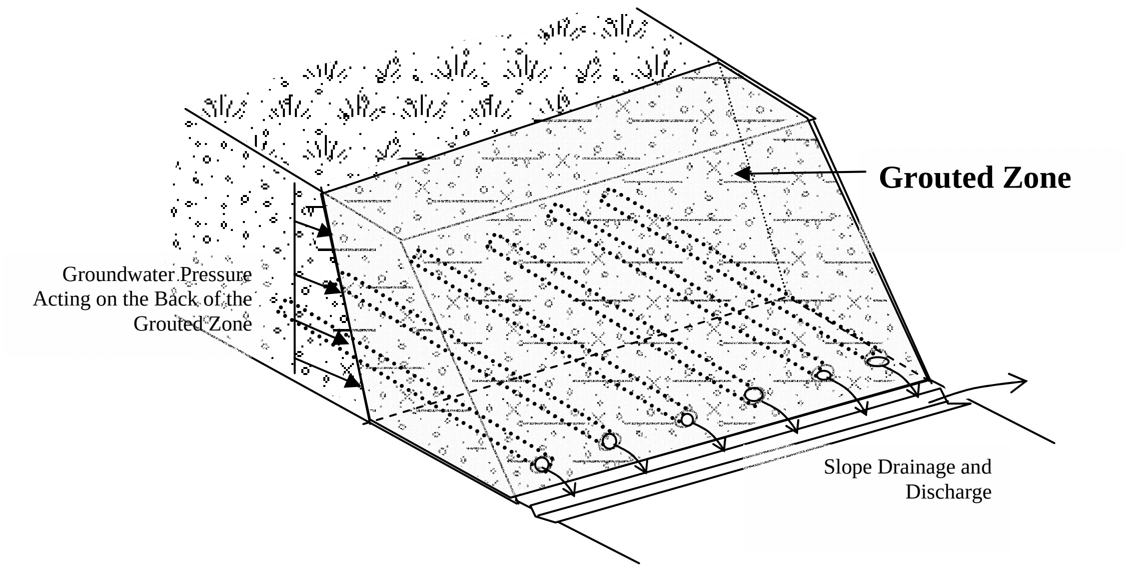


Figure 9.2: Grout stabilised soil section with drainage relief boreholes

Such water pressure could be relieved by the installation of sub-horizontal pressure relief boreholes set at the base of the section.

A disadvantage of both the grouting and surface treatment solutions to the conservation process would be that they would cause irrecoverable changes to the natural chemical balance of the exposed deposits, making scientific chemical studies difficult or impossible.

9.5 Complete enclosure

The ultimate extension of the concepts explored in Section 9.2, 9.3 and 9.4 is to provide complete enclosure of a geological conservation section. Clearly this would not be appropriate for less important sites. However, it is considered that there may be circumstances where, given the nature of the exposed deposits, complete enclosure within which environmental conditions could be controlled and would be justifiable. Precedent for these exists in archaeological sites of international importance, such as those in southern Italy (Pompeii) and in Egypt.

Few sites in the UK perhaps might warrant such treatment, but an example might be the National Nature Reserve at Swanscombe, site of the discovery of one of the oldest Palaeolithic human remains in Europe. This site was the subject of one of the Case Studies that are included as an appendix to this report. The current status of the Swanscombe site is largely of minimal management and subject to casual vandalisation and fly tipping.

It is considered that in order to raise the status of the Swanscombe site, it would be necessary to invest in the formation of a high quality national amenity, which could become a feature for the local community. The amenity might comprise a manned structure around one or more suitable sections (which could be locked), and the formation of a geology/archaeological park to up-grade other areas of the site. A sketch of a possible site layout is shown below as Figure 9.3.

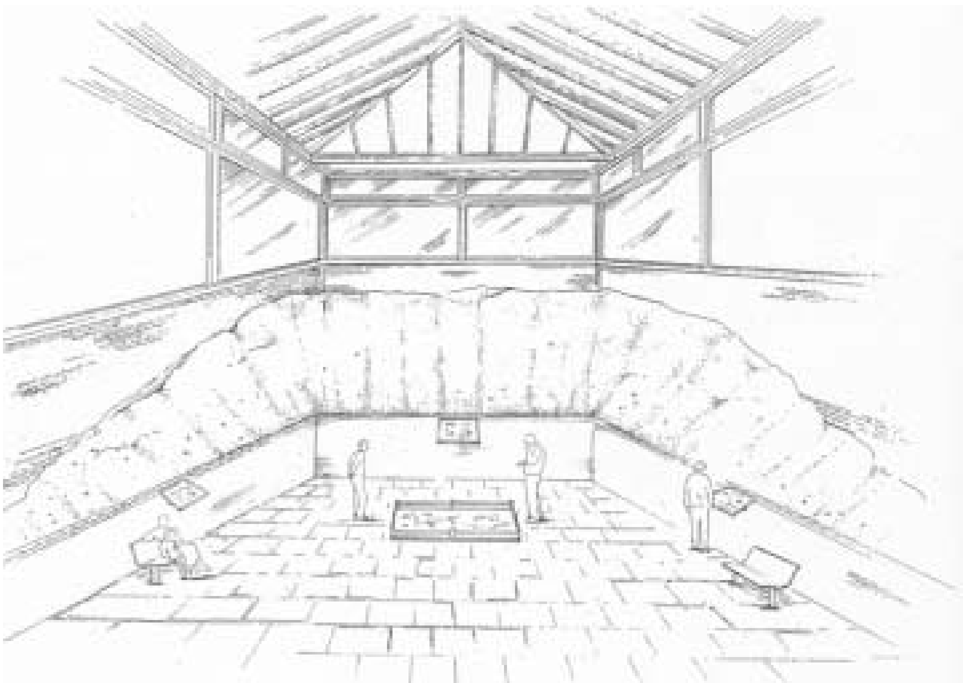


Figure 9.3: Idealistic sketch of conservation faces and floor sections within controlled environmental conditions

The formation of a permanently protected exposure would require careful design, possibly including full scale trials in less sensitive deposits. As well as the provision of drainage behind the structure and the design of appropriate foundations for the permanent enclosure, considerations would also have to be given to the control of temperature and humidity within the structure.

Essentially, the internally exposed slope would comprise unsaturated soils and their stability would rely, to a large extent, on the concept of ‘Soil Suction’.

9.6 Sand/wick drainage

The problems associated with groundwater drainage in sediments of rapidly varying grain size (in a vertical direction) have been described in several sections of this report including Section 6.2 – Groundwater Erosion, and Case Study No. 4, Wolston Pit SSSI.

Potential solutions to this problem have been proposed including: cut-off drainage and counterfort drains. However, both of these have limitations. The depth of influence of cut-off drains is restricted, realistically to about 3 metres (at most 5 metres); whilst counterfort drainage can only be applied to a relatively shallow depth, 1 to 2 metres, into a slope face.

As an alternative, consideration may be given to the creation of a ‘curtain’ of deep vertical drainage using wicks or sand drains, which can penetrate through soft and loose strata to depth of upto about 15 metres. If these are installed behind a slope section, it is possible that they could, locally, bring about a change to the hydrogeological regime from a largely horizontal flow direction to a vertical flow direction.

The application of this process should follow an appropriate hydrogeological investigation. Clearly, the procedure would only be affective if the intercepted horizontal groundwater flow could be discharged to a lower aquifer beneath the depth of influence of the exposed soil section. Where this is not possible, ie: if regional groundwater levels are above the base of the exposed soil, then it would be necessary to provide supplementary drainage, through sub-horizontal water relief boreholes, drilled from the base of the section.

The hydrogeological investigation should be sufficient to allow a prediction of the maximum rate at which water would need to be drained from the base of the face. Under these circumstances the design concept would be to ensure a balance between the available discharge potential from the face and the maximum rate of recharge from groundwater flow.

$$\text{Design Concept} = \frac{\text{Available discharge potential from face drainage}}{\text{Maximum recharge potential from groundwater flow}}$$

Unlike conventional slope stability calculations, the parameters in the above **design concept** contain critical uncertainties. For instance, the available discharge potential for the section would depend upon the effective length of intersection of the boreholes with permeable strata, along with an assumption concerning the efficiency of the boreholes (allowing for the potential for borehole blockage). Similarly the calculation of the recharge potential of the ground behind the slope face would, at best, be an educated estimate, based on a best available hydrogeological data. As a result of these uncertainties, it is considered that the design contains in-built conservatism.

9.7 Key points

This section of the report has looked at some possible alternative sediment stabilisation techniques. Many of these techniques rely on the principle of soil suction, which has only recently begun to be understood in detail. The suggested methods might therefore be regarded as areas where future research might be directed.

The suggested areas of potential research have included:

- face protection, using perspex sheeting;
- face coating to prevent water ingress (or moisture loss);
- soil strengthening using liquid grouts;
- enclosure;
- sand/wick drainage.

10. The way forward

10.1 Introduction

The purpose of this study has been to provide advice to non-technical personnel on issues relating to the conservation of geological sections of national and international importance.

The guidance is aimed primarily at the officers of **English Nature**, but may equally be followed by other stakeholders including: Planning Officers of Local Authorities, landowners, health and safety officials and developers.

It has been proposed in Section 1 of this report that two of the principle aims of geological conservation are as stated in Table 10.1.

Table 10.1: Two of the principle aims of geological conservation
∉ through **'notification'**, the conservation of key sites and exposures to facilitate study by experts;
∉ through **management**, the maintenance of visible and accessible exposures of geologically important sections for scientific study, training and general public benefit.

Conservation, in the broadest sense, centres around the **'notification'** process, under the system of Sites of Special Scientific Interest (SSSI) designation.

Current legislation identifies two scenarios whereby **English Nature** can '<u>intervene</u>' to seek adequate protection of designated geological SSSIs. These are described in Table 10.2.

Table 10.2: Current legislation covering SSSIs
∉ **Article 10 of the Town and Country Planning Act (General Development Procedures Order, 1995)** requires **Local Authorities** to consult with **English Nature** whenever proposed developments (including: construction, mineral exploitation and restoration) may impact on designated SSSIs;
∉ the **Countryside and Rights of Way Act (CROW), 2000**, gives further protection to SSSIs that may not be immediately threatened by proposed development. CROW, 2000 requires that by 2005 a **Management Statement** be prepared for all SSSIs within England and Wales. The Act also gives **English Nature** the power to enforce **Management Schemes** if there is a risk of 'harm' being done to a SSSI.

Where the geological SSSI exposures comprise relatively strong, indurated rocks that can be maintained at steep slope angles, their site conservation has enjoyed measurable success. This is probably because their impact on site redevelopment is relatively small and maintenance requirements are relatively light.

However, **English Nature** has encountered difficulties in conserving and managing faces in unconsolidated sediments. These difficulties arise as a result of both long-term slope stability and slope degradation issues, as well as from stabilisation works carried out during site development by others.

These geological environments often comprise Quaternary sediments deposited in landforms defined by glacial and inter-glacial activity during the past 1.8 or so million years.

One of the problems faced by **English Nature** is that while they have a strong geological understanding of these young and relatively weak sediments, this is not backed up by a geotechnical understanding of soil shear strength, drainage requirements and slope stabilisation issues. As a result, when faced with development proposals, often prepared by technical experts retained by the developer, **English Nature** is poorly equipped to evaluate these proposals and reach objective conclusions concerning the acceptability or otherwise, of development options; and to determine whether conclusions that have been reached are based on appropriate site investigation, testing and analysis.

It is an unfortunate fact, and one that should not be lost sight of, that just about everyone involved with the regeneration of brownfield sites, with or without SSSI status (except English Nature) would prefer to see soil slopes set at shallow angles with a covering of topsoil and vegetation, ie: 'Greening up'.

Slopes that are protected in this way are, by and large, low maintenance, and low risk. Such solutions are therefore favoured by developers, site users and funders. Arguments can be made by geotechnical experts for these solutions on the grounds of CDM and Health and Safety. But even Local Authorities are governed by **Planning Policy Guidances** (eg **PPG14 "Development on Unstable Ground"**), which requires Planning Officers to be satisfied that development proposals have been investigated and designed in sufficient detail that conclusions regarding long-tem stability might be expected to be reliable. However, Planning Officers also have a responsibility to maintain and enhance SSSI interests.

In the situation where the stakeholders have potentially conflicting objectives, an acceptable compromise situation can only be formed by evaluating the different slope options in a transparent manner. The evaluation process could follow a Risk Assessment approach, where various slope design options would be reviewed from the point of view of mitigating the consequences of failure.

This then is the position of **English Nature**, one of whose tasks is the conservation of sections of exposed geological sediments, often in prime positions for re-development, future mineral extraction or restoration through voids filling.

In defining **'the way ahead'**, the final section of this report sets out a number of 'check lists', to provide the guidance needed to arrive at rational solutions to both the conservation issues and proposed redevelopment that may satisfy developer and Local Authority needs, while also being in the National Interest.

10.2 Development considerations

The circumstances whereby English Nature becomes involved with major conservation issues include those listed in Table 10.3.

Table 10.3: Consultation Issues

- development planning applications within influencing distance of a designated SSSI;
- applications to change or extend extant minerals Planning Approvals for future extension of mineral extraction;
- restoration of quarrying activities that are nearing the end of their production life;
- management of otherwise unthreatened SSSIs, for the purpose of improving accessibility and visibility
- Local, regional and national plan and policy development.

10.3 Construction development or redevelopment

New development and redevelopment proposals are lodged with Local Authorities for Planning (or Outline Planning) Permission.

Under **Article 10 of the Town and Country Planning Act,** if these development proposals might have an impact on a designated SSSI, then **Local Authorities** are required to refer the application to **English Nature** as a designated consultee.

At this time **English Nature** would judge whether the proposed development could have a negative impact upon the conservation of the SSSI.

Once the answer to the above is found to be in the affirmative, then **English Nature** should determine if the following planning and geotechnical issues have been resolved:

Planning and geotechnical issues	
Planning	What is the 'sensitivity' of the geological interest and its relationship to development proposals? Can the layout of the proposed development be redesigned in such a way as to **prevent** any impact on the SSSI?
	Can the layout of the proposed development be redesigned in order to **reduce** any impact on the SSSI?
	Can the layout of the proposed development be redesigned in order to **minimise** any impact on existing exposures, but open up new exposures as a compromise?
Site Investigation	Have sufficient exploration points been located through or behind the SSSI sections to be able to adequately define the distribution of the Quaternary sediments? – (see Section 7.3).
	Were the exploration procedures appropriate for the full geotechnical characterisation of the sediments, and were there adequate and sufficient soil samples collected to enable the critical soil strength parameters to be determined? – (see Section 7.4).
	Were sufficient laboratory tests carried out to provide data for the appropriate determination of soil shear strength and other design parameters? – (see Section 7.5).
	Has the local groundwater regime been determined to the extent required for realistic slope stability assessment?

Planning and geotechnical issues	
Geotechnical Design	Have the geotechnical design data been assessed to a suitable standard? – (see Section 5.4).
	Has an appropriate model been generated to define the likely forms of future slope instability in a meaningful manner? – (see Section 6.2).
	Has the geotechnical design for the stability of the SSSI sediments been carried out using the most appropriate procedures? – (see Section 6.3).
	If appropriate investigations and design procedures have been undertaken, have sufficient slope stabilisation options been considered?

It is considered that if the measured answer to any of these questions is in the negative, then **English Nature** might be justified in requesting the Secretary of State to 'call in' any planning proposal. Following this procedure, **English Nature** should seek for appropriate investigation, analysis and stability design of SSSI sections.

It is imperative that time is allowed to address the above. This can be done in three ways:

∉ early/pre-application consultation;

∉ English Nature lodge objection;

∉ ultimately English Nature request a 'call in'.

10.4 Mineral operations and restoration

Minerals planning is assessed under a different Local Authority framework than general development planning. In particular, mineral extraction is governed for the most part by County Council or unitary Mineral Planning Departments.

The role of the Mineral Planning Departments is to consider the local requirements for mineral resources and to balance these with the environmental impact of mineral extraction on the affected communities and infrastructure. Minerals Planning is governed by a tranche of Local Authority guidelines including the Minerals Planning Guidances (MPGs). The requirements of selected guidances have been discussed in Section 4 of this report.

Extant planning permissions for mineral extraction are generally outside the control of **English Nature** even when they contain SSSIs. However, it is generally the case that mineral operators are amenable to allowing access to exposed geological sections for inspection by the scientific community. **English Nature** takes up a monitoring role in these circumstances.

Notwithstanding this, all mineral operators are now governed by the **Quarries Regulations, 1999** (enacted under the **Health and Safety at Work Act, 1974**). These regulations require that assessments are made to ensure that hazards, in the form of active quarry slopes and tips, present minimal risk to site operators, visitors and the local community and environment.

In terms of existing mineral operations, it is considered that **English Nature** can do little other than maintain good relations with minerals operators to enable mineral extraction to progress in unison with scientific study.

However, as the mineral extraction process draws to an end at individual quarry sites, mineral operators often look to exploit the resulting excavation void for landfilling. This may require the establishment of a new Planning Application governing the eventual restoration of the mineral extraction site.

Site investigation methods and design procedures for these works should be no less rigorous than are applied to the construction development opportunities described above. However, the lead-in time scales (which may extend over many years or decades) and the financial incentives for site restoration, provide opportunities for **English Nature** to establish the long-term protection of SSSI exposures.

There is a precedent for the establishment of '**conservation voids'** as part of the quarry filling and restoration programme. These comprise areas that are set aside from the filling and restoration processes, to provide access for the geological and scientific communities for long-term study.

Under the terms of restoration requirements, particularly where these involve large scale landfilling or backfilling, there is the opportunity for **English Nature** to establish a strategy whereby adequate conservation sections can be retained. Procedures exist for the long-term maintenance of these sections through the establishment of **Section 106 Agreements (Town and Country Planning Act, 1990**).

In general terms, a **Section 106 Agreement** allows for the investment, usually by the quarry operator, (or his funder) of a sum of money that, in principle, would provide sufficient revenue for the Local Authority or another body to adopt and maintain the SSSI exposure.

10.5 SSSI management

As opposed to the circumstances described previously, English Nature also have the opportunity to take control of the management of some SSSIs, that might be deemed of particular importance, due to their geological (and also archaeological) interest.

Unlike typical development, or even restoration projects, these management schemes may present opportunities to adopt more radical conservation measures than would otherwise be appropriate, where the potential consequence of slope instability are of low impact.

The case studies that form part of this study, particularly those carried out at Barnfield Pit (Swanscombe), Lion Pit Tramway and Wolston Pit, are examples of SSSIs of considerable geological importance, but which, in each case, fail to meet the second objective of English Nature as set out in Table 10.1. That is that:

'…through management, the maintenance of visible and accessible exposures of geologically important sections for scientific study, training and general public benefit'.

At the times of the site visits, none of these sections was accessible in a geological sense. Each has failed for reasons that, in some circumstances, may be beyond the immediate control of English Nature. However, it is considered that a satisfactory management scheme could be developed for each, provided sufficient investment were made both financially and in terms of site investigation, analysis and vision and technical knowledge.

Relevant stages of investigation and design that should be considered in the development of most schemes are shown in Table 10.4.

Table 10.4: Proposed Stages of Site Investigation and Design	
Desk Study and Site Reconnaissance (see Section 7.2 and 7.3)	Used to define the geological and topographic setting of the site. ∉ establishes physical constraints to stabilisation options (defined by section height and/or width; ∉ establishes constraints to intrusive site investigation (types of exploration equipment, access to exploration point locations); ∉ establishes historical context of the site.
Intrusive Investigation (see Section 7.4)	This should be designed to resolve the following issues: ∉ define the geological conditions in the vicinity of the SSSI section; ∉ provide a detailed description of the ground conditions forming the SSSI section (including distribution of cohesive and granular horizons) and immediate sub-slope ground conditions; ∉ provide information on the groundwater and surface water regimes; ∉ provide suitable samples for geotechnical testing.
Laboratory Testing (see Section 7.5)	This should be sufficient to provide the following design data: ∉ undrained shear strength of cohesive soils; ∉ drained shear strength parameters of granular and cohesive strata (peak); ∉ if there is precedent for historic (or geological) shear surfaces, then residual shear strength parameters should be obtained; ∉ bulk density; ∉ plasticity of cohesive strata; ∉ particle size distribution of granular soils.
Stability Analyses (see Section 6)	These should be based on the most likely forms of instability.
Stabilisation Options (see Section 8 and 9)	These might include the following: ∉ free standing face at a stable slope angle; ∉ formation of multiple slope faces at differing elevations, with support restraint applied in non-critical areas; ∉ provision of drainage to drawdown elevated groundwater levels; ∉ interception of surface and horizontal groundwater flow and discharge away from geological sections; ∉ consideration of alternative slope support methods – see Sections 8 and 9.

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12. Glossary

Term	Definition
Angle of friction	The largest angle that a slope of any height will stand indefinitely.
Back analysis	A slope stability analysis using known geometrical and geological conditions which models a known slope instability. By carrying out a sensitivity analysis, the back analysis of a failure condition can yield shear strength or groundwater conditions that prevailed at the time of failure.
Bearing capacity	The maximum load per unit area that can be accommodated by a soil before shear failure occurs.
Boulders	A separated rock mass larger than a cobble, having diameter greater than 200 mm. It is rounded in form or shaped by abrasion.
Clay	A detrital material of any composition having a diameter less than 0.004 mm.
Cobbles	Rock fragments, rounded or abraded, between 60 and 200 mm in diameter.
Cohesion	That component of the shear strength of a soil or rock that is independent or interparticle friction.
Cohesive soils	Soils that contain more than about 25% of its constituent grains with a nominal dimension less than 60σm (micron).
Compaction	The decrease in pore space of a sediment and consequent reduction in volume or thickness.
Consolidation	The process whereby excess pore water is expelled by a soil under external pressure. This is accompanied by volume reduction.
Drainage	The removal of excess meteoric water by rivers and streams or by seepage from a soil or rock slope.
Free draining	Soils that do not hold water, if allowed to drain - usually uniformly graded sand or gravel.
Geogrid	Synthetic material with an open grid with defined tensile strength in two directions. Used to reinforce layers of compacted soil, or to support loose ground between soil nails or rockbolts.
Geotechnical engineer	An Engineer who specialises in rock mechanics, soil mechanics, foundations and groundwater (preferably qualified through one of the relevant organisations).
Geotextile	Synthetic or natural permeable fabric used in conjunction with soil for erosion control, filtration and drainage.
Grading	The distribution of particle sizes that makes up a soil mass.
Granular soils	Soils that contain more than about 25% of its constituent grains with a nominal dimension greater than 60σm (micron).
Gravel	A detrital particle larger than a sand grain and smaller than a cobble, having a diameter in the range of 2 mm to 60 mm.
Grout	A liquid that is pumped into a soil or rock mass, which then hardens. The effect of the grouted structure is to reduce permeability and increase shear strength.
Gullies	Large erosion channels (larger than rills) formed when rivulets coalesce to produce deeper features.

Term	Definition
Hydraulic gradient	The ratio between the piezometric levels for a unit horizontal distance. the rate and direction of water movement in an aquifer are determined by the permeability and the hydraulic gradient.
Normal stress	That component of stress that is perpendicular to (ie normal to) a given plane. The stress may be either compressive or tensile.
Normally consolidated	A soil that has been subject to consolidation by their own weight, and have not been subjected in their geological history, to additional loads that have subsequently been removed. This may be the erosion of overlying deposits or removal of ice.
Organic soils	Largely comprise peat deposits made up of decaying plant remains. Organic soils can hold prodigious quantities of water, often more by weight than solids.
Over consolidated	A soil that has been subject to consolidation by their own weight and the weight of other materials (including soil and/or ice) that has since been removed. In this case the soil is said to have a stress history.
Overburden	The rocks and/or soil overlying a defined horizon.
Permeability	The measure of the ability of soils or rocks to transmit a fluid. It depends largely upon the size of the pore spaces and their connectedness.
Pore water pressure	The pressure of water contained within a soil sample. This can increase temporarily if a saturated soil is acted on by an external force.
Reinforced soils	The inclusion in a soil mass of layers of metallic, synthetic or natural materials to facilitate construction of a steeper slope than would otherwise stand unsupported.
Rills	Small erosion channels formed with soil slopes caused by uncontrolled surface water runoff.
Rivulets	Uncontrolled surface water runoff at a velocity that is capable of causing erosion to form shallow ‘rills’.
Rock	Any aggregate of minerals, whether consolidated or not. A rock may consist of only one type of mineral, but more commonly contains a variety of minerals.
Rotational failure	A deep seated soil slope failure in which the shape of the failure surface (in cross-section) approximates to an arc of a circle.
Sand	A detrital particle larger than a silt grain and smaller than a gravel, having a diameter in the range of 0.060 to 2 mm.
Seepage forces	A force created by the movement of groundwater under the affect of a hydraulic gradient.
Sensitivity Analysis	A series of stability calculations carried out varying just one parameter and determining the effect of this parameter on factor of safety.
Shear strength	The internal resistance of a body to shear stress. The total sheer strength of an isotropic substance is the sum of the internal friction and the cohesive strength.
Shoreing	The use of timber or other materials to provide temporary support to soil slopes or excavations.
Silt	A detrital particle, finer than very fine sand and coarser than clay, in the range of 0.004 to 0.060 mm.

Term	Definition
Slow draining	Soils that hold water and require a finite time for drainage to occur – usually well graded soils containing a high proportion of fines.
Soakaway	An excavation made for the purpose of allowing collected surface water to soak naturally into the ground.
Soil	A skeletal structure of solid particles in contact, forming a system of interconnecting voids or pores.
Soil nail	A steel or GRP shank that is grouted into a pre-formed borehole to provide slope reinforcement.
Soil sensitivity	The ration of undisturbed soil shear strength and remoulded shear strength.
Soil suction	Negative pore water pressure arising as a result of partial soil saturation. This results in an apparent increase in the component cohesion of soil shear strength.
Stress history	The magnitude and duration of overburden pressure to which an element of soil has been subjected.
Uniformly graded	A soil that contains the majority of its constituent grains at a single size (geologically – well sorted).
Wedge failure	Deep seated soil or rock failure where shear takes place along a preferential plane of weakness.
Well graded	A soil that contains a wide range of particle sizes (geologically – poorly sorted).
Yield strength	The stress at which non-linear stress strain behaviour begins.

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Peter Wakely/English Nature 17,396
Middle left: Co_2 experiment at Roudsea Wood and Mosses NNR, Lancashire.
Peter Wakely/English Nature 21,792
Bottom left: Radio tracking a hare on Pawlett Hams, Somerset.
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