Rock engineering guides to good practice: road rock slope excavation

A J Harber, I M Nettleton, G D Matheson, P McMillan and A J Butler
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by A J Harber, I M Nettleton, G D Matheson, P McMillan and A J Butler

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Foreword

This report is one of three that effectively form a series dealing with key issues related to rock engineering, as follows:

- Rock slope risk assessment.
- Rock engineering guides to good practice: road rock slope excavation.
- Rock engineering guides to good practice: rock slope remedial and maintenance works.

The reports were completed between 1995 and 2000 and although they were circulated to interested parties during the intervening years they were never formally published. The work that informed the reports was undertaken for the predecessor organisation of Transport Scotland over a period of around 20 years. At the request of the Highways Agency, and with the permission of Transport Scotland, these reports are now being published for the first time.

The available time and resources mean that updating and supplementing is not a viable option and the work undertaken to achieve this has been restricted to updating the format to suit the TRL Published Project Report Series and generally tidying up the unpublished versions. The sole major exception to this is the report on Rock slope risk assessment to which an appendix has been added on ravelling. This is intended to open up the system reported and to render it more usable for rock slopes in southern England as well as those in Scotland for which it was originally designed. This appendix was prepared by Ian Nettleton (Coffey Geotechnics), who was closely involved in the application of the system while in the employ of TRL. The authorship of this report has been amended to account for this addition.

This report, Rock engineering guides to good practice: road rock slope excavation, covers the definition of, background and the main issues for rock excavation; the stability of excavated rock slopes; and investigation (including personnel, objectives, procedures, data and ground investigation).

Rock slope design is covered including the provision and design of rock traps before an extensive section that covers issues relating to rock slope excavation. These include types of excavation, blasting, fracture formation and propagation, blasting techniques and design, drilling operations for blasting and vibrations from blasting.

The final section covers environmental considerations. It should be noted that this section is strongly weighted towards both the content of the Design Manual for Roads and Bridges extant at the time and to policies specific to Scotland.

I sincerely hope that these reports will be subject to wide industry uptake as have so many TRL Reports before them.

Dr Mike Winter
Head of Ground Engineering
March 2011
Abstract

This report provides advice and guidance on good practice in road rock slope excavation. The subjects covered include rock slope stability, site investigation, rock slope design, rock slope excavation and environmental considerations.

1 Introduction

The UK has a long history of rock excavation for the construction of transport infrastructure. Indeed the success of the early roads, canals, railways and the relatively recent motorway and trunk road systems, have been based on practical rock excavation experience developed through the ages. Extensive use is still being made of the many years of accumulated knowledge and experience, particularly in the hilly and mountainous terrain of Scotland, Northern England and Wales. In the road context, the evolution of theory and practice has been particularly rapid during the 20th century and many new techniques have been developed. However, these have not always achieved the success initially intended or claimed. It is therefore important that best practice is documented.

This guide concentrates on the main issues, describing concepts and techniques with a proven track record. The aim is to provide simple but sound practical guidance. A complete coverage of all issues is not intended.

Much of the information comes from the program of research carried out by the Transport Research Laboratory for the precursor organisation to what is now Transport Scotland over the 20 years prior to first release. This work has been the subject of numerous published and unpublished reports and papers. The intention of the present guide is to collect, collate and present the results of this research, and the experience gained, in a single report. Where the work of others is used it is referenced.

Although specifically targeted at geologists and engineers involved in road rock slopes in the UK, most of the concepts and techniques are applicable worldwide, to all projects involving the excavation of rock, including surface mining, quarrying and mineral extraction.

The style of the guide is illustrative and the issues covered vary from the simple to the complex. It is thus suitable for readers with a wide range of expertise and experience, including those in management who need a broad understanding of the concepts involved.

1.1 Definitions - What, When and How?

1.1.1 What is Rock Excavation?

Rock excavation in this guide is taken to mean the excavation of naturally occurring rock during the construction of a road project. It involves forming the rock slopes bounding the excavation and removing the rock in the excavation area. With careful design and time, the excavated area can blend in with the local scenery.

1.1.2 When is Rock Excavation necessary?

Rock excavation becomes necessary when any part of the design of a road intersects geological rock head. It becomes increasingly important in areas of high topographic contrast and with increasing standards of horizontal and vertical alignment (Figure 1.1).
1.1.3 *How is Rock Excavation carried out?*
Rock material is fragmented, heaved and loosened to enable excavation. Excavation can be carried out by manual, mechanical, or explosive means. The choice is dictated by many factors, including the strength of the rock mass, the degree of natural fracturing, and the local cost-effectiveness of the methods available (Figure 1.2).

1.2 *Background of Rock Excavation*

1.2.1 *Rock Excavation in the UK*
There is a long tradition of rock excavation in the UK. Considerable expertise was developed and experience gained during the construction of roads, canals, railways and bridges. Rock excavation has progressed from being a craft to a science. However, not all methods which have been developed have been successful, particularly where the geotechnical conditions have been ignored.

Rock has been excavated in the construction of roads from the inception of a transport system in the UK - at least since Roman times. Initially cuttings were small and the minimum required to allow passage (Figures 1.3 and 1.4). However, increasing infrastructure design standards have necessitated larger excavations to accommodate improved horizontal and vertical alignments (Figures 1.5 and 1.6).
The main issues that need to be considered in the process of designing and forming a rock excavation are: the causes of instability; ground conditions; rock slope design; excavation techniques; and environmental considerations.

1.3.1 **Rock Slope Excavation**

When forming a new road rock slope the objective is to ensure that the finished excavation does not pose an unacceptable risk to the road infrastructure or users. The achievement of this objective requires a fundamental understanding of the causes of risk from rock excavations. All of the stages of investigation, design and formation of a rock slope are influenced by this understanding.

There is much guidance available on site investigation, but only a very small portion is of direct relevance to design and excavation of rock slopes. It is essential that the investigation provides sufficient data on the critical design parameters in each case. Therefore, the various elements of the investigation should have a clearly defined purpose in terms of the type and amount of data to be recovered and for what end use. Failings at this stage are invariably costly and could be dangerous, to both the project and personnel.
A rock cutting is made up of a rock slope and an appropriately designed rock trap. A rock slope is any slope in rock material, whether it is excavated as part of a contract, is pre-existing or forms a natural rock outcrop. A rock trap is designed to contain any rock fall material, and to prevent it reaching the carriageway. The rock trap design for each rock slope should be optimised to provide the highest level of rock fall material retention that is practicably possible. The design of rock slopes and rock traps must be considered simultaneously as they are inherently linked.

Specification is an often overlooked stage in achieving a good result in rock excavation. The specification is a translation of the client's needs and a designer's intentions into a contractual requirement. Many good intentions and designs fail because of poor specification.

The design and specification need to take into account the appropriate excavation technique for each slope. Execution of the excavation design needs to utilise best practice to ensure that the intentions of the designer and needs of the client are realised.

Environmental considerations are of ever increasing importance and are an area of potential conflict with design objectives. The designer must ensure that a sustainable approach is taken to design, but must also deliver the required product, in terms of buildability and performance.
2 Rock Slope Stability

2.1 Introduction - What are Stability and Instability?

In absolute terms any structure that sits above the mean horizontal plane is in a meta-stable state. That is to say, in the long term there is a tendency for pronounced topographical highs to be reduced. However, this view is of little use in engineering terms. A more accepted approach is to define stability in terms of local mechanical equilibrium. For rock slopes this approach also presents problems, as many of the forces acting are transient and difficult to define. For the purposes of this guide therefore, a stable rock slope is one on which there are no active failures, such slopes are rare. Instability therefore implies that there are active failures (Figure 2.1).

![Figure 2.1 Stable and Unstable Rock Slopes (A835, Garve and a quarry slope).](image)

2.2 Factors Affecting the Stability of Rock Slopes

2.2.1 Types of Factors

Instability on rock slopes can be classified as either natural or induced and is influenced by a wide range of factors, some of which are controllable and others are uncontrollable (Figure 2.2). Natural instability has come about without the intervention of man. Induced instability is the result of the action of man. Controllable factors can be influenced by man’s actions while uncontrollable factors cannot be influenced by man. The rock in which slopes are formed may vary considerably. The properties of the rock mass are critical to determining the nature of instability that is present on a rock slope. Where slopes are formed in rocks in a high material strength i.e. moderately strong or stronger (BS5930:1999) the stability of the slope is largely controlled by discontinuities present in the rock mass and material strength is a secondary factor. Where slopes are formed in rocks with a low material strength i.e. moderately weak or weaker (BS5930:1999) the stability of the slope is controlled by a combination of material strength and discontinuities. Discontinuities are any flaws or weaknesses in the rock such as fractures, joints, bedding planes, fabrics (e.g. schistocity), etc, which separate the intact rock material into a discontinuous rock mass.
Natural instability in weak rock is both a function of the rock strength and it's susceptibility to weathering. In strong rock it is a function of the geometric relationships between the discontinuities present in the rock mass and the final face. The potential for natural instability is inherent in the natural rock mass before excavation. Excavation techniques induce further instability by disturbing the rock mass, through dilation of natural discontinuities and the production new fractures. Where stability cannot be achieved through design and there may be a risk to the road, costly remedial and maintenance work may be required.
2.2.2 Natural Instability

2.2.2.1 Strong Rock

Strong rock is taken to be rock material that is moderately strong or stronger (BS5930:1999). The data required for rock excavation design are the field characteristics of the discontinuities within the rock mass, and not the laboratory testing of intact samples. Certain rock, which may appear strong during site investigation and excavation, may weather rapidly to become weak, for example several types of volcanic deposits in central Scotland. This type of material should be treated as weak rock in terms of long term stability.

2.2.2.2 Weak Rock

Weak rock is taken to be rock material weaker than moderately strong (BS5930:1999). In weak rock, instability is usually caused by the action of weathering on the exposed face. Weathering of the rock mass can cause physical and mineralogical changes, which may weaken the rock mass further and cause progressive degradation (Figure 2.3).

![Figure 2.3 Extensive weathering near rockhead in shales/greywacke with overlying glacial deposits (A68, near Langholm).](image)

Even if it is possible to excavate relatively steep slopes, they may not be maintained in the long term at this angle. In terms of stability, excavations in weak rocks are often best treated as soil slopes. However, it is still critical to understand the geometry and influence of discontinuities.
2.2.2.3  Heterogeneous Rock Masses

Heterogeneous rock masses may contain rock material of more than one rock type or physical characteristic. More importantly such rock masses may contain heterogeneities in terms of rock strength. Such slopes include those made up of a mixture of strong and weak rock material, for example certain sedimentary and volcanic deposits or deeply weathered rock masses. Other heterogeneous rock masses may include features such as igneous intrusions (for example dykes and sills) or structurally complex areas containing folding and faults or fault systems. In certain areas igneous features such as dykes can form positive or negative topographical features, depending on the relative level of strength and weathering between the intrusion and the country rock. In rock excavations in heterogeneous rock masses, long term stability may be significantly controlled by the weaker material (Figure 2.4).

Figure 2.4 Heterogeneous rockmass - deep weathering along discontinuities and subsequent development of corestones in a doleritic intrusion (Back o'Hill, Stirling).

2.2.2.4  Other Geological and Geomorphological Features

There are many other geological and geomorphological features that may influence the stability of a rock slope and associated superficial excavations. Examples of such processes or features, which may be active or ancient, include rock or landslides, valley bulging and cambering, glaciation and solifluction. Particular geological and/or geomorphological areas may be prone to certain types of processes or features, for example solution features in areas of limestone.

Scree deposits form at the base of rock outcrops (Figure 2.5). They are made up of fragments of the rock mass that have become detached from the rock face. Scree deposits may be present at rock excavation sites. They behave as very coarse grained soils rather than rock masses. Undisturbed scree slopes are likely to be at, or near, their critical stability angle. The best indication of the natural
angle of stability is by observing the attitude of stable local slopes in the same material. It may not be possible to steepen the slopes further without adding support. Erosion protection and establishing vegetation may be used to increase stability.

![Figure 2.5 Natural scree slopes formed from volcanic lavas. Vegetation has established in the stable areas, at or below the critical stability angle (Loch Awe).](image)

2.2.3 **Induced Instability**
Many excavation techniques have an effect on the rock mass outside the immediate excavation area. Induced instability is damage to the host rock adjacent to the excavation and is most apparent in strong rock. The damage is characterised by an increase in the degree of fracturing, with dilation (disturbance) of both natural and introduced fractures, and a corresponding increase in potential instability. Such instability is superimposed on any natural instability that may already be present.

2.2.4 **Failure Mechanisms**
This guide concentrates on slopes formed in rocks that are moderately strong or stronger, i.e. slopes in which instability is largely discontinuity controlled. In such slopes the spatial distribution of discontinuities and their relationships to the excavation geometry determine the mechanism and size of potential failures.

The process of investigating the nature, frequency and scale of potential failures is **stability assessment**. A stability assessment involves the **analysis** of data collected from a rock mass.

**Plane, wedge** and **toppling** are the main failure mechanisms on rock slopes.
Plane failure - sliding movement of a rock block or mass along a discontinuity plane.

**General Conditions for Failure:**
- Dip of plane > Friction angle
- Azimuth of the plane within ±20° of the slope face azimuth
- Plane daylights on the slope

Wedge failure - sliding movement of a wedge-shaped rock block or mass.

**General Conditions for Failure:**
- Two intersecting planes dipping towards each other
- Sliding along both planes in line with intersection
- Dip of the intersection > friction angles on planes
- Intersection line daylights on the slope

Toppling failure - failure involving rotation of rock columns or blocks.

**General Conditions for Failure:**
- It requires at least two steep release planes and one basal release plane
- The basal release can be either a single plane or the intersection of two planes
- If the basal release is a single plane, it must dip out of the slope less than the rock’s angle of friction.
- If the basal release is two planes, the intersection must dip out of the slope at less than the rock’s angle of friction, and the azimuth must be +/-20° relative to the slope.
- The intersection of the two steep planes must dip into the slope at >60°.
- Toppling Wedges are a special case of wedge geometry and are formed where the intersection of the wedge is dipping out of the face at less than the rock’s angle of friction AND there is a single steep plane dipping into the slope at >60° and with an azimuth of 160° to 200° relative to the slope azimuth.
A fourth common failure mechanism is **ravelling failure** and this applies to small-scale, superficial rock fall, for which there is no defined mechanism:

**Ravelling failure** - small blocks of rock that become detached from the rock mass.

2.2.4.1 **Composite failure**

Where a failure geometry is repeated frequently throughout a rock mass or more than one failure geometry is present within a localised volume of a rock mass it is possible for composite failure to occur. In the former case the composite failure is made up of a “family” of failures of the same mechanism but differing in size. In the latter case different failure geometries interact to produce a composite geometry that is the limiting geometry in terms of stability.

All of the failure mechanisms described above are convenient models that approximate potential failure to relatively simple geometries that lend themselves to calculation. In reality the failure geometry may be complex, a best-fit model may be difficult to define and it may not fit any of the classical models described above. It is sometimes advantageous to apply more than one model to a potential failure mass to assess how it is most likely to fail.
3 Site investigation

3.1 Introduction

As Littlejohn et al (1994) state "Without site investigation ground is a hazard”. This statement is as true for the excavation of rock slopes as it is for any other branch of ground engineering. Indeed, without a well planned and targeted site investigation our knowledge of the structure, material variability and material characteristics of the ground are purely speculation. What sensible or financially responsible individual would risk the successful completion of a construction project, a budget or human life on pure speculation? This has major implications with the current HSE (1992) "Construction: Design and Management" (CDM) regulations.

The objective of a site investigation is to provide information on the structure, material variability and material characteristics of the ground, for use in the design of structures and earthworks (both rock and soil). However, as a site investigation can only provide information from limited “exploration points”, another equally important objective of any site investigation is to provide information on the location and nature of areas of the ground in which there are uncertainties. These uncertainties can either be dealt with by further site investigation, or where this is not practical, by making allowance for the increased risk in the design and budgets.

This guide concentrates specifically on those matters pertaining to site investigation for the design and excavation of road rock slopes. For general details of site investigation, reference should be made to BS 5930:1999 “Code of Practice for Site Investigations”.

Site investigation is the generic name given to all aspects of an investigation of a site. It encompasses literature surveys, study of existing maps and plans, study of photographs, site inspections, site and regional mapping (of all types) and ground investigation. Ground investigation refers to all exploration of the ground by “methods” such as excavations, boreholes, probing and geophysical surveying. Ground investigation also includes the taking of samples and testing (laboratory and in-situ).

3.2 Site Investigation Personnel

A “Geotechnical Advisor” should be responsible for planning, design, execution and supervision of a site investigation, from conception to completion. He may delegate tasks to “Geotechnical Specialists”, who may in turn delegate items of work to “Engineering Geologists”, “Geotechnical Engineers” or “Geotechnicians”. Detailed definitions of these personnel are given in Site Investigation Steering Group (1993) and BS 5930 (1999).

All personnel should be “competent” to undertake the work for which they are responsible. “Competent” can be defined as having sufficient training, experience knowledge and ability to enable duties to be undertaken. This requirement should overrule all others.

3.3 Objectives of Site Investigation

The main objectives of any site investigation for the design and excavation of road rock slopes is to provide information to allow:
Design and stability assessment of the rock slopes and any upper slopes, this will also allow:

- Estimation of the required land-take.
- Estimation of the earthworks quantities.
- Assessment of the excavatability of the rock mass.
- Estimation of the potential re-uses of the excavated material.

It is essential that the investigation be designed with the specific objectives in mind, and that the specific requirement of each element of the investigation with respect to the objectives is understood and defined. In essence each element of the site investigation should have a predefined purpose related to the main objectives.

All site investigation information should be recorded in general accordance with BS 5930 (1999) or similar robust and widely recognised standard (e.g. ISRM 1981, Geological Society Engineering Group Working Party 1970, 1977 and 1995). The standards used should be referenced in any reports or data produced.

### 3.3.1 Design and Stability Assessment

For the construction of road cuttings in rock, stability assessment and design for both the rock slopes and any upper slopes in superficial material must be undertaken. To enable this consideration of the information shown in Table 3.1 is required.
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<th>Geotechnical Factors</th>
<th>Description and Significance</th>
<th>Appropriate Site Investigation Techniques</th>
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<tr>
<td>Regional and Local Geological Structure</td>
<td>The presence and nature of folding, faulting and igneous intrusions and any other significant geological features should be determined. Knowledge of the regional geological structure can help with the understanding of the geology of a site where there is poor exposure, or in areas of complex structural geology.</td>
<td>Can be determined from literature reviews and published geological maps (at least 1:50,000 or greater). In some cases (poorly mapped areas) field geological mapping may be required (Section 3.5).</td>
</tr>
<tr>
<td>Geological Structure of the Site</td>
<td>The presence and nature of folding, faulting and igneous intrusions and any other significant geological features should be determined. Knowledge of the local geological structure is essential for understanding the geology of a site where there is poor exposure, or in areas of complex structural geology.</td>
<td>Can be determined from literature reviews and published geological maps, but will probably require field geological mapping (Section 3.5), at a large scale (1:500 down to 1:10,000). In poorly exposed areas Ground Investigation techniques may be required (Section 3.6).</td>
</tr>
<tr>
<td>Local Lithologies</td>
<td>The characteristics, distribution and sequence of soils and rocks underlying the site should be determined.</td>
<td>Can be predicted from geological maps and observed in surface exposures, but will may require Ground Investigation techniques (Section 3.6) such as trial pits and trenches, auguring and probing, and boreholes.</td>
</tr>
<tr>
<td>Rock Material &amp; Mass Characteristics</td>
<td>The characteristics of both the rock material and the rock mass should be determined. Characteristics include rock type, strength, structure, material weathering, mass weathering, groundwater, discontinuities (see below) etc.</td>
<td>Best determined from geotechnical mapping of field exposures (Section 3.5). If only limited exposures are available then trial pits or trenches, rock core from boreholes, and/or geophysical logging of boreholes can be used. In the field rock strength can be determined from manual test, and may be validated with simple index tests such as Point Load Index or Schmidt Hammer (BS 5930 1999 and ISRM 1981). Samples may also be taken for laboratory testing.</td>
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<tr>
<td>Discontinuity Geometry and Characteristics</td>
<td>The main controlling discontinuities should be identified and grouped into discontinuity sets. The dip, azimuth, principal spacing, trace length, dilation, wall strength, weathering, infill and seepage should all be determined for each set. These factors will allow the stability of the rock slopes to be determined.</td>
<td>Best determined from geotechnical mapping of field exposures (Section 3.5). If only limited exposures are available then trial pits or trenches, rock core from boreholes, or geophysical logging of boreholes can be used. Borehole data cannot be used to obtain trace length data, and generally cannot be used for determining the discontinuity sets. Boreholes should be</td>
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<tr>
<td>Geotechnical Factors</td>
<td>Description and Significance</td>
<td>Appropriate Site Investigation Techniques</td>
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<tr>
<td>Geomorphologic Controls</td>
<td>The presence and nature of any geomorphologic controls, such as streams or drainage systems, depressions, canyons, etc.</td>
<td>The presence of any geomorphologic controls should be determined. Mining, energy production, and other activities may impact the stability of the site.</td>
</tr>
<tr>
<td>Ground Water</td>
<td>The presence and piezometric level of any ground water should be determined. Unfavorable ground water may also have significant effects upon any structures to be built. The nature of the ground water may be the only source of water for isolated communities.</td>
<td>The presence and piezometric level of any ground water can be determined from monitored boreholes and from piezometer installations (BS 5930:1999). Samples can be taken for chemical analysis.</td>
</tr>
<tr>
<td>Rock Head and Superficial Materials</td>
<td>The location and nature of rock head need to be determined for the design of rock slopes and any upper slopes above them. The dip and azimuth of rock head can lead to stability problems with the upper slope.</td>
<td>Rock head location and orientation can be determined from mapping of exposures, trial pits, and trenches, boreholes, and from geophysical techniques (seismic refraction, resistivity). Auguring and probing can give an indication but are not reliable.</td>
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Table 3.1 Geotechnical factors and their significance for the design and stability assessment of rock slopes and upper slopes.
3.3.2 Location and Nature of Rock Head

Rock head is defined in Section 4.2.2. The location of rock head is one of the most fundamental parts of the site investigation process for rock slope excavations. The location of rock head (depth to) significantly affects the proportion of a cutting that will be in rock, as opposed to superficial materials (Figure 3.1). This affects the design of the cutting, requirement for land-take and earthworks quantities. In addition, the topography of rock head at a proposed cutting is important. If rock head dips into an excavation stability problems may arise at the interface between the rock and superficial material.

Rock head is a three-dimensional surface which can have a very complex topography, especially in glaciated, fluvial and weathered terrains. Hence, it is important when data is being collected and reviewed, that an accurate three-dimensional understanding of how and where the rock head surface intercepts the excavation area is obtained. This can be achieved by plotting of all relevant data on an accurate base map. A three-dimensional representation of the rock head surface can then be determined by interpolation and extrapolation. The position of rock head must be established before the design of the rock slopes and upper slopes.

![Figure 3.1 The importance of locating rock head. Note extra land-take, decreased volume of rock from excavation, increased volumes of superficial material to be excavated and larger upper slope.](image)

3.3.3 Excavatability of Rock

The excavatability of rock is generally taken to be determined by the strength of the rock material and the block size (degree of fracturing), within the rock mass (Pettifer & Fookes, 1994). The strength of a rock mass can be determined from manual, unconfined compressive strength (UCS), point load or schmidt hammer tests on rock samples or cores. The rock mass block size is controlled by the discontinuities within a rock mass. A minimum of three main delimiting discontinuity sets is required to define blocks within a rock mass. The block size within a rock mass can be determined from:
Discontinuity (fracture) spacing index $I_f$ - $I_f$ should where possible be determined from three-dimensional measurements (BS 5930:1999 and Pettifer & Fookes, 1994).

Volumetric joint count $J_v$ (ISRM 1981) - the sum of the number of discontinuities per metre for each discontinuity set.

Natural Block Size Distribution (NBSD) curves (Figure 5.20) (Wang et al 1992, Lu & Latham 1996) - the NBSD is essentially an in-situ natural block size distribution curve for a particular rock mass. This can be determined (Wang et al 1992) from discontinuity orientation and principal spacing data (spacing normal to discontinuity set orientation). These data can be obtained by field outcrop mapping, exposure scanline mapping, or from boreholes by logging core or geophysical surveys e.g. borehole CCTV, “Televiewer” or formation micro-scanner surveys (McMillan et al 1995, Nettleton 1993).

With these techniques it is essential to gain representative data in three-dimensions. This can be achieved by optimally orientating scanlines and boreholes, and by exposure mapping from three orthogonal outcrop faces.

Another approach to determining the excavatability of rock masses utilises the dependence of the seismic velocity on the strength and degree of fracturing of the rock mass (Caterpillar Tractor Company, 1982) (Figure 5.3). The seismic velocity of the rock mass can be determined from surface or borehole seismic refraction surveys (Kearey & Brooks, 1990) (Section 3.6.4).

\subsection*{3.3.4 Re-Use of Materials}

The potential re-use of rock materials will generally be governed by the lithologies (types of material) present, their material characteristics and their block size distributions.

The lithologies can be determined from exposure mapping, trial pits and trenches, auguring and probing, and boreholes. The material characteristics of the rock mass can be determined from samples of the rock obtained from exposure mapping, trial pits and trenches, and borehole rock cores. The characteristics required will vary depending upon the end use envisaged. Examples of the required characteristics for aggregates are given in The Geological Society (1993 & 1998) and for coastal and shoreline protection in CIRIA (1991). Rock testing methods are given in ISRM (1981). In many cases the mineralogical characteristics of the rock will influence the potential re-use of the excavated rock, in particular the presence of minerals prone to physical or chemical degradation (e.g. mica, chlorite) is important.

For a rock mass excavated by explosives, the post-excitation Block Size Distribution (BSD) is dependent upon the NBSD of the rock mass and the method of excvation (Section 5.4.2). To determine the NBSD, the methods given in Section 3.3.3 can be used. To determine the BSD from the NBSD the details of the blasting must be known. Essentially the excavation process causes the NBSD curve to be translated to a finer block size distribution, the BSD (Figure 5.21).

The presence of any possible “contaminant materials” that may render the re-use of excavated rock impracticable should also be determined, and the extent of such materials investigated. Contaminant materials may be undesirable due to their own physical or chemical characteristics, or due to the affect that they may have on other materials (e.g. sulphate attack on concrete structures). Contaminant materials include:

- Clay bands, spar veins or mineral veins within limestone.
- Weathered pelitic schist within psammitic schist.
• Weathered granite within unweathered granite.
• Other weathered rock material within unweathered material, for example weathered dykes and sills within intact country rock.

### 3.4 Site Investigation Procedure

A site investigation should have a systematic and staged approach. Relevant information and uncertainties determined from each stage can then be verified, and further investigated, at a subsequent stage, to optimise the site investigation process. In addition, the project’s route/alignment selection and conceptual designs can be tailored to the ground conditions.

Site investigation is the generic term that encompasses the following investigation stages (BS 5930:1999):

- **Stage 1:** Desk Study and Site Reconnaissance
- **Stage 2:** Detailed Investigations
- **Stage 3:** Construction Review

These stages and the corresponding site investigation and design activities are shown in Figure 3.2. The three stages of a site investigation are discussed in the following sections.

#### 3.4.1 Desk Study and Site Reconnaissance

At the feasibility study stage of a project, when conceptual designs are being considered, a desk study and site reconnaissance should be undertaken. If various routes or alignments are being considered, then the desk study and site reconnaissance can influence the options at an early and inexpensive stage.
<table>
<thead>
<tr>
<th>Site Investigation Stages</th>
<th>Sub-Stage</th>
<th>Site Investigation Activities</th>
<th>Design Activities</th>
</tr>
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<tbody>
<tr>
<td>Stage 1</td>
<td></td>
<td>Desk study to determine likely ground conditions</td>
<td>Route \ alignment selection &amp; conceptual designs</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Identify uncertainties &amp; major geotechnical hazards</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reconnaissance to confirm Desk Study</td>
<td></td>
</tr>
<tr>
<td>Stage 2</td>
<td></td>
<td>Desk Study &amp; Reconnaissance Report</td>
<td>Preliminary Design</td>
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<tr>
<td></td>
<td></td>
<td>Design of Preliminary Site Investigation</td>
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<tr>
<td></td>
<td></td>
<td>Preliminary Site Investigation Report</td>
<td></td>
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<td></td>
<td></td>
<td>Design of Main Site Investigation</td>
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<tr>
<td></td>
<td></td>
<td>Information from Main Site Investigation</td>
<td>Modify preliminary design</td>
</tr>
<tr>
<td>Stage 3</td>
<td></td>
<td>Main Site Investigation Report</td>
<td>Final Design</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Observation of ground conditions encountered</td>
<td>Modify design</td>
</tr>
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<td></td>
<td></td>
<td>Further investigation</td>
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<td>Completion of Construction</td>
<td></td>
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<tr>
<td></td>
<td></td>
<td>Inspection regime</td>
<td>Maintenance works</td>
</tr>
</tbody>
</table>

Information Transfer between Site Investigation and Design Activities
Activity Flow Path

Figure 3.2 Stages of a site investigation and the corresponding design activities (modified from GEO 1987).
An example of poor practice is to select a new road route without due consideration to ground conditions. In the case of rock slopes this means that there is little or no room for optimising geometric design. Such practice leaves little scope for achieving a cost-effective alignment that minimises the cost of dealing with, and implications of, unfavourable ground conditions. On construction of slopes this oversight may well lead to additional rock slope remedial works and their associated costs. Typically, such remedial works have long term maintenance issues, adding to the whole-life cost of the slopes.

The purpose of desk study and the site reconnaissance is to enable the identification of:

- Potential hazards to the project.
- Uncertainties about a site.
- Requirements of the subsequent site investigation stages.

Hazards to any investigations or the project (for example contaminated land, peat bog, etc.) and restrictions (for example access restrictions, environmental restrictions, etc.) to the subsequent stages of the site investigation.

### 3.4.2 Desk Study

The desk study should assimilate all the available existing information about a site. Lists of possible sources for existing site information can be found in Perry & West (1996) and BS 5930 (1999).

#### 3.4.2.1 Site Reconnaissance

A site reconnaissance should be carried out to confirm and supplement the information gathered during the desk study. Site reconnaissance requires a thorough visual examination of the site and its environs. Pertinent information should also be gathered from ground adjacent to the site. BS 5930 (1999) gives a detailed list of the procedure and the main points to be considered in a site reconnaissance.

The site reconnaissance should also consider the site with regard to the follow-up stages of the site investigation. In particular access, services and environmental constraints should be identified and recorded. A good photographic record should be made during any site reconnaissance.

### 3.4.3 Detailed Investigations

Detailed investigations of a site should be undertaken in at least two phases. These two phases are referred to as the Preliminary Investigation and the Main Investigation (Figure 3.2). Each of these phases should aim to verify and expand upon information and uncertainties already gathered.

The purposes of the Preliminary Investigation is to provide reliable and detailed information to:

- Confirm or amend the route/alignment selection and conceptual designs (if not already confirmed by the desk study and site reconnaissance).
- Provide information for the preliminary design works.
- Provide information for the design of the main investigation.

The purposes of the Main Investigation are to provide reliable and detailed information for the production of a safe and economic design.
3.4.4 Construction Review

Due to the complexity and variability of the ground, and the limited number of “exploration points” from a site investigation, it is likely that the ground conditions encountered during construction will vary to some degree from those predicted by the site investigation.

The purpose of the construction review is to record and monitor the ground conditions encountered. This will allow the findings of the site investigation to be validated, and revised. If there are significant differences between predicted and actual ground conditions then the design and/or construction techniques may have to be amended.

The construction review should be an ongoing process throughout the construction phase. Accurate descriptions of all geological and geotechnical structures, materials and features encountered should be made by a “Competent” geotechnical specialist, engineering geologist or geotechnical engineer (Section 3.2).

Accurate records should be agreed and kept by both the Engineer and the Contractor. The exact record management system will vary depending upon the type of contract.

The purpose of the records is outlined in BS 5930 (1999) and is summarised below:

- Check the adequacy of the design.
- Check safety of permanent and temporary works, and working methods.
- Validate and revise site investigation findings.
- Reassess construction methods and plant in light of actual ground conditions.
- Provide agreed records on ground conditions in the event of a dispute.
- Check suitability of proposed instrumentation and locations.
- Validate and reassess the potential re-use of excavated materials.

3.5 Engineering Geological Field Data

The main objectives of engineering geological field data collection is to:

- Establish the regional situation.
- Establish the local situation.
- Identify trends and variations.
- Identify areas with similar geotechnical characteristics (structural domains).
- Record geotechnical data from within structural domains.

3.5.1 Engineering Geological Field Mapping

Field mapping to establish the local geology and geological structure, and an accurate interpretation of these is vital for the design and excavation of rock slopes. The degree of bedrock exposure is usually the controlling factor in determining the accuracy of the geological map and the amount of interpolation and/or extrapolation required. This latter aspect is illustrated in Figure 3.3.

![Figure 3.3 The effect of the degree of exposure on data collection.](image)

In the unlikely event of full exposure over the proposed excavation all the data may be usable in the design. The only inference required would be that conditions at depth are the same as those on the surface, where the data were collected. With partial or poor exposure, interpolation and/or extrapolation will be required. Even if no exposures are available in the immediate area, knowledge of the local geology and geomorphology may permit extrapolation from well outside the area of immediate interest.

If it is not possible to refer data to the local geology then they should be collected and recorded separately on an individual outcrop or part outcrop basis.

3.5.1.1 Geotechnical Data Collection

Data relating to the rock mass and the discontinuities are collected in the field by observation and measurement. The data collected must be fit for purpose, i.e. it must be appropriate and adequate for the required task.

For the rock mass, a full description of all rock materials and their condition should be produced in accordance with BS 5930 (1999) or another appropriate standard or system. Any other relevant descriptive data that will be helpful in design should be included with the description.

The minimum information required about discontinuities is their type, orientation (defined by the dip and azimuth), principal spacing and trace length. Additional information, such as the dilation of the discontinuity, the presence of any infill
and the strength and weathering of the discontinuity walls may also be required (The Geological Society 1977).

3.5.1.2 Sampling

The objective in data collection is to obtain a representative sample of relevant discontinuity characteristics present in the rock mass. Such data can be collected using many sampling techniques, including Systematic and Judicious. Systematic sampling is based on Complete, Area or Scanline mapping. Judicious sampling relies on the expertise and experience of the collector to obtain a representative sample by a limited number of measurements. The principles of the techniques are illustrated in Figures 3.4, to 3.7.

When using any of the methods, subsequent analysis of the data will be inaccurate if the data is not representative of the rock mass. The rock mass is three-dimensional and Area sampling (which is two-dimensional) and Scanline sampling (which is one-dimensional) cannot therefore yield representative data. The missing dimension(s) can be included by sampling on an orthogonal basis or by orientating scanlines at high angles to the mean orientation of each discontinuity set, assuming that discontinuity sets have already been established.

It is important to emphasise the need for quality and relevance in the data; blanket coverage of all aspects is rarely the answer. In this respect it is often better to rely on a small number of well-chosen observations and measurements than to attempt to obtain a large, statistically significant number by systematic collection. Sheer quantity may actually hinder evaluation. Expertise and experience are therefore extremely important attributes for those carrying out data collection.

![Figure 3.4 Complete Data Collection. In this example all discontinuities (34) in the rock exposure are measured and recorded.](image-url)
Figure 3.5 Area Data Collection. In this example all discontinuities (13) in the selected area are measured and recorded.

Figure 3.6 Scanline Data Collection. In this example all discontinuities (5) intersecting the horizontal scanline are measured and recorded.
Figure 3.7 Judicious Data Collection. In this example discontinuities (3) representing the three discontinuity sets in the rock exposure are measured and recorded.

Where discontinuities are very persistent (generally over 10m), then their exact position relative to the design face can be of vital importance, since they can exert a dominant control over the final stability of the face. Their position must therefore be recorded by accurate field mapping.

### 3.5.1.3 Structural Domains

Fundamental to success in collecting data is the concept of a structural domain. A structural domain is a rock volume having a relatively uniform discontinuity pattern (Section 4.4.1). Recognition of such domains relies on accurate geological mapping (Matheson, 1985).

In most situations, data cannot be obtained directly from the rock mass in the position of the design slope and has to be inferred from surface rock exposures in the vicinity. The concept of a structural domain is therefore of fundamental importance to success in extrapolating and interpolating field data. The technique used for sampling must be such that data are not incorrectly aggregated and the discontinuity patterns confused. The aim is to identify and characterise a domain which is representative of the design area and from which data can be extrapolated (Figure 3.8). It is vital that only data representing the relevant structural domain is used to design slopes within that domain.

The concept of a structural domain is fundamental to success in establishing discontinuity patterns, and if the patterns are not recognised, then the failure potential cannot be assessed. Data should not therefore be aggregated such that patterns are confused. Grouping of the data according to local geological or geographical domains is strongly recommended. If preliminary analysis shows that individual groups have similar patterns, they can then be aggregated to simplify presentation and later evaluation.
3.6 Ground Investigation

3.6.1 Trial Pitting and Trenching

Trial pitting and trenching are excellent and economic methods of establishing the position and nature of rock head and to collect discontinuity data. They can also give valuable information on the stability of any superficial materials and their excavatability. These methods should therefore be considered before other less accurate and more expensive techniques are selected. In site investigations for roads, pits and trenches are generally shallow (2m to 4m deep), and excavated by hand or by hydraulic excavators without the need for specialist equipment. Guidance is given in BS 5930 (1999). Attention is drawn to the safety aspects of working in such trenches and the precautions necessary to ensure safety (Weltman & Head, 1983). Excavation is usually limited to levels above the water table, below this pumping may be necessary. The data collected should relate to the actual position and nature of rock head in the pit or trench.
Figure 3.8 The use of Field Data According to Structural Domains. A road cutting is proposed in an area with three rock exposures. Different underlying geological situations are shown to illustrate how the data collected from these exposures is used to evaluate the geological structure and to design rock slopes. It is vital that only data representing the relevant structural domain is used to design slopes within that domain.
3.6.2 Probing and Auguring

Probing and auguring are cheap and quick methods of determining the depth to hard strata where the overburden is thin and relatively weak. Specialist equipment is required but this is easy to use and readily portable. Probing involves pushing or hammering a variety of rod-like tools into the ground. Auguring involves imparting a rotary motion to a screw head to achieve penetration. These methods are limited to soft ground (e.g. peat, soft alluvium, etc) and shallow depths, up to approximately 5m. Penetration stops when strong material is encountered. However, the strong material may or may not be rock head and intelligent interpretation of results is therefore necessary. Misleading results may be obtained where boulders or cobbles are present, which might be mistaken for rock head.

In recent years, a number of proprietary, portable mechanical probing systems have become available which use "dynamic probing" (i.e. a small scale powered pushing/rotating rig system) to both sample and probe drift deposits to the rock head level. Depth limitations are again usually of the order of approximately 5m, and up to 15m depth can be reached in optimum conditions. However rock head type is difficult to definitively establish.

These systems are most suited to soft ground conditions where there is a sharp transition at rock head. Their main use is in assisting interpolation of depth data from established and proved rock head levels, i.e. between exposures, trial pits or boreholes.

3.6.3 Boreholes

Boreholes are the commonest means of determining the depth to rock head in site investigations for roads. The methods available range from Shell and Auger Percussive Boring and Rotary Mechanical Augering in weak and superficial material, to Rotary Drilling, either open hole or coring, in stronger material such as rock. The main problem again lies in the interpretation of when rock head has been reached and not boulders or rafts situated above true rock head. The working solution adopted is that bedrock can generally, but not universally, be accepted after 3-5 m of continuous drilling in rock.

3.6.3.1 Shell and Auger Boring

Shell and auger boring involves the repeated dropping of a weighted auger that gradually advances the borehole. Boreholes are usually between 150 mm to 300 mm in diameter and a maximum depth of approximately 60m can be reached. Samples of the fragmented material being bored are recovered regularly by raising the shell. These, and significantly lower penetration rates, can be used to indicate when strong strata (such as bedrock) has been reached. The technique was however developed for soil and overburden boring and is not suited to penetrating rock. Weak rocks can be bored using a special chisel tool but this can be time-consuming and expensive. Proving rock head can therefore be difficult.

Simple hydraulic attachments can be added to shell and auger rigs to allow rotary coring for a short distance to prove bedrock. Although the attachments are light enough to be winched across poor ground, they require special skills to operate effectively.

3.6.3.2 Rotary Drilling

Rotary drilling is traditionally the most effective technique for drilling in rock and can also be used in hard granular or cohesive soils. Drilling is accomplished by
transmitting rotation and pressure to a drill bit at the end of a string of drill rods. Depth is attained by adding further drill rods to the string. The bit is cooled and the cuttings flushed from the cutting area and brought to the surface by water, air or a mixture of both, injected along the hollow drill rod. Drill hole diameters vary from 75mm to 120mm depending on the application and the depth. Where unconsolidated or fragmented materials are being drilled, then it is common practice to case the drill hole with metal tubing to prevent collapse. The technique can be used to form very deep holes, easily within the range required of normal excavations for roads.

Various methods of rotary drilling are possible using a number of systems and drill types. Two methods are in common use in strong rock - diamond core and open hole drilling. Attention is drawn to earlier statements regarding the need to penetrate rock over 3-5m to be sure that true bedrock has been encountered.

Diamond core drilling uses a water-flushed diamond bit to obtain a core. Recovered core or core fragments are obtained by raising the drilling barrel and extracting the rock. These can be accurately logged and also the level of rock head reliably established. For site investigation for roads, much of the rotary core drilling is in weathered or fragmented rocks and considerable skill and expertise is required in order to recover core of satisfactory quality (Wakeling, 1972). Conventional, double or triple tube core barrels fitted with diamond or tungsten tipped core barrels (BS 5930:1999), may be required to obtain good core recovery. Experience and care needs to be taken when choosing the type of bit, amount of flush, speed of rotation, and bit pressure (BS 4019:1971).

Open hole drilling is occasionally used in site investigations. In open hole drilling, a hole is drilled but no coring takes place. Most open holes are drilled using down-the-hole hammers (DTH). This has the percussive mechanism located at the drill head, the rock is fragmented by the drill bit, and the chippings are air or water-flushed to the surface. An idea of the strata being drilled is obtained from the rate of penetration, and the nature and colour of the chippings and dust flushed to the surface. In ground that is not self-supporting, casing is needed and this can add considerable cost to the drilling. A suitably experienced engineering geologist or geotechnical engineer needs to be in attendance during drilling, in order to retrieve maximum information on the ground conditions (Joyce 1982). Open hole drilling is not a very accurate or definitive way of identifying rock head, as boulders can often give misleading results, and is best used where the transition to rock head is abrupt and unambiguous.

Down-the-hole Closed Circuit Television (CCTV) can be used to directly view rock head in open drill holes (see Section 3.6.4).

3.6.4 Geophysical Techniques

For all geophysical ground investigation techniques, a specialist organisation should be employed to undertake the geophysical surveys and interpret the results. It is also essential that the specialist organisation is involved with the selection of the technique that will be most suited for the objectives of the ground investigation. If concerns exist over the impartiality of the specialist organisation, then an independent geophysical advisor should be employed to advise on suitable techniques and to design and supervise the surveys (BS 5930:1999).

There are a large number of geophysical techniques which can be employed in ground investigation, and these are reviewed in BS 5930 (1999). However, only a limited number are routinely employed in ground investigation for road rock slope excavation schemes, and these are described here. These techniques can be generally divided into surface and borehole methods.
With both surface and borehole techniques it is essential that they are used with adequate ground control, i.e. real data from trial pits, probing and auguring, boreholes or field mapping.

3.6.4.1 Surface Geophysical Techniques

These techniques are rapid and can cover large areas quickly. They require little or no site preparation. As a result, these techniques can be utilised at the main or preliminary investigation stage (Section 3.4.3), and possibly even at the reconnaissance survey stage (Section 3.4.1). The main techniques and methods for surface geophysics, for use in site investigations for rock excavations are given in Table 3.2. BS 5930 (1999) should be referred to for more detailed descriptions of the methods and their suitability to particular objectives. A brief summary of surface geophysical techniques and their applications, including Ground Probing Radar, is given by Waltham (1994).

3.6.4.2 Borehole Geophysical Techniques

These techniques obviously require suitable boreholes for their use. As a result they are only usually used during the main site investigation (Section 3.4.3).

Borehole CCTV (Closed Circuit Television) allows the inspection and recording of the walls of open boreholes BS 5930 (1999). The CCTV system comprises a sonde which is winched down the borehole. The sonde contains a low-light sensitive camera and lighting heads. The camera may be either directional or the sonde may contain mirrors to allow a radial view of the borehole wall. A surface control unit is used to control the depth, focusing, light intensity and rotation. The CCTV system can be used in boreholes down to 100mm diameter. CCTV can be used to determine the dip and dip direction, aperture and spacing of rock mass discontinuities. It can also be used to determine the general lithology, borehole condition and any water ingress (above water level).

The Formation Micro Imager (FMI) is a micro resistivity tool and consists of arrays of microelectrodes mounted within several measurement pads, which are pressed against the borehole walls. Each measurement pad contains a current emitting electrode and potential measuring electrodes. Hence, each pad samples the resistivity of the borehole wall. The micro resistivity traces are processed to produce a high resolution image of two strips of the borehole wall. The electrical imaging penetrates a few centimetres into the borehole wall and produces an image which closely resembles recovered core, and shows subsurface fractures BS 5930 (1999). The FMI can be used, even if borehole walls are smothered in rock flour.

**Table 3.2 Surface Geophysical Methods in Ground Investigation for Rock Excavations (after BS 5930:1999 and Waltham 1994). GPR = Ground Probing Radar.**

<table>
<thead>
<tr>
<th>Feature</th>
<th>Example</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geological Features</td>
<td>Stratigraphical</td>
<td></td>
</tr>
<tr>
<td>Sediments over bedrock:</td>
<td>(i) Sands &amp; gravel over bedrock</td>
<td>Seismic Refraction, GPR (max. 20m in dry sand)</td>
</tr>
<tr>
<td></td>
<td>(ii) Clay over bedrock</td>
<td>Resistivity or Seismic Refraction, GPR (max. 3m in wet clay)</td>
</tr>
<tr>
<td>Erosional</td>
<td>(i) Buried channel</td>
<td>Seismic Refraction, Resistivity (for feature wider than depth of cover), GPR (shallow feature)</td>
</tr>
<tr>
<td></td>
<td>(ii) Buried karstic surface</td>
<td>Resistivity Contouring, GPR (voids and filled sinkholes)</td>
</tr>
</tbody>
</table>
The borehole Acoustic Televiewer (ATV) uses ultrasound to scan the borehole wall and records the amplitude of the reflected signal and the transit delay time. The amplitude and delay time can then be related to the borehole diameter and 'reflectivity' of the borehole wall. Discontinuities can be surveyed because of the continuous orientation information and high resolution. Generally the probe can be used in boreholes down to 45° below the horizontal. The data can be automatically manipulated in several formats, including a full 360° image of the 'unfolded' borehole wall and stereographic plots. In the unfolded image of the borehole wall, discontinuities appear as sinusoidal wave forms, unless they are perfectly perpendicular to the borehole orientation. An advantage of the ATV is that it can be used in boreholes that contain dirty water that would negate the use of CCTV or the optical televiewer. However, because of the nature of the acoustic probe it does require a fluid filled borehole (water or mud).

The borehole Optical Televiewer (OTV), like the ATV, produces a full 360° image of the borehole wall. The OTV uses a digital camera and a hyperbolic mirror to record the image of the borehole wall, recording scanned rings of pixels every 0.5mm to build into a continuous image. The image is displayed in 'real time' and recorded for later analysis. For the analysis of discontinuities, stereographic plots, and fracture densities and spacings can be produced. The OTV can be used instead of the ATV if there is no fluid in the borehole or if a higher resolution is required. However, because the OTV image is based on an optical view of the borehole wall, it is not suitable for boreholes containing dirty water or mud, unless the hole can be flushed clean.
4  Rock Slope design

4.1  Introduction to Rock Slope Design

Rock slopes must be designed to be sustainable. A truly sustainable rock slope will require minimal maintenance within its design life. The design of rock slope excavations for roads essentially comprises two elements; the design of the rock faces themselves, and the design of rock traps to contain any rockfall material. It is these two elements taken together that is important to ensure that the level of risk from rockfall to the finished road and road user is below an accepted level. Risk to roads and road users from rockfall is discussed in the Guide to Good Practice for Road Rock Slope Remedial and Maintenance Works McMillan et al, (2000).

The design of the rock faces, along with the natural and induced instability within the rock mass, control the stability of rock slopes. The design of a rock trap to remove risk to the road and road user must also be considered. It should be assumed that there will be rockfall from all rock slopes.

Comprehensive rock slope stability assessment considers the following elements:
- Rock slope geometry
- Groundwater conditions
- Discontinuity geometries
- Discontinuity properties
- Potential failure mechanisms
- Potential failure positions

4.2  Considerations in Slope Design

Instability in rock slopes is unacceptable if it presents a significant risk to the road or road users.

The geotechnical characteristics of the rock mass (natural instability) and the damage that can be introduced by some methods of excavation (induced instability) are of prime importance. Environmental considerations for new rock slopes have a strong influence on the design and costs. Such considerations are extremely difficult to define, specify and price. An acceptable design will be a compromise between minimising instability, providing an adequate rock trap and satisfying environmental considerations. It is important to ensure that balanced consideration is given to all requirements.

4.2.1  Factors in Rock Excavation

If stability considerations are disregarded, there are three simple requirements for the design, relating to the position of the slope and to risks associated with different slope profiles (Matheson 1995):

4.2.1.1  The optimum volume of rock for excavation (Figure 4.1)

The volume of rock excavated is often minimised in an attempt to keep down the price of the road construction. However, the cost of rock excavation itself is relatively low, and may only be a few percent of the total contract value. Other
factors associated with excavation may increase the total costs. For example; purchasing extra land to increase the size of the excavation, disposing of surplus excavated material, or if the excavated rock slope is unstable and remedial works are required.

**Figure 4.1 The optimum volume of rock for excavation.**

An environmentally acceptable slope, may be designed at a relatively low angle, to blend with natural rock outcrops in the area and for establishing vegetation. This will increase the volume of rock that needs to be excavated. Other requirements in the road design may also require additional rock to be excavated. For instance, high standards for horizontal and vertical alignment for sightlines may require the formation of wide verges. The cut and fill balance of rock material is also an important consideration. A negative balance will require the import of additional material and a positive balance may produce environmental problems in disposing of surplus material (i.e. material to landfill).

4.2.1.2 **Lifts and Benches (Figure 4.2)**

Considerable damage can be caused to the rock mass by some methods of blasting (see Chapter 5). Even presplit blasting is ineffective over the stemmed length at the top of each drill hole. Damage from fragmentation blasting can therefore still penetrate the host rock in this area. Using a top-down form of excavation this means that narrow benches are seldom formed successfully, and become bevelled ledges. If rock falling from the face above strikes such a ledge it can be projected outwards towards the carriageway, and can pose a severe risk to road users.
Guidance on the design of rock traps is given in Section 4.5. However, benches should be at least 4m wide, and should be able to contain any rock debris from faces or slopes above. The action of benches is therefore as on-the-face rock traps, and they should be designed as such.

4.2.1.3 Ravelling failure and the design angle of the slope (Figure 4.3)

The risk to road users from instability in rock faces can vary according to the slope angle. Generally, on vertical slopes 'ravelling' rock blocks fall freely from the face and land on the verge. Their trajectory is such that there is little tendency for them to move away from the face. In this case, a steep slope reduces the risk from rockfall to the road user, but a sufficient rock trap verge is required. As the angle of the face decreases, blocks tend to roll rather than fall and to gain angular momentum. This causes the blocks to move towards the carriageway and, when the momentum is sufficient, to roll out onto the pavement where they can pose a severe hazard to road users. A similar effect is produced on slopes with rough profiles, and/or badly-formed or narrow benches (berms).

However, producing such a steep face has other implications. The stability of the rock slope is a prime concern. A steep slope may also not be appropriate for environmental reasons.
Excavation to Rock Head

There is no universally accepted definition of rock head. Rock head can be defined simplistically as: the interface between soil and in situ sound rock.

In many cases it may be difficult to define what is regarded as soil and what is regarded as rock. The soil-rock interface may be a sharp transition, a mixture of material types (e.g. corestones in a matrix of weathered rock material) or a gradual transition (e.g. a penetratively weathered profile).

Rock head could alternatively be regarded as: the limit between material that can be directly excavated using standard machines (e.g. back-actors) and material that needs to be broken before excavation (e.g. ripping or blasting). Reference could then be made to the guidance for excavatability (Figure 5.1) for material in the transition between strong soil and weak rock. Techniques for locating rock head have been outlined in Chapter 3. In a rock slope excavation, there may be a contrast between the long-term stability of rock and that of any overlying superficial material.

Superficial materials can be stabilised by either designing the slope at their natural angle of repose, or by reinforcement or retaining measures. Effective stresses, water flow (there is often water seepage at rockhead) and vegetation all play an important part in stability. Studies of the stability of superficial slopes have been carried out by Perry (1989).

It is unlikely that the stable angle of superficial material will be as steep as that of the underlying rock and erosion could result in failure of the material down the rock face. A bench should be designed and excavated at rock head, separating the two materials (Figure 4.4). The bench width will be dependent on the stability of the superficial material. Forming a bench by cutting into the superficial material may over-steepen it. Support, reinforcement or erosion protection may then be required. The rock head bench may be used to provide a stable drilling platform for any drilling and blasting operation required to form the rock face.

Figure 4.4 Bench at rockhead - with a sharp transition between the rock (dolerite) and overlying superficial (glacial) deposits.
4.2.3 Safety Fences and Barriers

TD 19/85 (DoT 1985) states that safety fences or barriers are required on all new or existing trunk roads, "At substantial obstructions such as...steep sided (1 in 2 or steeper) rock face cuttings...closer than 4.5m to the edge of the running carriageway on roads with speed limits above 50 mph...."

The construction of safety fences or barriers below rock excavations is environmentally undesirable, as they are visually intrusive (see Chapter 6, Environmental Considerations). They are also unsustainable as they require maintenance and have a lower design life than the road. Safety fences or barriers are generally expensive in the long term compared to the cost of buying land and excavating rock.

Therefore, on any new trunk road, if no safety fences or barriers are used, there should be a distance of greater than 4.5m between any rock slope steeper than 1 in 2 (~63°) and the running carriageway. This 4.5m verge may provide a suitable rock trap for many rock slopes. However, this suitability needs to be determined at the outset of design.

4.3 Approaches to Rock Slope Design

In the past, two approaches were used to design rock slopes in strong rock in the UK:

- A traditional approach, that drew on the skills and experience accumulated over many years.
- A standardised design approach, developed to utilise the power of the high explosives, as they became readily available.
- The modern approach, outlined in the present guide should replace both the traditional and the standardised techniques.

4.3.1 The Traditional Approach

Traditional, labour-intensive methods used low-energy sources to achieve their objectives and involved minimum rock removal. Rock slopes were not designed in advance, but were the steepest that could be attained during construction. In most instances this depended on the geological and geotechnical conditions of the host rock. Traditional techniques, although time consuming and laborious, were relatively successful. The techniques tended to minimise induced instability and therefore produced relatively stable slopes, due to the low-energy techniques employed. Generally, road rock slopes were small because there was little requirement for horizontal and vertical alignment and it was easier to bypass an area, rather than to excavate a slope. Most of these rock slopes, where they have not been replaced, have survived in a generally stable form to the present day. This is true of canal, railway and early road excavations.

4.3.2 The Standard Design Approach

In the early part of the 20th century in the UK, dynamite and other nitro-glycerine based explosives became readily available and economical to use. Increasingly efficient and effective rock drills enabled explosive charges to be placed quickly and accurately within a rock mass. New techniques of detonation and new explosives based on ANFO (Ammonium Nitrate/Fuel Oil) were also developed. These explosives were relatively cheap to manufacture and easy to
use. As a result, techniques of excavating larger volumes of rock at lower cost rapidly evolved.

The new approach was dominated by the economics of blasting in bulk, with high energy explosives. In general, the larger the volume of rock blasted, then the cheaper the unit price. Bulk blasting was primarily intended to reduce the rock mass to a fragment size easily handled by mechanised plant. Little attention was paid to the instability induced in the final rock slope. Rock slope designs with slopes of 3 in 1 or 4 in 1 were widely adopted, regardless of the local geology (Figure 4.5). This approach was generally used on road rock slopes excavated between the 1930s and the 1970s, and is still in limited use today.

The final faces of rock slopes produced by the standard design technique, particularly where it is associated with bulk blasting, are characteristically unstable and affected by extensive induced instability due to blast damage. If the stability of such rock slopes is critical (i.e. if it presents a significant risk to the road), remedial works will be required after excavation and will require significant maintenance during their design lives.

![Figure 4.5 Standardised Design Approach.](image)

4.4 **Slope Design Procedures**

This guide deals with the excavation of rock slopes in rock masses that are strong (i.e. moderately strong or stronger). Such rock masses are largely controlled by discontinuities. However, if a slope exceeds the height where the stress due to the “column” of rock material exceeds the strength of the rock, then failure of the intact rock material can occur. Failure would follow the path of least resistance and hence it would most likely involve shearing of intact rock bridges between discontinuities. This is highly dependent on the relative strengths of the rock material and discontinuities. Slopes in rock masses that are weak (i.e. Moderately Weak or weaker) are usually controlled by a combination of discontinuities and failure of the rock material itself. Hence such slopes require analysis of both the discontinuities and the rock mass itself (by soil mechanics techniques).

Figures 4.6 and 4.7 show the general design procedures for rock slope design.
4.4.1 Data Collation and Analysis

The geotechnical data collected during the site investigation (Section 3.5) requires careful analysis before it can be used for the design of rock slopes. The first stage of the process is to determine the structural domains present. Structural domains should, in areas of sufficient outcrop, be identified during the field mapping part of the site investigation, as it is easier to identify domains in the field than from ground investigation data.

In areas of limited outcrop it will be necessary to try to establish the structural domains from the ground investigation data. This can be achieved by forming a geological model of the site from the site investigation data. To facilitate this, the discontinuity data and the location from which the data was obtained should be entered into a stereographic projection package. This allows the geological structure at different locations to be determined and the geological model formed. Once a representative geological model of the site has been created, the structural domains can be identified.

Once the structural domains have been identified all the site investigation data should be assigned to the appropriate domains.

4.4.2 Determination of Potential Failure Mechanisms

The determination of potential failure mechanisms must be undertaken for each structural domain individually. The first stage is to plot all the discontinuity data for the domain on a stereographic projection. This can be done manually (Priest 1985), but is more usually done using the various software packages available (e.g. RocScience “Dips”, TRL “Rockstab”). Generally, the lower-hemisphere, equatorial, equal-area method of stereographic projection is used for assessing potential failure mechanisms. Discontinuities are plotted as poles to the planes and great circles.
The data should be plotted on a stereographic projection and the individual discontinuity sets identified and compared with the discontinuity sets identified during any field mapping. If the stereographic projection of the discontinuity data is contoured (Priest 1985) than clusters of discontinuity poles will be found. These can generally be interpreted as discontinuity data belonging to individual discontinuity sets. However, caution should be exercised so that the contouring process does not obscure important / significant discontinuity sets which due to spatial distribution are only sampled infrequently. This problem may be avoided by applying some weighting factor to discontinuity data (Matheson 1983).
<table>
<thead>
<tr>
<th>Procedure</th>
<th>Product or Output</th>
<th>Activities</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Data Collection</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>All Data</strong></td>
<td>Site Investigation Report</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Data Set for Each Domain</td>
<td></td>
</tr>
<tr>
<td><strong>Data Collation</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Data Analysis</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Structural Domain Data Separately</strong></td>
<td>Stereographic Projections of Discontinuity Data</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Stereographic Plot of Discontinuity Data for each Domain Data Set</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Identify Controlling Discontinuity Sets</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Discontinuity Set &amp; Rock Mass Parameters for Design</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Determination of Discontinuity Set &amp; Rock Mass Properties</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Consider Design Azimuth and Dip of Slope</td>
<td></td>
</tr>
<tr>
<td><strong>Determination of Potential Failure Mechanisms</strong></td>
<td>Potential Failure Mechanisms on Slope</td>
<td></td>
</tr>
<tr>
<td><strong>Each Slope Separately</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Assessment of Failure Probability</strong></td>
<td>Probability of Failure of Potential Failure Mechanisms on Slope</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Probabilistic Assessment for Potential Wedge, Plane &amp; Toppling</td>
<td></td>
</tr>
</tbody>
</table>

**Figure 4.7 Rock slope design procedure - the overall approach.**

It is critically important that any important individual or rare discontinuities are identified and considered during any slope design. These “rogue” discontinuities often control the stability of rock masses. Such rogue discontinuities often have large persistence and principal spacings.

Once all the discontinuity data has been assigned to the appropriate discontinuity sets then work can begin on determining the potential failure mechanisms which are kinematically feasible, given a particular slope geometry.
Various methods can be used to identify the potential failure mechanisms for a given slope:

Stereographic Projection of discontinuities and rock slope orientation to allow the block delimiting discontinuity sets to be determined and the angular relationships of the block delimiting discontinuities and the rock slope orientation to be determined. This allows the use of the graphical methods described in Figures 4.8 and 4.9 (Matheson 1983, Hoek & Bray 1983, Priest 1985). The published overlays for toppling failure (Matheson, 1983) do not adequately cater for the range of potential toppling geometries that can occur on a rock slope. This is particularly the case where the rock slope profile is uneven. A more complete definition of toppling failure criteria is given in Appendix 1.

Graphical or Vector Methods where blocks delimited by discontinuities can be represented by polygons and their kinematic stability assessed (Goodman & Shi 1985). There are numerous computer programs that can be used for this type of analysis (for example “Slide” and “Swedge” from RocScience).

Numerical Analysis such as finite element, boundary element or distinct element can be used to assess failure. In particular distinct element methods (for example UDEC 2D and 3D) can be used to model the blocks within a rock mass.

All these methods can be used to determine the stability of discontinuity delimited blocks within a rock mass.
Figure 4.8 Stereographic assessment of Potential Failure Mechanisms (a) Flowchart, (b) Function and Limitations. * Note: the published overlays for toppling failure (Matheson, 1983) do not adequately cater for the range of potential toppling geometries that can occur on a rock slope. This is particularly the case where the rock slope profile is uneven. A more complete definition of toppling failure criteria is given in Appendix 1.

Figure 4.9 Preliminary stability assessment using stereographic method for wedge failure (after Matheson 1983).
4.4.3 Assessment of Failure Probability

Once potential failure mechanisms have been identified for a particular slope then an assessment of the probability of failure of each potential failure mass should be made. This will require an assessment of the sensitivity of the failure to changes in the properties of its controlling factors (e.g. water pressure, discontinuity friction angle, and discontinuity persistence). Sensitivity analysis is dealt with in Hoek (1999).

The assessment of the probability of failure can vary according to the scale and consequences of failure. At one extreme the assessment may be based on professional judgement of the likelihood of failure within a given timescale. Whilst at the other end of the spectrum a full sensitivity analysis of each failure may be appropriate.

4.5 Rock Trap Design

The objective of a rock trap is to ensure that any rockfall material is contained within a safe zone and, therefore, does not present a significant risk to the road. Rock traps vary from verges and ditches to complex systems of flexible and rigid barriers. However, this Guide only deals with rock traps “designed-in” at the slope excavation stage, i.e. rock trap verges, ditches and berms (benches). For the design of rock traps as remedial works (i.e. as rock trap fences and barriers), reference should be made to the Rock Engineering Guide to Good Practice for Road Rock Slope Remedial and Maintenance Works (McMillan et al., 2000). It is good practice in producing sustainable rock slopes to incorporate designed-in rock traps in every rock slope design. They are the simplest, most economical and effective way to ensure minimum rockfall risk to the road and road users.

Rock traps can be constructed to intercept falling material either on or at the base of the rock slope. Rock trap verges and ditches are constructed at the base of the slope. Rock trap berms are constructed on the rock slope.

4.5.1 Types of Rock Traps

There are three types of designed-in rock trap: verges, ditches and berms.

Verges - act as rock traps by dissipating the energy of falling blocks so that they do not reach the road and by providing adequate room to hold the volume of failed material (Figure 4.10). The verge must be wide enough so that all falling blocks stop moving on the verge and, therefore, are often unsuitable for use as rock traps because of the excessive land-take required to provide for stopping distance. Generally, verges can be used as rock traps only when the verge is wide; the slope is of limited height, is steep (>70°) and has a regular profile. Verges should regularly be cleared of any rockfall debris (as for ditches). The width of verge required to form an effective rock trap is dependent upon the slope height, angle and profile and on the shape of the ravelling failures occurring on the slope. The required verge width can be reduced if it is vegetated, or has a thick layer of fine gravel.
Ditches - act effectively as rock traps because they provide a physical barrier to the horizontal movement of rock material (Figure 4.11). The effectiveness of the ditch will decrease if rockfall material is allowed to collect in it. If a failure fills the ditch, the rock debris will act as a hard surface for any further failures to impact...
on, resulting in an increased likelihood of the failure reaching the road. As a consequence, a regular programme of clearance of accumulated material should be undertaken as part of the post-construction management strategy.

**Figure 4.12 Effect of ditch at the bottom of a slope on run-out distance (from Azzoni and de Freitas 1995).**

Ditches have a significant effect on risk reduction, according to the field testing carried out by Azzoni and de Freitas (1995), as shown in Figure 4.12. The testing was performed on a slope approximately 45m high and 60m long, and with an overall slope angle of about 50° (30°-60°). Two tests were carried out, one without a ditch and one with a ditch 1.5m wide and 0.9m deep. Approximately 85% of falling blocks were stopped by the ditch. Without the ditch, almost 80% of the blocks stopped in the range of 10 –25m from the toe of the slope.

Rock trap ditches are usually positioned at the base of slopes, but they can be formed on a berm or just above the crest of a slope (to catch superficial failures). The required ditch dimensions (width and depth) are dependent on a variety of factors, including slope height, angle, profile, block size and block shape.

**Berms** - rock traps are constructed on the slope to intercept falling material before it reaches the base of the slope. The design of berms as rock traps is similar to that for traps at the base of the slope. Berms act as rock traps if the rock face is steep and they are carefully constructed. Berms must be wide (at least 4.0m after excavation) and horizontal or inclined towards the slope. The effectiveness of a berm is increased by providing a covering of an impact absorbing material (e.g. vegetated peat). For road rock slopes, in many cases, berms are simply a “by-product” of the drill and blast technique, i.e. each berm is effectively a drilling platform. The use of berms to aid slope stability originates in the open-pit extractive industry, where they were used to limit the extent of large failures. Poorly constructed berms (especially with outward dipping surfaces) can act as “launch ramps” for falling material, projecting it out from the face and towards the road.

### 4.5.2 The Design of Rock Traps

The run-out distance of falling material, the bounce height of rock blocks and the volume of the failure material are the primary factors to be considered in the design of rock traps.

TRL (McMillan, 1994) have made a comparison between three rock trap design charts, from Ritchie (1963), Mak and Blomfield (1986) and Fookes and Weltman (1989). The comparison revealed the considerable variation in rock trap design, derived for the same slope profile, using the different design charts (Table 4.1 and Figure 4.13).

It is recommended that Ritchie’s data (see Table 4.2) should be used for irregular slopes (for example bulk-blasted) and Mak and Blomfield’s data for regular slopes (for example pre-split). It should be noted however that the trial slope used by Mak and Blomfield had a vertical height of only 12m. The charts are only for relatively small scale “rockfall”, and the possibility of larger failures has not been considered. Determination of rock trap dimensions for larger, discrete masses requires assessment of the overall failure volume and calculation of the energy of blocks within the failure mass.

**Table 4.1 Comparison of rock trap design chart recommendations.**

<table>
<thead>
<tr>
<th>Rock slope details</th>
<th>Rock trap design chart recommendations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Height (m)</td>
<td>Angle (deg.)</td>
</tr>
<tr>
<td>5</td>
<td>60</td>
</tr>
<tr>
<td>5</td>
<td>80</td>
</tr>
<tr>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>10</td>
<td>80</td>
</tr>
<tr>
<td>12</td>
<td>60</td>
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<tr>
<td>12</td>
<td>80</td>
</tr>
<tr>
<td>20</td>
<td>60</td>
</tr>
<tr>
<td>20</td>
<td>80</td>
</tr>
</tbody>
</table>

* For notation of W, H_d and H_f, see Figure 4.13
The trap design chart produced by Fookes and Sweeney (1976), and based on Ritchie (1963), generally gives a reasonable value for depth but high values for width. Whiteside (1986) revised the Fookes and Sweeney chart (see Figure 4.14) to give relatively reliable guidelines, for low to intermediate height slopes (0 to ~35m). Azzoni and de Freitas (1995) carried out field tests and suggested that Whiteside’s chart is generally accurate but is conservative for the case when the main block motion at the bottom of the slope is rolling combined with small bounces.

Computer based rockfall modelling can aid the design of rock traps (see the Rock Engineering Guide to Good Practice for Road Rock Slope Remedial and Maintenance Works: McMillan et al, 2000). Computer models now deal (to some extent) with irregular block shapes, bounces and movement in more than one plane (Azzoni et al, 1995). The understanding of energy absorption and conservation in relation to rock materials has also improved (Kane et al, 1993). The use of computer modelling can improve the cost-effectiveness of rock trap design, by tailoring it to the specific requirements of each situation. As with all computer modelling, the quality of the input data directly affects the results. The input data must be obtained from investigations at the site, i.e. slope height, angle and roughness, and range of block sizes, etc. The coefficients of restitution for the surfaces that the blocks impact on are particularly important, with respect to how much energy the blocks lose with each impact on the slope.
### Table 4.2 Design criteria for rock trap ditches to catch falling rock (Based on Ritchie 1963).

<table>
<thead>
<tr>
<th>Rock slope angle</th>
<th>Height $H$ (m)</th>
<th>Fallout area width $W$ (m)</th>
<th>Ditch depth $H_d$ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>5-10</td>
<td>10-20</td>
<td>&gt;20</td>
</tr>
<tr>
<td>Near vertical ($90^\circ$)</td>
<td>3.1</td>
<td>4.6</td>
<td>6.1</td>
</tr>
<tr>
<td>0.25-0.3:1 ($73.3-76^\circ$)</td>
<td>5-10</td>
<td>10-20</td>
<td>&gt;30</td>
</tr>
<tr>
<td></td>
<td>3.1</td>
<td>4.6</td>
<td>6.1</td>
</tr>
<tr>
<td>0.5:1 ($63.3^\circ$)</td>
<td>5-10</td>
<td>10-20</td>
<td>&gt;30</td>
</tr>
<tr>
<td></td>
<td>3.1</td>
<td>4.6</td>
<td>6.1</td>
</tr>
<tr>
<td>0.75:1 ($53.3^\circ$)</td>
<td>0-10</td>
<td>10-20</td>
<td>&gt;20</td>
</tr>
<tr>
<td></td>
<td>3.1</td>
<td>4.6</td>
<td>6.1</td>
</tr>
<tr>
<td>1:1 ($45^\circ$)</td>
<td>0-10</td>
<td>10-20</td>
<td>&gt;20</td>
</tr>
<tr>
<td></td>
<td>3.1</td>
<td>4.6</td>
<td>6.1</td>
</tr>
</tbody>
</table>

*May be 1.2 m if catch fence is used

Rock trap design is still partially empirical due to the "random" nature of block energies and trajectories. Best practice for rock trap design would involve referring to the most relevant published data, computer rock fall modelling (site calibrated using field observations) and field trials (for critical locations).

For slopes that exceed the heights given in the published data, rockfall modelling, rock fall field trials and extrapolation of the published data are techniques that should be used to determine suitable rock trap dimensions. The use of these techniques should only be undertaken by suitably trained and experienced staff, as the design of rock traps will directly affect the risk to the road and road user.
Figure 4.14 Guidelines for the design of rock traps (from Whiteside 1986).
5 Rock slope excavation

5.1 Introduction to Rock Slope Excavation

The end product of a rock slope excavation should be a rock slope, consisting of a rock face and an appropriate rock trap, with a minimal risk to the road and road user from rockfall. This is achieved by a combination of limiting natural and induced instability of the rock slope, and by excavating sufficient material to form a rock trap. The rock trap should be designed with respect to the height, angle, vertical profile and degree of instability present on the slope.

Rock slope excavation can be undertaken by manual, mechanical and/or explosive methods. Manual or mechanical means can usually only be used in weak, fragmented and/or weathered rock masses. Explosives are used to form rock slopes in strong rock masses (Figure 5.1).

![Figure 5.1 General guidance on excavatability using Unconfined Compressive Strength (UCS) and Fracture Spacing Index (after Pettifer & Fookes 1994).](image)

The excavation method should be selected so as to form an acceptable final slope and to fragment the rock mass and to enable efficient excavation, by suitable plant, of the fragmented material. There may also be a requirement to produce fill material of a specified type, for example armour stone, aggregate or road base material. Where the excavated material is used as fill, it is important that the percentage of blocks falling within the specified limits is as high as possible.
5.1.1 Manual Excavation

In modern construction projects, manual methods of rock excavation are only used for very small quantities of material. In the past it was the main means of excavation, and was used extensively in the construction of early road, canal and railway slopes. Manual excavation involved loosening, wedging and removal of individual natural blocks using bars and chisels. Areas of strong rock, which could not be excavated by hand, were avoided. The minimum volume of rock possible to create the face was removed.

The energy input of manual excavation is low and therefore minimal induced instability is produced. Manual techniques are extremely labour intensive and costly in today's market and find little or no place in modern road excavation, except in specialist activities such as scaling of potential failures, after the main excavation. Guidance on scaling and rock removal is contained in the Rock Engineering Guide to Good Practice for Road Rock Slope Remedial and Maintenance Works (Harber et al, 2000).

5.1.2 Chemical Excavation

Expansive grouts can be used to break rock material by means of a chemical reaction. The grout is supplied in powdered form and is mixed with water to produce a slurry. The slurry is placed in confined areas of the rock mass, either in drilled holes or voids, and is left to expand. The grout exposed at the surface forms a confining cap. The grout slowly expands, breaking the rock material, usually along pre-existing lines of weakness (i.e. discontinuities) in strong rock.

The grout's expansive reaction is exothermic and depends on the ambient temperature of the environment that it is being used in. A range of grouts are produced for use in different ambient temperatures. The rate of the expansion of the grout is relatively slow, taking place over several hours, and it is impossible to predict when the rock will break. This can have safety implications and steps should be taken to minimise any hazard. Using expansive grout to break rock is labour-intensive and costly and is therefore limited to small, specific areas and where other techniques cannot be used. Pyrotechnic materials (see 5.2.2) can be used as an alternative to expansive grouts and break rock instantaneously.

5.1.3 Mechanical Excavation

Removal of rock using mechanical plant can be both efficient and cost-effective. The controlling factors on mechanical excavation are the strength of the rock, the degree of jointing, the fracturing and disturbance of the rock mass, the attitude of any fabric (e.g. bedding, banding, etc) and the size of the mechanical plant available. Compared to explosive excavation, mechanical excavation will generally produce lower levels of disturbance and vibration and therefore induced instability. In suitable rock material, costs are generally lower than for explosive excavation, but if unforeseen strong or relatively unfractured rock is encountered, and it cannot be mechanically excavated, then significant extra costs may be incurred.

Mechanical excavation includes direct excavation, ripping (Figure 5.2) and pneumatic breaking. The method used depends on the rock mass being excavated. Direct excavation is carried out by mechanical excavators, such as back-actors and wheeled shovels. Ripping is carried out by tracked plant, usually bulldozers, mounted with rippers. Pneumatic breaking or "peckering" is carried out by excavators mounted with pneumatic breaking tools.
Figure 5.2 Excavating Mudstone with a Single Tooth Ripper Mounted on a D9 Bulldozer.

Pneumatic breaking can generally be used to break even the strongest and least fractured rock. For large volumes of rock material it is a relatively inefficient technique and, therefore, may only be cost-effective for limited volumes that cannot be excavated by other mechanical means. However, the plant and personnel for pneumatic breaking may be available on a road construction site, and therefore it may be more cost-effective to utilise it for limited areas, rather than to mobilise more specialist plant (e.g. equipment for drilling and blasting).

There are several methods of measuring the "excavatability" of rock mass material. An indication of the potential rippability of rock can be obtained from the seismic velocity in the rock mass. The technique depends on an apparent reduction in the seismic velocity of the rock mass due to fracturing and weathering. The more fractured and weathered a rock mass, the easier it will be to excavate. Therefore, strong igneous rocks need a lower minimum seismic velocity to be machine excavated than bedded sedimentary rocks (Figure 5.3). Hence, rocks with a high seismic velocity (e.g. strong igneous rocks) will be more difficult to excavate with machines than closely bedded sedimentary rock.
Figure 5.3 Rippability in terms of Seismic Velocity for a D10 Bulldozer.

Other methods for assessing the ease of mechanical excavation include empirical methods. The Excavatability Index (Kirsten 1982) is based on the Rock Quality Designation (RQD) and Unconfined Compressive Strength (Figure 5.4). Excavation can also be based on the Rock Mass Rating (RMR) (Bieniawski 1989). Generally, RMR values of up to 20 can be directly excavated, whilst those up to 60 can be ripped.

![Figure 5.4 Empirical methods for calculating excavatability (Kirsten 1982).](image)

5.2 Blasting

Blasting is employed to create surface and underground excavations in rock. It is used where the rock mass is too strong to be excavated efficiently or cost-effectively by manual means or mechanical plant alone. The aim of blasting is to fragment and heave the rock mass, by exploiting the natural discontinuities within the rock mass, or by the formation of new fractures.

Blasting normally involves:

- Drilling of “shot holes”/“blastholes”
- Loading of explosives (“charging”)
- Firing the explosives
• Excavation of blasted rock material (known as “mucking out“)

The detailed science and practice of blasting is beyond the scope of this publication and reference should be made to Hoek and Bray (1983), Jimeno et al (1995), White and Robinson (1995) and Hustrulid (1999) for specific technical information. A schematic arrangement of blasting components is shown in Figure 5.5.

![Figure 5.5 Schematic arrangement showing blasting components.](image)

### 5.2.1 Definitions

There are a number of technical terms used in blasting and before proceeding with any explanation of the subject it is necessary to give brief definitions, which are expanded upon later, for some of the more fundamental terms:

**Base Charge** - a quantity of explosive which has a higher release of energy than column charge. The high release of energy is required at the base of the hole because rock has to be sheared rather than fail under tension.

**Detonator (Blasting Cap)** - a small (e.g. 45mm long and 7mm diameter) cylindrical aluminium or copper capsule containing a primary charge of an initiating explosive. Used to initiate detonation in a Primer or the Main Column Charge.

**Booster** - a high strength charge to complete initiation process of a Primer, or to create zones of high energy release in the Main Column Charge.

**Column Charge** - the explosive loaded in the majority of a blasthole.

**Deflagration** - an exothermic burning reaction propagated by production of heat, in which the propagation of the reaction is based mainly on thermal conduction and some radiation. The deflagration front advances through the explosive at rates (typically < 1000ms⁻¹) below the sonic velocity of the material. Such a reaction does not require an excess of oxygen above atmospheric levels.

**Detonating Cord** - consists of a core of PETN high explosive (charge weight typically 6, 10, 12, 15, 20, 40, 80, and 100 gm⁻¹) sheathed in a textile fibre braid and a plastic jacket.
**Detonation** - a physico-chemical process that produces a shock wave, in which high temperature and pressure gradients are created. The reaction is initiated and sustained by a supersonic shockwave. The detonation front advances through the explosive at supersonic rates in the range of 1500 to 9000 ms\(^{-1}\).

**Explosion** - a reduction / oxidation reaction which takes place over a very short time. Such a reaction requires an oxidising agent and a fuel.

**Primary Explosive** - high strength, sensitive explosive used to initiate Secondary Explosives e.g. main column charge. Sensitive to detonators and detonating cord, in low and high charge weights. PETN is an example of a primary explosive.

**Primer** - a charge of Primary Explosive used to initiate a Secondary Explosive

**Secondary Explosive** - an explosive in which detonation is initiated by the detonation of a Primary Explosive. They are less sensitive than Primary Explosives, but do more useful work. May or may not be sensitive to Detonators or Detonating Cord.

**Stemming** - the inert material packed into the top of a blast hole, to provide confinement of the gases and to prevent fly rock problems.

**Velocity of Detonation (VOD)** - the rate at which the detonation (physico-chemical reaction) front propagates through the explosive. The VOD of ANFO and emulsions (see 5.2.2) are typically 4,500 ms\(^{-1}\) and 5,500 ms\(^{-1}\) respectively.

### 5.2.2 Explosive Types

The sensitivity to detonation for examples of different types of explosives is shown in Figure 5.6.

#### 5.2.2.1 Low Energy Deflagrating Explosives

These burn relatively slowly (i.e. they Deflagrate), producing mainly gas with relatively little associated vibration. The deflagration reaction can be easily initiated with a flame or spark. Their usefulness is a function of the speed at which they burn and the degree of containment of the gas by the host rock mass. Where containment is poor, for instance in highly fractured rock or rock with open fractures, then they are less effective. Their action loosens and heaves, rather than fragments, the rock mass.
The main advantages or Low Explosives over High Explosives are increased safety, and less onerous regulations concerning their use, transportation and storage. Their main disadvantages are their high cost and the relatively small volume applications in which they can be used at present. They tend to be used in scaling, controlled removal and repainting remedial works, see Guide to Good Practice for Road Rock Slope Remedial and Maintenance Works (McMillan et al, 2000).

**Blackpowder** was used extensively until the early part of the 20th Century. It is no longer used for road rock excavation. This is mainly due to a poor safety record, slow detonation velocity and lack of tolerance of wet conditions.

**Pyrotechnic Materials** are not usually classified as explosive substances. On ignition they burn at a relatively low velocity, generating high volumes of gas. The gases expand rapidly and cause dilation of the rock mass, mainly along pre-existing weaknesses. They come in pre-packed into small diameter polypropylene tubes, which can be linked, and initiated electrically. The use of a pyrotechnic charge in a single drill hole to break a rock block is shown in Figure 5.7.

5.2.2.2 **High Explosives**

High Explosives can be sub-divided into Primary and Secondary Explosives. Primary explosives are high energy and high sensitivity explosives, which are sensitive to Detonators and Detonating Cord. They are used as Primers to initiate Secondary Explosives, or in the manufacture of Detonators and Detonating Cord. They are mainly used to amplify or boost the input energy of Detonators and Detonating Cord to a point where it will effectively detonate Secondary Explosives (Jimeno et al 1995). Typically Primary Explosives produce a high shockwave pressure.
Figure 5.7 Preparation and breaking of a dolerite boulder with a pyrotechnic charge (Back o’Hill, Stirling).

Secondary explosives are less sensitive than Primary Explosives, and may or may not be sensitive to Detonators and Detonating Cord. They are used to do most of the work in a bulk blast, by the production of a shockwave pressure (lower than Primary Explosives), and by producing and maintaining a higher gas pressure during the expansion phase than Primary Explosives.

High explosives can also be classified (Jimeno et al, 1995) into:

- Blasting Agents that are mixtures that, with a few exceptions, do not contain materials classified as explosive e.g. ANFO, Slurries, and Emulsions. They cannot be initiated with a No. 8 detonator.

- Conventional Explosives which are made up of explosive substances e.g. Gelatine Dynamite and Granular Dynamite.

Since the first synthesis of Nitro-Glycerine (NG) by Alfred Nobel in 1876, high explosives have evolved rapidly. Initial types were relatively unstable and the trend in civil engineering has been towards safer and more powerful explosives. They include NG types and those based on Ammonium Nitrate Fuel Oil (ANFO) mixtures.

**Nitro-Glycerine-Based Explosives.** Nitro-Glycerine (NG) is important for commercial explosives. The properties of NG, and the way in which it is mixed with other ingredients, determines the type of explosive produced. NG is available in powder and gelatine form, with the gelatine form being commonly used for large diameter applications.

**Ammonium Nitrate Fuel Oil Mixtures.** Ammonium Nitrate Fuel Oil (ANFO) is manufactured through the action of nitric acid and ammonia. The solid product is formed into small spheres termed prills. Approximately 6% fuel oil is then added and absorbed by the prill to produce the explosive.
ANFO is an important ingredient in the explosives industry, and when mixed with solid fuels it forms fixed explosives. It is commercially available in bulk or packaged form, or can be mixed on-site. Improved performance in wet conditions can be obtained by packaging in plastic containers and/or by forming slurry or emulsions with other compounds.

The VOD of ANFO is generally less than NG explosives and the material has poor water resistance. A priming charge of NG explosive is generally required for detonation, in addition to an electric detonator. Basic ANFO can be enhanced and modified for particular purposes by adding other chemicals or incorporation into other mixtures.

**Slurry Explosives.** Slurry explosives were developed to waterproof, strengthen and sensitise ammonium nitrate explosives. Slurries are composed of inorganic and organic nitrates and light metals, all held in an organic gel. They are sensitised by air incorporated during their manufacture. They are popular because of their water resistance and favourable fume characteristics. Their performance is dependent on their formulation and technical advice should be sought from the manufacturer.

**Emulsion Explosives.** Emulsion explosives contain similar ingredients to slurry explosives, but their structure, properties and performance are quite different. They generally have a higher velocity of detonation (VOD) and better fume characteristics. Gas-sensitised emulsion explosives can suffer from similar pressure effects as slurry explosives. Being a water-in-oil emulsion, they are completely waterproof.

### 5.2.3 Initiation Systems

The actual explosives used in blasting form only part of the blasting system (Figure 5.5). To trigger a blast, an Exploder (Energy Source) is required. The energy from this is then amplified, boosted and transmitted to the explosive in the individual blastholes, to initiate detonation of the explosive. Depending on the purpose of a blast, the initiation of explosives in different blastholes may be required at differing times, for example in Presplit Blasting. To achieve this delays in the transmission of the energy may be required. Examples of methods of charging presplit and bulk blast holes are shown in Figures 5.8 and 5.9.

The transmission of the energy is achieved by trunklines running between blastholes, and down downlines running down individual blastholes (Hustrulid 1999). The amplification and boosting of the energy is achieved with the aid of Detonators, Primers and Boosters.

#### 5.2.3.1 Exploders (Energy Sources)

Various types of exploders are available and they should be carefully chosen, depending on the application. There are three main types of exploder:

**Dynamo.** Dynamo exploders consist of a small twist-handle which when rotated produces an electrical current. Dynamo exploders are used for small rounds of between 1 and 30 shots.

**Capacitor Discharge.** Capacitor discharge exploders consist of a winding handle that is used to charge up a capacitor. A key or button is then used to discharge the capacitor to the initiation system. Capacitor exploders are very common, are extremely reliable, and are used for shots of up to 200 detonations.
Figure 5.8 Schematic Examples of Methods of Charging Presplit Blast Holes (a) 'Omega' Clip (which incorporates detonating cord) holding Packaged Slurry and (b) Heavy Detonating Cord.

Figure 5.9 Schematic example of a method of charging a bulk blast hole, using an electronic blasting machine, connected to a detonating cap, to detonate a nitro-glycerine primer which initiates a packaged slurry booster, which then initiates the main ANFO column charge.

**Electronic Blasting Machines.** Electronic type exploders enable multi-channel initiation systems to be used. The electronics allow channels to be energised at varying time intervals. This enables blastholes connected to different channels to initiated at differing times. It requires a high degree of training to use electronic detonators and exploders, and therefore they only tend to be used for specialist applications. In addition specialist exploders are available for specific initiation systems e.g. special firing pistol for shocktube.
5.2.3.2  Trunklines and Downlines

Trunklines and downlines provide an energy transfer network between the exploder and the detonators. This network carries energy from the exploder to the detonator. Various electrical and non-electrical methods can be used, either singly or in combination.

**Shock Tubing.** Shock tubing is flexible plastic tubing, coated internally with a reactive substance that propagates a low energy shock wave at approximately 2000 ms\(^{-1}\). The shock wave initiates the primer or delay in a detonator. The small amount of reactive coating in the tube means that tube remains intact, producing no lateral shock, and merely conducts the signal. Shock tubing is extremely resistant to accidental initiation by flame, impact, stray currents, static or friction. Shock tubing can be used for initiating systems.

**Detonating Cord.** Whilst detonating cord can be used as an explosive or a detonator it can also be used as a communication network. A full description of detonating cord is given in Detonators and Accessories.

**Electrical Cable.** Electrical cable can be used to transfer the energy from the exploder to the detonator.

5.2.3.3  Detonators and Accessories

The initiation devices used for detonation have evolved over time, with the explosives. Modern explosives are more insensitive and far safer than classic dynamites and therefore require more energetic initiation. Modern initiation devices allow better control over initiation timings for blasting, enabling improved fragmentation and giving a reduction in vibration, air blast over pressure and fly-rock. They also provide greater efficiency, flexibility and safety in blasting operations (Jimeno et al 1995).

A powerful, localised shock is required to initiate detonation in modern commercial explosives. The purpose of a detonator is to initiate either the main charge or the primer charge. They are supplied as either No.8 (0.45g base charge) or No.8* (0.8g base charge) strengths, and are sufficient to initiate total effective detonation in a detonator-sensitive explosive.

Modern detonators used most commonly consist mainly of electric, electronic or shock tube detonators (Figure 5.10).

Detonators are linked by an initiation system. This allows the blast to be precisely controlled and allows individual explosive charges in the blast to be detonated sequentially, either singly or in groups. Initiation can be used to improve fragmentation and reduce blast vibration. Delay periods can be varied from a few tens to several hundreds of milliseconds, and are manufactured in a limited number of standard timings.

There are various types of initiation system available, varying from using detonating cord, through to more sophisticated electric initiation systems and non-electric systems.

Consideration must be given to the use of free faces for blasting efficiency. With multi-row shots, hole placement and timing sequence are fundamental in controlling the blast efficiency by reducing fly-rock and vibration, and improving fragmentation.
Figure 5.10 Modern explosive detonators - electric, Nonel and electronic (Not to Scale).
**Electric Detonators.** Electric detonators use an electrical current to set off a primer charge and this in turn initiates a base charge. Most modern electric detonators have delay systems built-in, which postpone detonation for a set period. This may be required for efficient blasting, with the charges detonated in a predetermined sequence and at specific time intervals. This can reduce blast vibration, the possibility of fly-rock and cut-offs, and improves blast fragmentation. The delay element consists of a column of a slow burning composition, contained in the detonator. The size and burning rate of the column determines the delay time. Delay periods range from a few milliseconds to several hundred milliseconds (25 to 845ms), and detonators are manufactured with a limited number of standard timings.

Electric detonators in a single blast are normally connected in series. However, parallel and series-in-parallel circuits may be needed where:

- There is an increased chance of misfires due to current leakage, e.g. if a large number of detonators are simply connected in series.
- To fire a number of rounds instantaneously, using a single energy source.
- Using a parallel circuit may introduce the possibility of misfires caused by an unbalanced circuit and is therefore not recommended for normal practice.

**Electronic Detonators.** With electronic detonators, an electronic blasting machine is necessary to precisely set the delay and to activate the detonator. Once activated they can "sleep" in-situ until a precise pre-set time for detonation. This can avoid "cut-offs", where the wire in an electric system may be severed by ground movement caused by an earlier detonation. Extremely short delays are possible with programmable electronic delay detonators and they are claimed to both improve fragmentation and reduce blast vibration.

**Detonating Cord.** Detonating cord can be used as a detonator in itself. Not all explosives are sensitive to detonating cord. Detonating cord consists of a central core of high explosive (usually PETN), in varying amounts (6, 10, 12, 15, 20, 40, 80, and 100 gm⁻¹), and is contained in a protective wrapping and a plastic sheath. The cord has a high velocity of detonation (6000 to 7000ms⁻¹). Detonating cord can be used to detonate high explosives lying alongside it in a drill hole, for the simultaneous firing of widely spaced charges and for the mass initiation of large charges. Detonating cord is extremely water resistant and much more robust than a detonator. It can also be used for some specialised controlled blasting techniques.

**Primers.** The primer is the part of an explosive charge or column into which the detonator and/or the detonating cord is placed. The purpose of the primer is to initiate the main charge. The choice of primer in both size and type is crucial to obtaining the best performance from any explosive charge and it should be chosen to suit local conditions. To transmit the detonation wave from the primer to another explosive, the primer has to generate sufficient shock energy at not less than the steady state velocity of detonation of the explosive. The shock effect of the primer depends on the detonation pressure of the composition and the ratio of the area of contact of the primer, to the cross sectional area of the primed explosive.

**Boosters.** Boosters are charges of higher bulk strength than the main column charge. Their purpose is to complete the initiation work of the primer by boosting the energy release, or to increase the energy release in the column, and hence the breaking ability. They generally ensure efficient detonation of the column charge.
5.3 Fracture Formation and Propagation

5.3.1 The Relationship between Pressure, Time and Fracturing

The detonation of an explosive causes high volumes and pressure (which can exceed 10GPa) of gas to be generated over a very short interval of time. This is sufficient to shatter rock adjacent to the hole and causes a shock wave to be transferred into the surrounding material. Continued burning of the explosive yields a supply of gas, but the pressure reduces with time. An idea of the pressure/time relationship is given in Figure 5.11. The rock mass reacts to the stresses induced according to the type and level of stress induced, and its own physical properties.

![Figure 5.11 Explosion in rock - Pressure, Time and Fracturing.](image)

The shock wave causes high stresses to be formed in the borehole wall. These stresses are rapidly transferred into the surrounding rock mass at a velocity typically in the range 3000ms\(^{-1}\) to 5000ms\(^{-1}\). This normally occurs a few tens of microseconds after detonation. If the stress levels exceed the compressive strength of the rock, then a crush zone is formed around the drill hole wall. If the stress levels lie between the compressive and tensile strength of the rock, then tensile fractures are formed, radially to the drill hole. Because, the shock wave rapidly loses energy, the level of stress quickly becomes insufficient to cause tensile failure and the fracturing ceases.

The leading front of the stress wave is compressive, but it is closely followed by the tensile stresses that are mainly responsible for rock fragmentation. A compressive wave reflects when it reaches a nearby exposed rock free surface, and on reflection, becomes a tensile strain pulse. Rock breaks much more easily in tension than compression, and “cratering” progresses backward from the free surface.

This usually produces a zone of radial fractures in the rock surrounding the drill hole. Continuing gas pressure in the hole, lasting some tens of milliseconds after detonation, causes gas to expand into the fractures in the surrounding rock. In the radial fractures, this results in a wedging action which causes crack tip stresses, promotes extension radially and gives rise to fracture dilation and rock mass heave.

The degree to which radial fracturing is produced is dependent on the grain size of the rock material involved. Fine-grained rock material shows the most developed patterns of radial fracturing.
The effects of fracture formation and propagation on ideal materials have been modelled in the laboratory using blocks of epoxy resin and 'plexiglass' (Matheson 1983b). These illustrate how fractures form and propagate following detonation of a high explosive charge.

The nature and degree of fracturing surrounding a drill hole, after detonating explosives, depends on many factors. These include the diameter of the hole, the explosives used, the diameter of the charge, the degree of coupling between charge and drill hole wall, and the properties of the rock mass. The effect of varying the drill hole diameter whilst keeping the other factors constant, is illustrated in Figure 5.12.

![Fracture Formation and Propagation](image1)

**Figure 5.12 Effect of Altering the Drill Hole Diameter (after Matheson 1983).**

### 5.3.2 Effect of Regional and Local Stress Fields

If a regional or local stress field exists in the rock mass, then the pattern and behaviour of the fractures may be modified. The effect is likely to be greatest when the major principal stress is orientated normal to the drill hole and will have most influence during the phase of continuing gas pressures. Fracture propagation parallel to this stress will be favoured, and propagation at high angles inhibited (Figure 5.13).

In the UK, the near-surface stress field in a rock mass is not considered significant and is unlikely to affect fracture formation and propagation during blasting. The directional stress fields formed during blasting can, however, be controlled and exploited to improve slope stability or enhance fragmentation.
5.3.2.1 Energy Partitioning

Most of the energy released in the detonation of an explosive substance is dissipated in heat, fracturing, and moving the rock. The term “energy partitioning” is used to describe the characteristics of an explosive in such terms (Figure 5.14), and indicates the proportion of energy used in each stage. Each type of explosive has its own energy profile, which can be modified by the charging and detonation technique used.

Figure 5.14 shows the energy present in the various components of a blast as areas under the Pressure/Volume curve following detonation. The static and dynamic energies (areas 2 and 3) are responsible for crack formation and growth, and the strain and heave energies (areas 4 and 5a) for ground movement. High velocity of detonation (VOD) explosives tend to have high kinetic and static energies, and low strain and heave energies. Low VOD explosives have low kinetic and static energies and high strain and heave energies. Consequently, the principal effect of high VOD explosives is to cause new fractures. The principal effect of low VOD explosives is to cause high gas pressures, crack dilation and heave.

Although most of the energy emanating from the explosive is utilised in these processes, considerable energy is lost in the shock wave and in escaping gas. Maximum efficiency is therefore achieved when both are kept to a minimum by carefully considering the amount of new fracturing required and limiting gas escape.
5.4 Blasting Techniques

In road rock excavations, there are two main applications of explosives; in forming slopes and in fragmenting the rock to allow its removal and reuse. “Controlled” blasting is used to limit induced instability in the rock face. “Fragmentation” (or “bulk”) blasting is used to fragment the rock mass. It is emphasised that the use of this terminology does not imply that fragmentation blasting should not be carefully controlled. The techniques are often used together, the first to form the final rock face and the second to fragment the rock in the region to be excavated allowing its removal (Figures 5.15 and 5.16). Where the final rock face stability is not critical, i.e. where it does not pose a significant risk to the road or public, fragmentation blasting can be used alone to provide economic and environmental advantages.

Blasting is usually carried out in panels, with a sequence of blasts followed by removal of blasted material. Alternatively, entire excavations can be formed in a single blast. The former technique is used on road construction schemes involving significant numbers of slopes, and where there are few restrictions for blasting. It also utilises a smaller amount of plant and personnel, in any part of the operation. Where there are blasting or time restrictions, the excavation or excavations may need to be formed in single blasts.

Deeper excavations, requiring multiple benches, have to be excavated in stages. Individual bench heights are often limited to 15m to prevent drill hole deviation. Competent drillers and good drilling equipment, freed from overly tight time restrictions, are capable of accurate drilling to far greater depths. However, many excavations in the UK exhibit the signs of drill hole deviation. Apart from drill equipment, set-up and the geological conditions, this is mainly due to excessive pull-down on the drill stem and excessive air pressures, because the job is being rushed for “economy”.

\[ \text{Fragmentation Energy} \]

- 1 = Kinetic Energy of Shock Wave
- 2 = Static Energy of Shock, equals Strain Energy in rock after passage of shock
- 3 = Dynamic Energy during crack growth
- 4 = Strain Energy in burden at release of burden
- 5a = Heave Energy
- 5b = Energy Lost in escaping gas
5.4.1 Control Blasting

Controlled blasting is used in situations where accurate removal of rock and minimisation of induced instability is required. In road excavations it is often employed to form the final face of the excavation to a given design, whilst optimising stability.

Two techniques are in common use, “presplit” blasting and “smooth” blasting (sometimes termed “trim” blasting). Both techniques use relatively small diameter holes, lightly charged and at relatively close centres along, or near, the

Figure 5.15 Controlled and Fragmentation Blasting Techniques - Presplit Blasting.

Figure 5.16 Controlled and Fragmentation Blasting Techniques - Smooth Blasting.
design face. The main differences are in the timing of detonations, relative to those used in the accompanying fragmentation blasting (Figures 5.15 and 5.16). In presplit blasting, charges on the design face are detonated simultaneously and before adjacent fragmentation charges. In smooth blasting, charges along or near the final face are detonated after adjacent fragmentation charges. Both techniques are aimed at minimising induced damage to the final face.

A stability assessment (Chapter 4) is required to ensure that the optimum design is selected. Controlled blasting techniques are only designed to minimise damage to the rock mass in the design face and cannot compensate for a poorly designed rock slope.

5.4.1.1 Presplit Blasting

In presplit blasting, the objective is to form an artificial plane or discontinuity (the presplit plane), along the design slope. This is accomplished by drilling a series of closely spaced holes along the design slope and charging them evenly with relatively light charges. The charges are detonated simultaneously to form the presplit plane. The plane limits the extent of the disturbance arising from fragmentation blasting to minimise the induced instability in the final rock face (Figure 5.17). The fragmentation blasting occurs in front of the presplit plane, which must be formed first.

Forming the presplit plane depends on generating a stress field between holes, which facilitates the propagation of an induced fracture between adjacent holes, whilst inhibiting the formation or growth of other fractures. The stress field forms during the quasi-static phase following detonation and can only fully develop in a rock mass that is confined, i.e. where lateral movement is restricted. The burden in front of the design slope is therefore of critical importance. Insufficient burden, or circumstances where the rock in front of the design slope can move outwards, means that a presplit plane may not be properly formed. A minimum of 10m burden is recommended in confined situations. This should be greater in weak or unstable ground.
The sequence of drilling, charging, detonation and mucking out associated with presplit blasting is illustrated in Figure 5.18.

![Figure 5.18 Presplit Blasting.](image)

The successful formation of a presplit plane is dependent on a number of factors, which must be carefully controlled. Hole diameter, hole spacing, burden and explosive charging are all fundamental and interdependent variables. The variables should be kept constant within a particular rock mass.

The drill hole spacings are critical. Too large and the presplit plane will not propagate between the holes. Too close and it will be uneconomic. The separation must be maintained for the entire length of the drill holes and therefore precision drilling is essential. Generally, drill hole spacing should be in the order of 10 times the drill hole diameter. Spacings of 1m, for 100mm diameter holes, have generally been successful in a wide variety of rock masses. A satisfactory face has been produced in some surface mining applications using larger spacings. However, in extremely strong or poorly fractured rocks, or where the face stability is critical, the spacing may have to be reduced slightly to attain an acceptable presplit plane.

In strong rock, carefully designed and implemented presplitting, can produce rock faces with minimal induced instability, where the burden is less than 10m. It should be noted that this reduction in burden may produce fly-rock and therefore may only be used where this can be tolerated (Nettleton et al 2000).

Horne (1998) describes the successful use of presplit blasting in a large opencast operation, to form a high wall face, using a drill hole spacing of 2.0m and 159mm diameter drill holes.

### 5.4.1.2 Smooth Blasting

In smooth blasting, charges along or near the final face, are detonated after adjacent fragmentation ones. The objective of smooth blasting is to form a stable face along the design slope. This is accomplished by drilling a series of closely spaced holes near, and parallel to, the design slope and charging them evenly and relatively lightly (Figure 5.16). The charges are detonated last in the fragmentation blasting sequence. Often they form the last part of a series of
delayed blasts in rows excavating rock as a series of benches (bench blasting) up to the final face. A relatively low burden causes movement towards the free face produced by earlier blasting and the close spacing causes outward “cratering” from adjacent holes. This, and the relatively low and even charging, limits but does not eliminate, blast disturbance in the rock mass forming the final face. Induced instability is therefore generally less than that typical of fragmentation blasting but more than presplit blasting.

For smooth blasting it is recommended that the same techniques described for presplit blasting are used, with the same degree of control, for setting out, drilling, hole diameter, hole spacing and charging. Detonation does not however need to be simultaneous and delays can be used between holes.

![Figure 5.19 Smooth Blasted Rock Slope (A9, Garry Weir).](image)

Smooth blasting is carried out in unconfined conditions, towards a free face. The burden is therefore less than that required for presplitting. Where burdens are low (less than 10m), or the conditions are unconfined, smooth blasting may be more appropriate than presplitting (see comments on presplit blasting with low burdens). The end result cannot however be expected to be as good as presplitting and some damage to the final face can be expected (Figure 5.19).

### 5.4.2 Fragmentation Blasting

Fragmentation blasting is used to break up the blocks naturally present within a rock mass to a size consistent with their removal and the intended use. For instance, in excavating rock in road construction projects, the resultant block sizes that must be able to be easily loaded and transported by mechanical plant out of the excavation area. The degree to which fragmentation occurs when bulk blasting depends on the range of block sizes originally present in the rock mass, the blasting technique and the particular blast design.

Fragmentation blasting can be used to form the final rock face, where the stability is not critical, and rockfall could not pose a significant risk to the road. Using fragmentation blasting alone to form a face is more economical in terms of blasting. However, this needs to be weighed against the economics of excavating larger volumes of material in creating the slope a safe distance away from the road, at a lower angle and disposing of the excess material. Once excavated, a
slope formed by fragmentation blasting may produce a more irregular and environmentally acceptable slope, similar to those in the surrounding landscape.

All rock masses are divided into discrete blocks by the discontinuities which they contain. The size and shape of the blocks is primarily a function of the orientation and spacing of these discontinuities. If a rock mass could be excavated without introducing new discontinuities, then a distribution of sizes characteristic of the volume of rock excavated could be established (Figure 5.20). This is the natural block size distribution (NBSD) within the rock mass and is analogous to a particle size distribution in a soil.

![Natural Block Size Distribution](image)

**Figure 5.20 Schematic Example of a Natural Block Size Distribution (NBSD) Curve for a Rock Mass.**

It should be noted that others (e.g. Lu & Latham 1996) use the term “In-Situ Block Size Distribution” (IBSD) to describe the block size distribution of a rock mass in place, and “Blasted Block Size Distribution” (BBSD) for that after blasting. The terms NBSD and BSD respectively have been used in this guide.

Some in-situ block distributions may not be either original or natural, such as where a rock mass has previously been damaged by explosives and new fracturing introduced. It is particularly apparent in quarry blasting for aggregates and in existing road cuttings formed by fragmentation blasting, where the in-situ block size distribution in the rock mass forming the face is smaller than that naturally occurring.

When a rock mass is excavated, the block size distribution (BSD) in the excavated material depends on the original NBSD, and the method of excavation used (Figure 5.21). In strong rock with a coarse NBSD, the explosives are required to reduce block sizes to dimensions at which they can be handled by mechanised plant and, if required, used as rock fill. If not achieved, then secondary breakage of the larger rock blocks will be required. This is both time consuming and expensive.
Techniques of estimating both the NBSD and the BSD (Figure 5.21) have been developed (Wang et al. 1992). These rely on the use of field data on the spacing and orientation of discontinuities present within the area to be excavated. The data can be obtained by extrapolating data from outcrop mapping, from scanlines on surface exposures, or from CCTV scanning of boreholes in the rock mass. The NBSD is determined either by a dissection technique in which the bounding coordinates of every block formed by the intersecting discontinuities are calculated, or by using equations involving the principal mean spacings and characteristic angles of up to three discontinuity sets present in the rock mass.

The BSD is then calculated using one of two widely used models for fragmentation blasting. The first combines the theory of Bond (1961) with the Rosin-Rammler equation (Cunningham 1983) to generate the resultant fragmentation curve, and is termed the Bond-Ram model. The second combines relationships in blast fragmentation proposed by Kuznetsov (1973) with the Rosin-Rammler equation to form the Kuz-Ram model, which is then used to generate the fragmentation curve. Both techniques and models have been incorporated into a computer program that allows the effects of varying the blast pattern and explosive charging to be estimated (Wang et al. 1992). Other techniques of estimating the BSD using the equation method and not requiring the computer programs have been outlined by Lu & Latham (1996).

In theory therefore, it is possible to calculate the NBSD of a rock mass, and choose a blasting technique and design that will fragment the rock mass to a BSD consistent with efficient mucking out and a size range suitable for use as any required rock fill type. However, the technique is not simple and considerable geotechnical expertise and experience is required to use it successfully.
5.5 Blast Design

Blasting or “explosive excavation” has evolved into an operation which is influenced both by the nature of the rock mass and by the selected technique, and the machinery available for drilling, blasting, excavating and hauling.

Any blast design must:

- Be safe.
- Produce optimum results.
- Be adequately flexible.
- Be simple to employ.

To produce the required final rock slope design, a balance between the hole pattern and the properties of the explosive type being employed must be found.

The blast geometry is defined by the hole depth, diameter and inclination, burden, spacing and the number of holes per row. Hole depth, diameter and inclination, and burden are all interrelated (Figure 5.22). The amount of sub-grade drilling (sub-drill) and the depth of any stemming can be considered as part of the blast geometry (White and Robinson 1995).

![Figure 5.22 Terminology used in blasting.](image)

5.5.1 Drilling

5.5.1.1 Direction of Excavation

For safe working, the direction of excavation should be carefully considered with respect to the geological conditions (Figure 5.23). The safest direction of working should be selected, that minimises potential failures into the open excavation, during the muck-shift and drilling of the next excavation panel.
5.5.1.2  Face Height

The face height can significantly affect the results of the blasting. The ratio of the face height (individual benches) to burden is important in achieving the desired fragmentation. High faces pose the problem of drill bit wander and thus drilling deviation considerations often limit the bench height. As well as geological conditions, hole deviation is caused by poor drill set-up, inadequate drilling equipment, excessive pull-down on the drill stem (due to time pressures) and exceeding the optimum compressed air pressure. These factors can be a particular problem in the UK, and are in part due to the way construction contracts have typically been run, on very tight budgets and to very tight time scales. In the UK individual presplit faces, of between 15 and 25m per bench, have been successfully formed. However, faces over 25m require careful selection of drilling equipment, contractors and timescales.

5.5.1.3  Drill Hole Diameter

The ideal diameter for the drill hole for blasting depends on; the rock mass properties, the fragmentation required, the face height, the charge configuration, drilling and blasting costs and the available plant (Jimeno et al 1995). Small diameter holes achieve a better distribution of charge but the costs of drilling, priming, initiation, charging, stemming and connection are high. A larger drill hole diameter and blasting pattern, for a given set of conditions, will generally produce coarser fragmentation. The drill hole spacing, should generally be less than the mean discontinuity spacing.

Drilling costs can generally be reduced by increasing the drill hole diameter and the blasting pattern. Increasing the blast ratio (i.e. the weight of explosives required to fragment a tonne of a particular rock mass to the required level) can make up for the loss of fragmentation. In rock types that exhibit networks of natural discontinuities such as limestone, the blast hole diameter has little affect, as the rock mass controls the fragmentation. If bulk explosives or slurries (ANFO and ANFO/emulsion blends) are used, greater fragmentation can be obtained by increasing the diameter, because their VOD increases with the diameter of the borehole, up to a maximum of 400mm (White and Robinson 1995).
5.5.1.4 Burden
The burden is the perpendicular distance from the drill hole to the free face. For "critical burden" to be reached the rock must be completely fragmented, but not displaced. Due to the complex nature of the breakage mechanisms involved for different rock masses and explosives mixtures, there is no exact relationship between burden and hole diameter. However, the burden should generally be between twenty-five and forty times the borehole diameter, with higher values for weaker rock masses and lower values for stronger rock masses (Jimeno et al 1995).

5.5.1.5 Hole Spacing
The spacing is a function of the burden, delay timing and initiation sequence. Small spacings potentially cause excessive fragmentation between holes, cratering, insufficient fragmentation in front of the holes and problems at the toe. Larger spacings potentially cause inadequate fragmentation between holes, overhangs and problems at the toe (Jimeno et al 1995). In the UK, hole spacing is usually made equal to the burden but most texts recommend a spacing of between 1 to 1.5 times the burden to improve fragmentation.

5.5.1.6 Inclination of the Drill Hole
Inclined drilling offers several advantages, these potentially include (after Jimeno et al 1995):

• Increased fragmentation, displacement and bulking of the muck pile due to better charge distribution.
• Reduced probability of a misfire caused by burden movement.
• Less irregular and more stable faces.
• Less sub-drilling and more efficient use of explosive energy and reduced vibration.
• Increased drilling per unit volume excavated.
• Reduction in the volume of rock unaffected by the explosive charge (Figure 5.24).

5.5.1.7 Sub-Grade Drilling
To ensure all material is fragmented, down to the designed floor level of the excavation, blast holes are drilled to below this level. The sub-grade drill (or "sub-drill") is usually a maximum of one third of the burden distance. Explosives placed below the proposed floor level have an effect on fragmentation and movement of the rock at this level, but it is usually minimal.

To reduce the amount of sub-drill, the drill holes should be inclined and a high energy base charge should be used. Where a rock trap ditch is to be formed, it should be properly accounted for in the excavation design, separately to the sub-drill.
5.5.1.8 Hole Depth
The total hole depth is the height of the face (including any rock trap ditch) plus the sub-grade drilling, taking account of the inclination of the drill hole.

5.5.2 Explosives

5.5.2.1 Base Charge
Explosives at the base of the blast hole have to fragment the rock and form the new floor level. They are required to do approximately three times the amount of work as the rest of the explosive column. Therefore the base charge is usually a higher energy concentration than the column charge. In practice, the weight of the base charge of the explosive column can be increased so that the explosive is so far from the base that it produces excessive fragmentation above floor level and damages the face. The following ‘rule of thumb’ can be followed (White and Robinson 1995):

Maximum height of base charge = Subgrade + Burden

Field trials may show that less than this maximum height of base charge is required. If so, it should be possible to increase the burden, if optimum fragmentation can still be obtained.
5.5.2.2 **Column Charge**
The column charge is the explosives placed between the base charge and the stemming. The charge concentration is usually lower than the base charge.

5.5.2.3 **Stemming**
Stemming is the part of the drill hole above the charge that is filled with inert material that confines the explosive gases. The aim of stemming is to improve the efficiency of the blast and therefore fragmentation. Insufficient stemming will potentially increase blast vibration and fly-rock. The optimum length of stemming increases as the quality of the rock mass decreases, and is somewhere in the range of twenty to forty times the drill hole diameter. Generally, in strong rock, the stemming should be greater or equal to the burden. Stemming material should be 10mm chippings, or similar acceptable material, and drill returns should not be used. Stemming can be disrupted by the use of detonating cord to ignite the explosive column.

5.5.2.4 **Detonating Method**
The method of detonation of a column of explosives will significantly affect the resultant fragmentation and final face stability. There has been a move away from using down-the-hole detonating cord, to initiate a sensitive primer, which then initiates a sensitive or non-sensitive column of explosive. Nowadays, point initiation of a primer is used to ensure that the explosive column is initiated only at its optimum point, and low order detonations do not take place within the explosive column. Higher efficiency can thus be obtained by point initiation, which will not disrupt the stemming or cause the “waterfall effect” of explosive being detonated from the top of the column.

In general, detonating cord running from the surface to the bottom of the hole is robust and simple to understand and use. Using down-the-hole detonators provides environmental advantages and better fragmentation.

5.5.2.5 **Detonating Sequence**
Delayed detonation sequences are used for efficiency, fragmentation and environmental purposes. The maximum instantaneous charge (MIC) is used to predict vibration levels. The creation and use of “free faces” during the blast must be considered. Consideration of free faces gives much better control of the blast, reducing vibration levels, improving fragmentation and reducing the possibility of cut-offs (non-detonations) and fly-rock. With multiple rows of charges, the delay sequence and hole spacing are fundamental in achieving effective excavation.

Typical delays of 10, 17 and 25 milliseconds are used between adjacent blast holes. Detonators are can be used on the surface to initiate detonating cord running down the hole. This gives more flexibility when sequencing the blast, as well as reducing environmental disturbance. However, detonating cord running through the stemming will still disrupt the blast hole, and the charge will not be as well confined as it should be. Using detonators placed directly in primer cartridges in the hole, this effect is reduced, and the explosives work more efficiently.
5.6 Drilling Operations for Blasting

5.6.1 Site Preparation

It is important that the site is properly prepared for the start of drilling operations. Unprepared rough ground will cause delays and problems in both access for the drill rig and associated plant, and setting it up accurately. Temporary roadways may need to be constructed.

Before drilling commences, any overburden and loose or weathered material should be removed from the drilling area. Failure to do so may result in material falling into the hole and causing blockages or sticking of drill rods. It may also cause the drill to deviate off the planned position of the hole as the drill bit cannot “bite” cleanly into rock material.

The position of each hole must be accurately surveyed-in, and clearly marked on the rock surface, before drilling commences. Such positions are usually calculated from the location of the toe of the design slope and the design angle. The drill line, the straight line joining drill holes, will deviate with the topography and will only be a straight line if the ground surface is absolutely level and/or the design face is vertical. The accurate surveying of drill holes is especially critical for those that are to be used for controlled blasting techniques, as they will directly affect the stability of the final face.

5.6.2 Drilling

Drilling accuracy is one of the most important factors governing successful explosive excavation. Drill holes must be positioned on the design face at the required spacing. To accomplish this requires the accurate alignment of the drill in terms of both the dip and azimuth (dip direction) (Figure 5.25). Drill rod alignment must be carried out using methods that remove subjective judgement from the set up.

For accurate drilling, the use of a drill hole orientation device (DOD) (Matheson 1984), that can consider both dip and azimuth and is not influenced by the presence of the drill steel, is recommended (Figure 5.26).

With increases in the height of the design face, the accuracy required to maintain the required drill hole position, in relation to adjoining holes, also increases. Accuracy should therefore be defined in terms of linear deviation. This can be measured at the base of the pre-split hole and defined in terms of deviation in, and from, the proposed design plane. The objective in setting limits for deviation is to ensure that, at any part of the design face, drill hole spacing does not exceed that which is required for the successful formation of the designed face. It also ensures that charge concentration, with the potential for face damage, does not occur.
Dip and azimuth variations corresponding to in-plane tolerances of 300mm and from-plane tolerances of 200mm are given in Figure 5.27, for varying drill hole lengths. These can be used to indicate the drill set-up tolerances allowed to achieve the required hole accuracy. It should be noted that these refer to perfectly straight holes. In general, holes should be drilled in one lift whenever possible.
5.6.3 **Drill Hole Deviation**

Although accuracy in setting up is of vital importance, deviation of the drill head with depth can occur (Figure 5.28) as the result of a number of conditions. It is important that these are minimised.

Drill head deviation causing misalignment generally takes place as the hammer first bites into the rock or within approximately the first half metre. Up to this depth corrections to the drill rod orientation are generally possible. Following an initial alignment before drilling starts, a further alignment is required immediately after the drill bit bites and again when it is approximately 0.3m into rock. Drill hole deviation with depth can be caused either by the drilling, equipment, technique or by the geological conditions encountered by the drill bit. Deviation is limited by the use of down-the-hole hammer (DTH) rigs, rather than “drifter” or “hydraulic” rigs. For DTH rigs the drill rods are thicker, and therefore stiffer, and the percussive mechanism is located at the drill head. For drifter and hydraulic rigs, the drill rods are thinner and the rotary and percussive mechanism are both located on the drill rig, the action and load being transferred down the drill rod to the drill bit. DTH rigs are now used almost exclusively for drilling accurate drill holes for blasting.
Figure 5.28 Examples of Drill Hole Deviation.

Geological/geotechnical conditions likely to cause deviation are prominent discontinuities, alternating hard and soft bands in the rock or a change in rock type encountered by the drill head. If the drill bit encounters these at a high angle it will tend to become normal to the feature, and at a low angle it will tend to run along it. Other deviations can almost exclusively be attributed either to the drilling technique or the condition of the equipment, particularly the drill bit. The technique of using high pull-down on the drill stem and high output from the compressor, in an attempt to increase the rate of drilling, will also cause deviation.

Most geological/geotechnical conditions will not normally change dramatically in the scale of the drill hole separation. Holes drilled by down-the-hole hammer techniques should therefore tend to remain parallel to each other (e.g. on the A837 at Garelochhead, Figure 5.28). This generally satisfies the requirements of presplit blasting and a successful, although curvi-planar, surface can result. Such curvature is only of practical importance if it causes increases in face dip towards the base of the face and results in the daylighting of failure surfaces, thus increasing failure potential.

Completed drill holes can be checked for accuracy by inserting a scaffolding tube, suitably positioned in the hole and with a small cross bar to stop the tube slipping into the hole, and by using a DOD. The deviation of “half barrels” formed during presplit blasting can be directly measured on the face after blasting and excavation.

5.7 Vibrations from Blasting

5.7.1 Introduction to Blasting Vibration

When explosives are detonated, shock waves are produced. These are transmitted outwards from the source through the ground and the air. Groundborne vibrations take the form of compressive, shear and body waves (Figure 5.29). The compressive wave is the strongest, propagates outwards the fastest and is responsible for most damage to nearby structures. Shock waves are also transmitted through the air and, when strong enough, can also lead to
damage (Figure 5.30). However, the most notable effect is usually the disturbance to local residents.

Figure 5.29 Type and propagation of Ground borne vibrations from surface blasting.

Figure 5.30 Type and propagation of airborne vibrations from surface blasting.
5.7.1.1 Groundborne Vibration

A compressive wave originating from a blast can be defined in terms of displacement and frequency (Figure 5.31). Displacement is a measure of the movement of a particle affected by the wave before returning to its original position. Frequency is the number of oscillations per second of that movement.

![Diagram showing Peak Particle Velocity (PPV), Velocity, and Acceleration](image)

**Figure 5.31** Compressive wave terminology and Peak Particle Velocity (PPV).

The Peak Particle Velocity (PPV) is normally used as the measure of maximum velocity (Figure 5.31). This is the highest velocity of a particle affected by the wave.

![Equations for Velocity and Acceleration](image)

**Figure 5.32** Calculation of velocity and acceleration.

The velocity and acceleration can be calculated from the displacement and frequency (Figure 5.32). Velocity is a measure of the speed of a particle as it moves about its rest position. Acceleration is the rate at which the particle changes velocity.
5.7.1.2  Measurement

A seismograph (Figure 5.33) is used to obtain a continuous record of the passing vibration. Because of the 3-dimensional nature of the wave, it is necessary to measure the wave in three separate planes. Independent geophones, mounted orthogonally, are therefore required. Three traces of vibration in the Vertical (V), Longitudinal (L) and Transverse (T) planes are consequently recorded (Figure 5.34). The plane containing the maximum vibration may not be orientated along a straight line between the blast and the location of measurement and the maximum may not coincide with one of the planes of measurement. It is therefore necessary to determine this by calculating the resultant.

![Seismograph for recording ground and airborne vibrations.](image)

The resultant PPV for the entire wave can be calculated by vectorily summing the PPV from each trace by taking the square root of the sum of the squares of each maximum PPV (Figure 5.35). This is conservative approximation as individual PPVs may not be coincident in time.

![Vibration traces from a seismograph.](image)

\[
\text{Resultant PPV} = \sqrt{V^2 + L^2 + T^2}
\]

Both PPV and frequency are required to characterise groundborne vibrations. Peak values of both parameters are used to fully define a wave and it is usual to
express both as maximum values experienced during the passing of the wave (Figure 5.36). However, it should be noted that these may not be coincident in time. The approach is therefore again conservative.

The true resultant PPV is obtained by vectorily summing the three orthogonal components coincident with time. The peak true resultant PPV is then the maximum value of the true vector sum over a given time interval. This figure is the PPV usually calculated automatically by modern seismographs and printed out as a record of the vibration measurement (Figure 5.37). It is likely to be the most accurate and should therefore be used in evaluations.

![Figure 5.36 PPV and frequency.](image)

5.7.1.3 Airborne Vibration

Shock waves are transmitted through the air as compressive waves radiating outwards from the source. This can be defined in terms of air overpressure as pressure (bars or Pascals) or noise (decibels). This can be measured relatively easily, by an acoustic microphone on a standard seismograph. The relationship

![Figure 5.37 Printout from a seismograph.](image)
between pressure and noise for the range of concern to blasting is shown in Figure 5.38.

Transmission through the air is influenced by atmospheric conditions. High wind, low cloud or temperature inversion can give rise to abnormal levels of air overpressure (Figure 5.39). It is good practice to avoid blasting if there are adverse weather conditions or if they are expected. Weather forecasts should therefore be obtained covering the period of the time of the blast.

**Figure 5.38 Relationship between pressure and noise.**

**Figure 5.39 Effects of varying wind speed and a temperature inversion on airborne vibration.**

5.7.1.4 **Prediction & Monitoring**

Geological conditions can influence both the magnitude and direction characteristics of groundborne blast-induced vibrations. These include the attitude and nature of stratigraphic layers, the presence of igneous intrusions, the banding and composition of metamorphic rocks, the existence and location of folds, faults, fractures, the nature and thickness of overburden and the existence and depth of the water table and so on.

Generally, the level of ground vibration experienced is a function of the original charge weight and the distance from the source. The relationship can be determined by plotting peak true resultant PPVs against a range of scaled distances, defined as the distance from the explosion divided by the square root
of the explosive charge weight, for different distances and/or charge weights (Figure 5.40).

A first order regression line through the points is normally used to quantify the relationship. Alternatively the 95% upper confidence level can be used, below which most of the vibration levels will fall.

If a large number of regression lines obtained from a wide variety of civil engineering projects are plotted together (Figure 5.41), then a range of experience can be defined (New 1982). Individual PPVs and regression lines should therefore be within this range. If they are outside the range it could indicate suspect data, enhancement or abnormal ground conditions, requiring special evaluation.

![Figure 5.40 Plot of PPV against scaled distance for groundborne vibration.](image)

It is therefore good practice to superimpose the bounds of experience on any new set of vibration data to verify “normality”. Theoretical predictions of the magnitude and frequency of blast-induced vibrations at a location distant from the blast source can be unreliable. It is not good practice to assume that the strongest vibrations will invariably be transmitted directly from the blast source. In all cases a blasting trial is strongly recommended.

![Figure 5.41 Range of experience for civil engineering blasting.](image)
A blasting trial can be carried out in advance of the excavation by detonating relatively small charges and measuring the vibration characteristics at increasing distances away from the source. It is good practice to mount geophones on all sensitive structures, not only those nearest the vibration source. This will allow any local anomalies to be identified and actual, rather than predicted, vibration levels to be established and used to limit charge weights.

5.7.1.5 Specification and Control

Guidance for the control of transient (short-lived) blast vibration, to avoid any damage to structures is given in BS7385 Part 2:1993. This uses the level of component PPV at the predominant frequency to set an upper limit of component PPV for the onset of damage and is reproduced in Table 5.42. It should be noted that this PPV is the maximum of all components of the vibration. Frequently this is taken to be the highest PPV from the vibration recorder. However, such a PPV may not actually be the maximum of the wave. The maximum is most accurately represented by the peak true resultant PPV.

The increased importance of low frequency vibrations to light framed structures such as residential buildings is taken into account. The changing lower limit relationship between PPV and frequency for the two types of building (Lines 1 and 2) is clearly illustrated in graphical form in Figure 5.43. Whereas a 50mms-1 upper limit for all frequencies is appropriate to relatively strong structures, this must be progressively reduced to 15mms-1 over the longer frequencies below 40 Hz.

Table 5.42 Damage limits for transient vibrations.

<table>
<thead>
<tr>
<th>Type of Building</th>
<th>PPV at predominant</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>4 Hz to 15</td>
</tr>
<tr>
<td>Reinforced or framed Industrial and heavy buildings</td>
<td>50mms(^{-1}) at 4 Hz and above</td>
</tr>
<tr>
<td>Unreinforced or light framed structure Residential or light type</td>
<td>15mms(^{-1}) at 4 increasing to 20mms(^{-1}) at 15 Hz</td>
</tr>
</tbody>
</table>

20mms\(^{-1}\) at 15 increasing to 50mms\(^{-1}\) at 40 and

NOTE 1 Values referred to are at the base of the building.
NOTE 2 For Line 2, at frequencies below 4 Hz, a maximum displacement of 0.6 mm (zero to peak) should not be exceeded.
Figure 5.43 Damage limits in terms of PPV and frequency.

The methods and limits in BS7385: Part 2:1993 are based on actual field experience and should be used without modification and in preference to other methods.

For airborne vibration, the noise levels likely to produce damage are regarded as above 140dB (the generally accepted damage threshold), although structural damage is not likely below 180dB. These are related to pressure and to common sources of noise in Figure 5.44.

Figure 5.44 Everyday levels and damage limits for airborne noise.
6 Environmental considerations

6.1 Environmental Impact
In general, environmental impact concerns changes that we impose on our surroundings. Such impacts can be broadly split into two groups:

- Perceived Impacts
- Actual Impacts

Perceived impacts are those that generally concern our appreciation of our environment. Actual impacts are those that concern damage or change to the environment. These impacts can be viewed as good or bad, acceptable or unacceptable and are judged by the nature and magnitude of the effects or perceived effects.

Rock slopes are excavated in natural materials. It is the environmental impact of rock slope excavations and their associated construction methods that is discussed here.

Evaluation of the perceived environmental impact of rock slope excavations is strongly influenced by personal judgement and opinion. Tourists, local residents, conservationists, motorists, road engineers and geologists can, and do, have widely differing views of acceptability. It is impossible to quantify these differences and therefore reaching a consensus on what is environmentally acceptable is always going to be difficult. A further complication with achieving consensus of view is that the attitudes of all of the interested parties vary with time, and at different rates.

Evaluation of actual impacts is slightly simpler than with perceived impact as we are dealing with effects that are at least in part measurable and quantifiable. The difficulties arise in agreeing thresholds of acceptable effects. Once again there are significant differences in opinion of what is acceptable between the interested parties. However, consensus is more likely when dealing with actual impacts than when dealing with perceived impacts.

When considering both perceived and actual environmental impact it is important to differentiate between the short and long term. Short-term impacts are those that nature is capable of repairing or minimising with the passage of time. Long-term impacts are permanent or effectively permanent in terms of the lifetime of the structure. Both can be minimised by good design and construction practice.

6.2 Acceptance and Tolerance
Acceptance and tolerance of environmental impact are very different. Questions of acceptance and tolerance of both perceived and actual environmental impact are closely linked to social attitudes and economic conditions. These are of course dynamic variables and change with time.

At any given time there are likely to be differences between what is tolerated and what is accepted. In general, impacts arising from historical activities will be tolerated at different levels to that set for acceptance of new activities. There are two main reasons for this:

1. Altering historical excavations to meet current standards will in itself cause an impact, even though it should eventually produce a better outcome.
2. Altering historical excavations is often expensive, and the approach of continually improving existing excavations to meet ever-changing new standards is impractical and unsustainable.

There is a third reason that applies to certain types of structures and impacts. Constructions, while not meeting current standards, may become of significant historical importance. If this occurs they will not only be tolerated but also accepted because of this importance. It is unlikely that any road rock slope excavation will ever fall into this category of construction.

6.3 Good Practice

There are therefore difficulties in defining perceived and actual environmental impact standards that are going to be tolerated by or accepted by all interested parties. It is clear that the engineering geologist or geotechnical engineer designing or constructing rock slope excavations is faced with significant hurdles to overcome. The designer and construction supervisor need to consider all of the views of interested parties and make difficult decisions based on current best practice and published guidance. Sections 7.4 to 7.7 set out the existing guidance on environmental impact that are currently applicable to Scotland. It is essential that the designer and construction supervisor keep abreast of developments in best practice if they are to achieve any degree of acceptance for their work. Section 7.7 gives practical guidance specifically on reducing environmental impact for rock slope excavations.

For excavations in areas of high environmental sensitivity or importance such as Sites of Special Scientific Interest (SSSI), consultation becomes an integral part of the design and construction process. The authors have been involved in a number of such situations and in all cases have achieved at minimum tolerance of the works and in some cases full acceptance and even appreciation.

6.4 The Policy in Scotland

The Scottish policy on landscape design and management is set out in "Cost Effective Landscape: Learning from Nature" (The Scottish Office, 1998). The document forms the procedure for landscape design and management for the Scottish Trunk Road Network, but the concepts and procedures are equally applicable elsewhere.

"Cost Effective Landscape: Learning from Nature (CEL:LFN)" does not give guidance on environmental assessment or route planning but The Scottish Executive Development Department endorses the procedures, guidance and good practice described in the Design Manual for Roads and Bridges (DMRB). They require all professionals concerned with trunk road assessment, planning, design and management to comply with the DMRB. Volume 10 of the DMRB deals with "Environmental Design" and Volume 11 deals with "Environmental Assessment". The advice given in CEL:LFN should be followed in Scotland where advice or procedure differs from Volume 10 of the DMRB. The main aims of the policy are to improve the quality and efficiency of road landscape design and management through the application of natural characteristics and to apply the policy to all relevant landscape tasks. The application of the policy will be subjected to procedural inspections arranged by the client.

The Bottom Dead Centre design approach is recommended in CEF:LFN. Bottom dead centre is the state of rest that represents the natural self-reliant landscape and displacement from this position represents the degree of artificiality. The landscape will begin to revert to its natural state or “bottom dead centre” if energy input declines.
Therefore, the aim is to achieve design objectives as close to the natural state as possible through working with nature. Of particular relevance to excavations are the following considerations:

Identification of the natural processes which can be best harnessed to achieve the desired landscape objectives.

Awareness of the short term (capital/construction) costs and long term (revenue/maintenance) costs and the need to balance both types of expenditure.

Search for the sustainability benefits which can be delivered.

To produce quality and cost-effective landscape, designers must demonstrate that:

<table>
<thead>
<tr>
<th>Landscape Design &amp; Management</th>
<th>Cost Effective Landscapes</th>
</tr>
</thead>
<tbody>
<tr>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Learning from Nature</td>
<td>Environmental Benefits</td>
</tr>
</tbody>
</table>

The CEF:LFN policy encourages landscape designers to explore alternatives in finding the most appropriate solution. This will produce a more sensitive approach and greater attention to detail and promote the wise and sustainable use of resources. It will also give a more cost-effective use of expenditure.

The policy refers to rock outcrops only in terms of environmental design (as opposed to stability design). The following advice is given:

- Understanding the site and its natural characteristics.
- The natural characteristics of established and natural outcrops in the vicinity should be used for inspiration to design a rock cutting.
- Produce niches in the rock for pioneer plant species to create a diversity of appropriate vegetation.

**6.5 The DMRB - Environmental Design**

The DMRB (Design Manual for Roads and Bridges) gives advice on the “Environmental Design” of rock slopes, in Volume 10 Section 1 Part 1 Chapter 17 (HA55/92).

Issues covered include the use of terraces (berms), using the natural character of the rock and vegetation on cuttings. Photographic examples of good and bad practice are given.

Both the DMRB Volume 10 and CEL:LFN refer to environmental aspects of design and not to stable rock slope design. In reality, when excavating a rock slope, the design will be a compromise between these two factors. Some comments referring to the design of rock slopes in the DMRB are unhelpful, for example: “Good practice - creating the profile that might be found on a quarry...”. This is unhelpful both in terms of stability and aesthetics, quarry slopes are generally less stable and have a less “natural” appearance than road rock slopes and would therefore not be acceptable.

**6.6 Application of the CEL:LFN and DMRB Policies**

Rock slope excavations are a change to the natural landscape and will therefore have some environmental impact. This may be beneficial or detrimental.

The primary concern of the designer of rock slope excavations should be the minimisation of risk from rockfall to the road. To find the most appropriate
approach or solutions for a particular situation, alternatives that achieve the same minimisation of risk should be considered.

The self-reliance or "bottom dead centre" design approach outlined in CEL:LFN is particularly pertinent to rock slope excavations. Ideally, once constructed, rock slope excavations would never require further intervention. The long-term cost-effectiveness of rock slope excavations, in terms of whole-life costs, is particularly important. The less intervention or maintenance required by rock slope excavations, the more likely it is that their whole-life cost will be low.

Rock slopes themselves are not strictly sustainable because, being above the mean horizontal plane, they are all to some extent, unstable (see Chapter 2). Consideration should be given to the sustainability of features integral to the design of the excavation, for example rock traps at the base of slopes. Such features constructed from natural materials, for example rock traps formed by earth bunds or ditches, are generally more environmentally acceptable, cost-effective and self-reliant than "artificial" materials. However, even these systems require maintenance, i.e. the clearance of fallen debris that reduces the effectiveness of the rock trap. Efforts should be made to reduce the environmental impact of a solution, rather than to consider unpractical and less cost-effective alternatives.

Environmental impact is usually a secondary issue, as public safety is the prime concern for those involved in commissioning rock slope excavations. Consideration of environmental issues, in sensitive areas, inevitably increases the cost of a project through mediation with relevant experts on the site's special sensitivity, for example botanists, biologists or archaeologists.

Even in circumstances where public safety is the priority, and only limited financial resources are available, environmental concerns can still be addressed. If it is accepted that the rock slope excavation will bring some change to the pre-existing status quo, then efforts can be concentrated on limiting the environmental impact.

### 6.7 Assessment of Environmental Impact

Environmental Impact Assessments (EIA's) are not mandatory for most rock slope excavations. However, the principles of EIA should be considered as representing and encouraging best environmental practice. The EC Directive 85/337 (Assessment of the Effects of Certain Public and Private Projects on the Environment) set out the requirement for EIA's (McKirdy et al, 1998).

The EIA process is important because it allows all relevant matters, including geological issues, to be considered during the planning process. It is up to the planning authority to ensure that all relevant issues are adequately explored in the EIA. Only certain developments require an EIA by law, for example motorway construction. Other developments are listed as requiring an EIA if the planning authority decides it is necessary (McKirdy et al, 1998).

For an EIA, an environmental baseline is established to describe the present state of the environment and the way it would change, assuming that the development did not go ahead. The baseline provides a measure against which any changes resulting from the development can be estimated. Various factors should be considered in establishing the baseline, including soil, geology, geomorphology, and ground and surface water. In many projects, geological and surface processes are not considered in depth. The EIA allows for monitoring of the development post-construction, to measure the actual against the predicted impact (McKirdy et al, 1998).
Those who are most affected are the residents that live adjacent to the proposed development site and there is a requirement in an EIA for a non-technical summary of the issues (McKirdy et al, 1998).

EIA's are unique to the individual circumstances of the scheme. Major schemes in highly sensitive areas will require a more comprehensive assessment than those in less sensitive environments.

Wiltshire et al (1987) give a basic guide to the geological input required for an EIA for a mineral extraction operation. This can be adopted for rock slope excavations:

**Stage 1:** A statement of the objectives, including a description of the area and a project plan (with time scales). This will be provided by the commissioner and will include geological advice independent of the EIA.

**Stage 2:** A review of the present environmental conditions, including ecology, hydrology, soils and agriculture, and issues such as landscape value, visual aesthetics, historic buildings, protected sites (for example SSSI's), etc.

**Stage 3:** Requirements of the planning policy should be considered. A dialogue between the contractor, consultants and planning authority should be established.

**Stage 4:** Systematic consideration of the environmental impacts of alternative options. This requires a thorough description of the proposed development actions, particularly those with potentially deleterious consequences.

**Stage 5:** The likely environmental consequences of alternative schemes must be investigated. The impact is assessed by predicting the likely changes in the natural and human environments. The subsequent selection will therefore take account of the possibility of refusal of planning permission on environmental grounds and of any requirement for mitigating measures. Therefore, geological factors must be assessed before advice on minimum impacts is given.

**Stage 6:** The risk reduction, economic and environmental advantages of the preferred scheme must be clearly stated so that the planning authority can understand the basis for the selection of a particular option.

**Stage 7:** A detailed analysis of the potential impacts of the preferred scheme should be made, if this has not been provided in Stage 5. Geological and/or geotechnical specialists may be required to provide expert advice.

**Stage 8:** If the impacts are serious, then the viability of measures, which can limit the potential for environmental damage, must be considered. This may include additional temporary works.

The public community's perception of the development is an important part of the EIA. The benefits of the rock slope excavation should be discussed at a public meeting, especially if the proposed construction is going to cause any inconvenience to the local community, for example traffic congestion. If necessary, arrangements should be made to minimise disruption to the local community. However, with adequate notice and explanation of the benefits of the development, this may not be necessary.

### 6.8 Practical Methods to Minimise Environmental Impact

Advice, advantages and practical tips to minimise the environmental impact for rock slope excavations are given in Table 6.1.

Innovative approaches are sometimes required. A slope that is successful in terms of environmental impact is one that remains largely unnoticed within the surrounding landscape.
Examples of good practice for minimising the environmental impact of rock slope excavations are shown in Figures 6.1 to 6.12.
Table 6.1 Practical Advice and Tips on Minimising Environmental Impact for Rock Slope Excavations

<table>
<thead>
<tr>
<th>Options and Techniques</th>
<th>Advice and Tips</th>
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<td><strong>General</strong></td>
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| General Design         | • Primary objective is to create an acceptable slope - in terms of risk to the road and environmentally.  
                        | • Use natural discontinuities to design the new slope, by observing slope angles of local natural outcrops - important for producing both a stable slope and a natural appearance.  
                        | • If land-take allows, form an irregular face at a relatively low overall angle (without compromising the safety of the road).  
                        | • Place a layer of soil/peat on berms and vegetate with appropriate* native varieties - vegetation helps to dissipate the energy of any rockfalls.  
                        | • Consider forming longitudinally inclined berms to avoid horizontal “lines” (NB. berms must be a minimum of 4m wide and must not dip out of the slope).  
                        | • Rock colour weathers quickly to natural colour.  
                        | • Use planting of appropriate* local vegetation and/or encourage the growth of existing plant species. |
| **Rock Traps** (Verges, Ditches/Bunds and Fences) | • Use ditches/bunds constructed of local, natural materials (rather than fences) wherever possible and encourage vegetation growth.  
                        | • Paint fence posts, anchors, attachments, etc, an appropriate colour.  
                        | • Use green netting for any fences.  
                        | • Remove the minimum amount of established vegetation, for example by altering the fence line to avoid mature trees (vegetation also compliments the effect of rock traps) and encourage growth of local plant species.  
                        | • Plant shrubs and small trees in rockfall ditches. |

* Appropriate vegetation is vegetation that does not have invasive deep root systems, which could loosen or exacerbate existing failure geometries.
Figure 6.1 Rock slope with a retaining wall above. The rock slope is dominated by the large retaining wall, which is out of character with the landscape (A9, Killiecrankie, northbound view).

Figure 6.2 Rock Slope with a Retaining Wall above. The rock slope is dominated by the large retaining wall, which is out of character with the landscape (A9, Killiecrankie, southbound view).
Figure 6.3 Rock Slope circa 10 years old. Environmentally acceptable, well vegetated slope (A887, Glen Moriston).

Figure 6.4 Rock Slope during construction. The large expanse of the slope has been broken-up using embayments, creating an irregular horizontal profile (A830, Polnish).
Figure 6.5 Box cut. Slope in view formed on natural planes, vegetation becoming established in natural niches to break up flat expanse and soften the edges of the slope (A835, Garve northbound view).

Figure 6.6 Box cut. Slope in view formed across natural planes, good contrast with opposite side of cut, see Figure 6.5 (A835, Garve southbound view).
Figure 6.7 Presplit slope. The excavation has broken back to persistent and widely spaced discontinuities, which dip into the cutting and which were clearly visible forming natural rock outcrops in the area (A830, Loch Nan Uamh).

Figure 6.8 Presplit slope during construction. Contrasting with natural rock outcrops on the hillside above, this contrast was later decreased by vegetation restoration (A830, Loch Nan Uamh).
Figure 6.9 Well established vegetation on a small cutting. The slope blends-in well with the landscape. Note the importance of ditch to contain small rock fall debris (A86, Strath Mashie).

Figure 6.10 Presplit slope. The sharpness of the slope contrasts with the natural hillside and the slope is too steep and regular for vegetation to become established (A9, Drumochter).
Figure 6.11 Natural rock face in horizontally bedded rocks. Vegetation has established on all but the steepest areas of the face (Jedburgh).

Figure 6.12 Large slopes formed at the angle of the dominant discontinuity set. The outline of the slopes is being broken up by vegetation (A87, Kyle of Lochalsh).
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Appendix A  Potential Toppling Failure
A.1 Potential Toppling Failure Criteria and Stereographic Overlay

The general conditions for toppling failure geometry to be realised are set-out in Section 2.3 and are repeated here. Because of the potential variety and complicated nature of toppling failure geometries it should be emphasised that these conditions are general. **Toppling failures may occur outwith these conditions due to local peculiarities in geometry and/or the face profile.**

General Conditions for Toppling Failure:

- It requires at least two steep release planes and one basal release plane.
- The basal release can be either a single plane or the intersection of two planes.
- If the basal release is a single plane, it must dip out of the slope less than the rock's angle of friction.
- If the basal release is two planes, the intersection must dip out of the slope less than the rock's angle of friction.
- The intersection of the two steep planes must dip into the slope.
- “Toppling Wedges” are an example of a special case of toppling geometry. They are formed where the intersection of two discontinuities forming a “wedge” dip out of the face at less than angle of friction for wedge failure to occur. For a “toppling wedge”, there must be a single steep plane dipping into the slope, with an azimuth of approximately 160° to 200° relative to the slope azimuth. An example of a toppling wedge is illustrated in Figure A1.1.

![Figure A1.1 - An example of a toppling wedge failure.](image)

**Recognition of Potential Toppling Failure**

Stereographic overlays for toppling failure (Matheson, 1983) are general and do not cover all potential failure geometries. It is therefore possible to have toppling failure geometries outside of the traditional toppling failure overlay envelope.

Special cases of toppling failure outside the traditional toppling failure overlay envelope (for example toppling wedges) should be carefully considered during the recognition of
potential failure mechanisms. This highlights the importance of a rock slope inspection in the recognition of all potential failure mechanisms. Failures that are theoretically impossible according to the stereographic overlay envelopes can actually exist on the slope because of local peculiarities in discontinuity geometries and/or the face profile. This is partly a product of the assumption of the stereographic overlays that the “design” slope is planar, whereas in reality it is often irregular.

The overlay envelope, based on an equal-area projection for “classical” toppling failure, is shown in Figure A1.2.

![Figure A1.2 Overlay for Recognition of “Classical” Potential Toppling Failure.](image)
This report provides advice and guidance on good practice in road rock slope excavation. The subjects covered include rock slope stability, site investigation, rock slope design, rock slope excavation and environmental considerations.

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