4 Site investigation

> '… if you do not know what you should be looking for in a site investigation you are not likely to find much of value.'
>
> (Glossop, 1968)

This much-quoted quote is worth repeating because it sums up the philosophy of site investigation very well. Critical features need to be anticipated and looked for. Without care, the important details might be hidden within a pile of essentially irrelevant information. The difficulty and skill, of course, is in recognising what is critical.

4.1 Nature of site investigation

At any site, the ground conditions need to be assessed to enable safe and cost-effective design, construction and operation of civil engineering projects. This will generally include sub-surface ground investigation (GI), which needs to be focused on the particular project needs and unknowns. The requirements for GI will be very different for a tunnel compared to the design of foundations for a high-rise building or for stability assessment of a cut slope. There needs to be a preliminary review of the nature of the project, the constraints for construction and the uncertainties about the engineering geological conditions at the site. The British Code of Practice for Site Investigation, BS 5930 (BSI, 1999), sets out the objectives broadly as follows:

1. *Suitability*: to assess the general suitability of a site and its environs for the proposed works.
2. *Design*: to enable an adequate and economic design, including for temporary works.
3. *Construction*: To plan the best method of construction and, for some projects, to identify sources of suitable materials such as concrete aggregate and fill and to locate sites for disposal of waste.
4. *Effect of changes*: to consider ground and environmental changes on the works (e.g. intense rainfall and earthquakes) and to assess the impact of the works on adjacent properties and on the environment.
5. *Choice of site*: where appropriate, to identify alternative sites or to allow optimal planning of the works.

4.2 Scope and extent of ground investigation

4.2.1 *Scope and programme of investigation*

The scope of site investigation is set out in Box 4-1. This should include everything relevant to use of the site, including site history and long-term environmental hazards and not just geology. All authorities (e.g. AGS, 2006) agree that site investigation should, ideally, be carried out in stages, each building on the information gained at the previous stage, as outlined in Box 4-2. A preliminary engineering geological model should be developed for the site from desk study and field reconnaissance, as outlined in Chapter 3. That model should then be used to consider the project constraints and optimisation (e.g. the likely need for deep foundations or the best location for a dam) and for designing the first phase of GI. For a large project, this first phase is usually carried out during the conceptual phase. Further GI campaigns might be carried out for basic design, for detailed design and often additional works during construction. Engineering geologists should readily appreciate that all sites do not require the same level of ground investigation. Some have simple ground conditions, others more complex. At some locations, existing exposures will allow the broad geology to be assessed and reduce the need for GI. Projects may be situated in areas where the geology and ground conditions are already well understood. For example, if designing piles in London Clay, because of the wealth of published data and industry experience, GI requirements should be fairly routine[1] – little should be needed in the way of testing to determine parameters for design.

Taking this further, experience shows that the majority of sites worldwide do not have any particularly inherently hazardous conditions and might be categorised as *forgiving*. Even with no, or no competent investigation, the project is often completed without geotechnical difficulty. Such sites need little investigation – enough to establish that there are no particularly adverse hazards. In a review of the scope of ground investigations for foundation projects in the UK, Egan (2008) found that GI was either not conducted or was lacking borehole plans for 30% out of 221 projects, but he reported no adverse consequences. In other words, the engineers took a risk, perhaps on the basis of previous experience in an area, and apparently got away with it, although, as Egan points out, a ground investigation might have allowed more cost-effective solutions. Unfortunately, the world also has relatively rare *unforgiving* sites with inherently difficult geotechnical conditions that need careful and insightful investigation if problems are to be

[1] It does not follow that London Clay is without hazards for construction projects, for example, the Heathrow Express Tunnel collapsed during construction, as discussed in Chapter 7. De Freitas (2009) also provides a warning over geological variation through the London Clay stratum and argues that data banks of geotechnical properties need to be used with care from one area to another.

avoided. The big problem is identifying whether any particular site is unforgiving and in what way. It is the task of the engineering geologist, through his knowledge of geological processes, to anticipate hazardous geological conditions and to make sure that a GI is properly focused. A checklist approach to hazard prediction is advocated below.

Box 4-1 Overall scope of site investigation

1. Hazards and constraints during construction and in the longer term	– Previous site use – obstructions, contamination – Any history of mining or other underlying or adjacent projects (e.g. tunnels or pipelines) – Sensitive receivers – such as neighbours that might be affected by noise, dust, vibration and changes in water levels – Regulatory restrictions – Natural hazards, including flooding, wind, earthquakes, subsidence and landslides
2. Assess and record site characteristics	– Access constraints for investigation and construction – Need for traffic control, access for plant and waste disposal – Access to services – Site condition survey (partly as a record for any future dispute)
3. Geological profile at site	– Distribution and nature of soil and rock underlying the site, to an adequate degree, to allow safe and cost-effective design – Usually this will require a sub-surface ground investigation
4. Physical properties of soil and rock units and design parameters	Key parameters: – mass strength (to avoid failure) – deformability (to ensure movements are tolerable) – permeability (flow to and from site, response to rainfall and loading/unloading) Other factors: – chemical stability (e.g. reactivity in concrete, potential for dissolution) – potential for piping and collapse – abrasivity (sometimes a major consideration for construction)
5. Changes with time	– install instruments to check physical nature of the site – e.g. groundwater response to rainfall – install instruments to monitor settlement and effect on adjacent structures during construction – consider the potential for deterioration and need for maintenance

Box 4-2 Stages in a site investigation

Stage 1: Desk study at project conception stage

- Identification of key geological and environmental hazards at optional sites based on broad desk study and possibly site visits.
- Consider site constraints, engineering considerations and economic factors.

Stage 2: Detailed desk study and reconnaissance survey

- Collect and review all documents relevant to the preferred site, including topographic and geological maps, aerial and terrestrial photographs and any previous investigation reports. Review site history including previous building works and mining. Look for hazards such as landslides.
- Site mapping, possibly with advance contract allowing safe access, vegetation clearance and trial pits or trenches.

The Preliminary Ground Model

Develop a preliminary geological and geotechnical working ground model that can be used as a reference for the rest of the ground investigation.

This preliminary model should be used as a reference by all the team, including those logging boreholes and trial pits. The loggers need to know what to expect and to be able to identify anything that necessitates revisions to the ground model.

Site-specific ground investigation should be aimed at verifying the model, answering any unknowns and allowing design parameters to be derived.

Stage 3: Preliminary ground investigation linked to basic engineering design

- Consider use of geophysical techniques to investigate large areas and volumes.
- Preliminary boreholes designed to prove geological model (rather than design parameters).
- Instrumentation as appropriate (e.g. to establish groundwater conditions and seismicity).

Stage 4: Detailed ground investigation

- Further investigation to prepare detailed ground model and allow detailed design.
- *In situ* and laboratory testing to establish parameters.
- Detailed instrumentation and monitoring.

Stage 5: Construction

- Review of ground models during construction (including logging of excavations).
- Testing to confirm design parameters.
- Instrumentation to monitor behaviour and check performance against predictions.
- Revision to design as necessary.

Stage 6: Maintenance

- Ongoing review – e.g. of settlement, slope distortion, groundwater changes and other environmental impacts, possibly linked to a risk management system.

Typically, the cost of a site investigation is only a small part of the overall project cost (less than a few percent), yet clients often require some persuasion that the money will be well spent and might be especially reluctant to allow a staged approach because of the impact on programme. He might be reluctant to allow thinking and planning time as the GI data are received and especially unwilling to pay for a revised design as the ground models are developed and refined. Sometimes the engineer might adopt a fast-track approach whereby GI, design and construction are carried out concurrently, although this approach carries the risk that information gained later might impact on earlier parts of the design and even on constructed parts of the works. The programming can sometimes go awry, as on a site in Algeria where the author was trying to set out locations for drilling rigs in the same area as a contractor was preparing to construct foundations which obviously did not make sense. It turned out that design engineers had made assumptions about the ground conditions without waiting for GI, thinking that surface footings would be adequate. This proved incorrect and the design needed complete revision. In a similar manner to fast tracking, an observational approach is sometimes adopted, especially for tunnelling, whereby ground conditions are predicted, often on rather sparse data, and provisions made for change if and when ground conditions turn out to be different from those anticipated (Powderham, 1994). The observational method often relies on instrumentation of ground movements, measured loads in structural members, or water levels, whereby performance is checked against predictions. This can go seriously wrong where the ground behaves outside predictions – perhaps because the geological model is fundamentally incorrect or because instrument systems fail or are not reacted to quickly enough. Examples where instruments were not reacted to early enough include the Heathrow Express Tunnel (Muir Wood, 2000) and the Nicholl Highway collapse in Singapore (Hight, 2009); these are described in some detail in Chapter 7. An observational approach should also generally be adopted for rock slope construction, although it is seldom referred to as such. Basically, it is very difficult to characterise the complete rock fracture network from a few boreholes and therefore it is very important to check any design assumptions during construction and to be prepared to come up with different solutions for stabilisation as the rock is exposed and structures identified and mapped (see Box 1-1).

4.2.2 *Extent of ground investigation*

A large part of any site investigation budget will generally be taken up in sub-surface investigation and characterisation of the ground conditions (Items 3 and 4 in Box 4-1). Important questions are, how much ground investigation is required and how should it be done? There are no hard and fast rules, even though some authors try to provide guidance on the basis of site area or volume for particular types of operation (e.g. Figure 4.1 for dredging) or on hypothetical considerations (e.g. Jaksa *et al*., 2005). In reality, it depends upon the complexity of the geology at the site, how much is already known about the area, the nature of the project and cost. For sites with simple geology, the plan might be for boreholes at 10 m to 30 m spacing, for discrete structures like a building (BS 5939: 1999). For a linear structure like a road or railway project, the spacing might be anywhere between 30 and 300 m spacing, depending on perceived variability (Clayton *et al*., 1995). West *et al*. (1981) consider the particular difficulties in planning investigations for tunnels. So much depends upon the depth of tunnel, the topography and variability of geology. Often, considerable reliance is made on aerial photography interpretation, geological mapping, a few widely spaced preliminary boreholes and other boreholes targeted at particular perceived hazards such as faults that might be associated with poor quality rock and high water inflows. For example, Figure 4.2 shows the route of a planned tunnel in Hong Kong, with potential hazards identified, together with a rationale for their mitigation and additional GI. Where steeply dipping geological structures such as faults are anticipated, inclined boreholes may be required. Figure 4.3 shows an

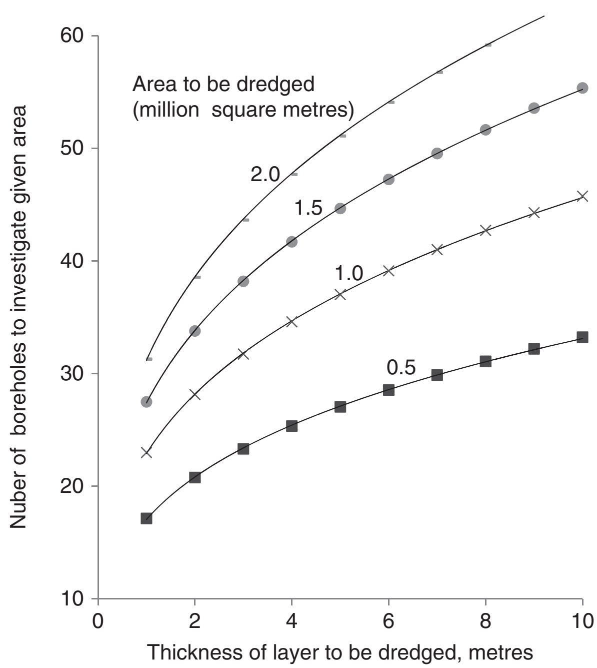


Figure 4.1 Number of boreholes for dredging area (in millions of square metres) vs. average thickness of material to be removed, based on equation of Bates (1981), as presented in PIANC (2000). Other factors that should be taken into account are variability of ground conditions and existing knowledge about the area.

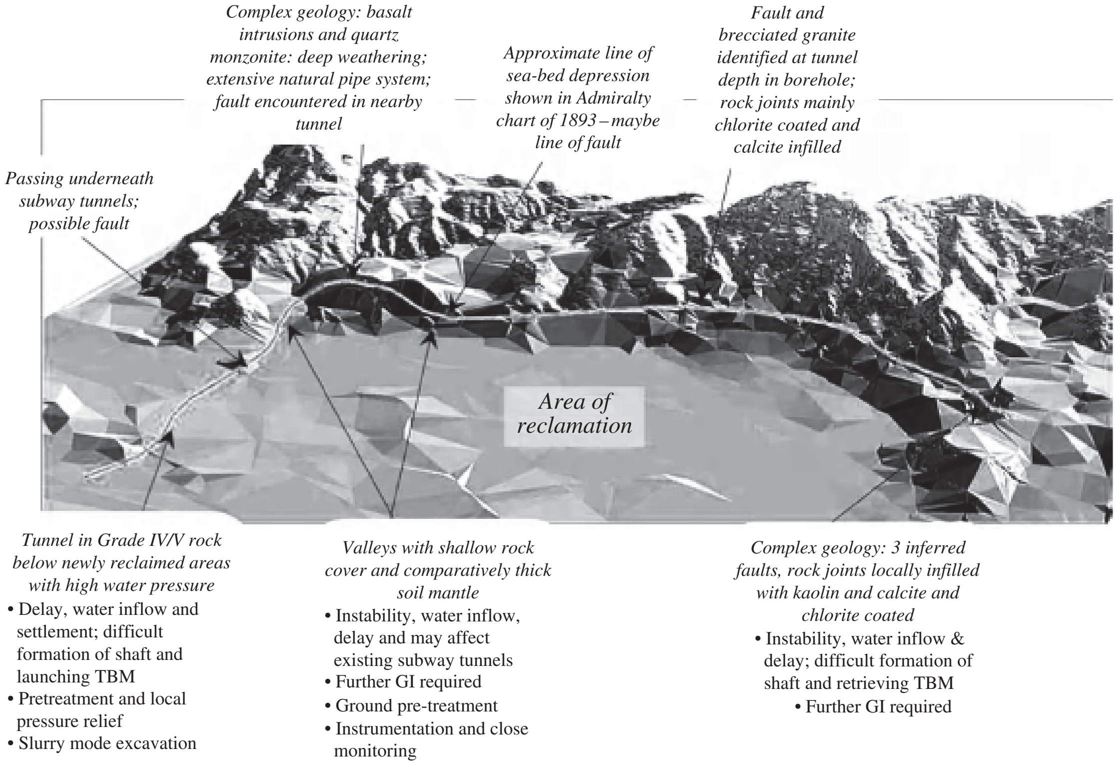


Figure 4.2 Preliminary assessment of ground investigation requirements for a new tunnel, Hong Kong.

assessment of possible conditions under the Eastern Tower of Stonecutters Bridge in Hong Kong at tender stage, based on desk study together with a proposed borehole investigation targeted at likely faults and zones of deep weathering. Some broad details of what was actually found are given in Fletcher (2004) and consequences by Tapley *et al.* (2006).

Requirements and practice for GI vary around the world. In Hong Kong, for example, it is normal practice to put down a borehole at the location of every bored pile (called a pre-drill). Elsewhere, a pattern of perhaps three, four or five boreholes might be adopted below each pile cap for a major structure. For example, for the 2nd Incheon Bridge in South Korea, opened in 2009, for each of the main cable stay bridge towers there were four boreholes per pile cap, each of which was about 70 by 25 m in plan and supported by 24 large-diameter bored piles. For the Busan-Geoje fixed link crossing, completed in 2010, also in South Korea, there were two cable-stay bridge sections, one with two towers and main span of 475 m, the other with three towers. The towers were founded on gravity caissons sitting

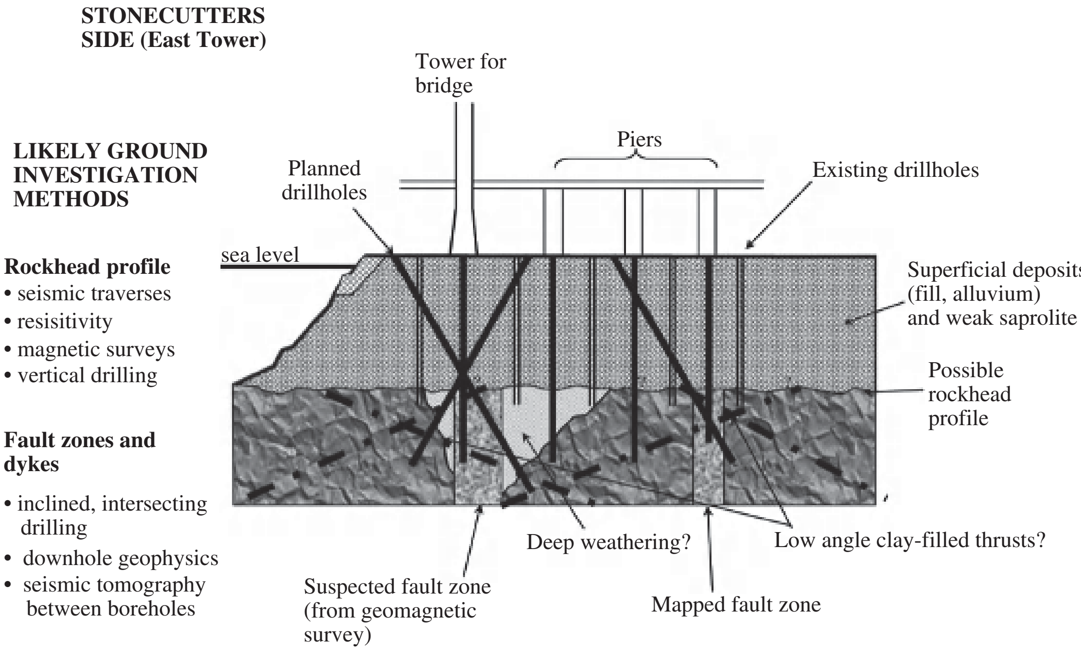


Figure 4.3 Preliminary assessment of ground conditions by Halcrow for East Tower of Stonecutters Bridge, Hong Kong, and need for inclined boreholes to investigate major fault structures.

on excavated rock (Chapter 6) and with plan dimensions of up to 40× 20m. For each of these foundations, there were usually about six boreholes, typically one put down at the conceptual stage, three for the basic design and two for detailed design. For most of the other viaduct piers with plan caisson dimensions of 17 × 17m, there were from one to five boreholes – less where the geology was better known, close to shore.

Obviously, where the site reconnaissance, together with desk study or findings from preliminary boreholes, indicate potentially complex and hazardous conditions, it may prove necessary to put down far more boreholes. For the design of the new South West Transport Corridor near Brisbane, Australia, the preliminary investigation over a critical section of more than 500 m comprised five boreholes and a few trial pits, mostly along the centre line of the road. As the earth works were approaching completion, minor landslides occurred at road level, together with some indications of deeper-seated movements. Over the next few months, an additional 70+ deep boreholes were put down, 56 trial pits and 54 inclinometers installed, despite almost 100% rock exposure in the cuttings (which was carefully examined and mapped). This intensive investigation allowed the landslide mechanisms to be identified in this very complex site and

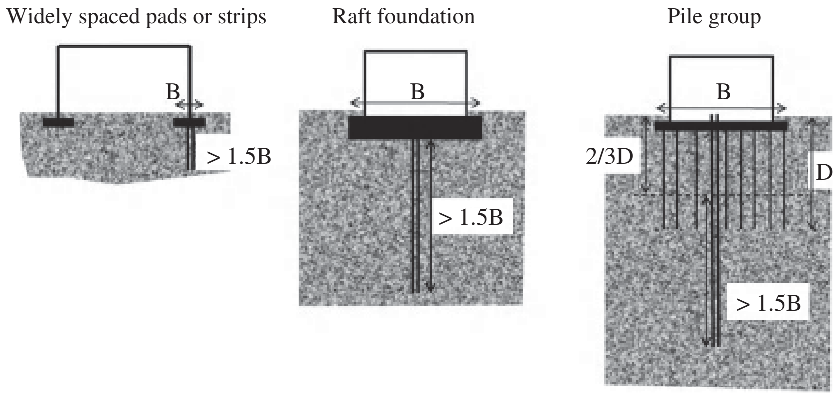


Figure 4.4 Criteria usually adopted for investigating the ground for foundations. Where geology is or may be complex, ground conditions might need to be proved to greater depth and several boreholes might be required. Similarly, these criteria do not apply or limit the need to consider particular site hazards, such as slope stability above or below the site.

remedial works to be implemented, which permitted the project to be completed on time (Starr *et al.*, 2010). In hindsight, the preliminary boreholes, which would have been more than adequate for a normal stretch of road, gave no indication of the degree of difficulty and complexity at this unforgiving site, which only became clear following intensive work involving a wide range of experts. In a similar manner, the landslide at Pos Selim, Malaysia, described in Chapter 7, could not have been anticipated from a few boreholes. The mechanism was at a very large scale and involved too many components to have been understood before the major displacements occurred.

As a general rule, at any site, at least one borehole should be put down to prove ground conditions to a depth far greater than the depth of ground to be stressed significantly by the works. Generally, for foundations, at least one borehole should be taken to at least 1.5 times the breadth (B) of the foundation (Figure 4.4). For pile groups, it is generally assumed that there is an equivalent raft at a depth of 2/3D where D is the length of piles and the ground should be proved to at least 1.5B below that level. This is only a general guideline – if there is any reason to suspect more variable conditions and, where the geology is non-uniform, one borehole will probably not be enough (Figure 4.5). Poulos (2005) discusses the consequences of 'geological imperfections' on pile design and performance. Boreholes are often terminated once rock has been proved to at least 5m, but this may be inadequate to prove bedrock in weathered terrain (Hencher & McNicholl, 1995). Whether or not one has reached *in situ* bedrock might be established by geological interpretation of consistent rock fabric or structure across a site, but elsewhere it may be more difficult, in which case it

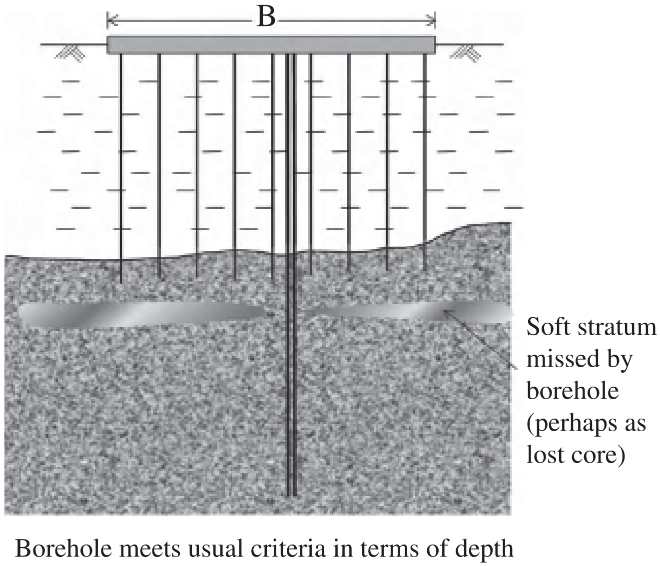


Figure 4.5 Example of situations where a single borehole (or few boreholes) might miss important information that will affect the integrity of the structure.

is best to take one or more boreholes even deeper if important to the design.

4.3 Procedures for site investigation

4.3.1 General

Guidance on procedures and methodologies for site investigation is given for the UK by Clayton *et al.* (1995) and for the USA and more broadly by Hunt (2005). The British Code of Practice for Site Investigations, BS 5930 (BSI, 1999), provides comprehensive advice on procedures and techniques and for soil and rock description for the UK. Other codes exist for different countries (e.g. Australia, China and New Zealand). Generally, there is consistent advice over the overall approach to site investigation, although terminology and recommended techniques differ. All agree, however, that the first step should be a comprehensive review of all available maps and documents pertaining to a site – this is called a desk study.

4.3.2 Desk study

4.3.2.1 Sources of information

For any site, it is important to conduct a thorough document search. This should include topographic and geological maps. Hazard maps are sometimes available. These include broad seismic zoning maps for countries linked to seismic design codes. In some countries, there are also local seismic micro-zoning maps showing locations of active faults and hazards such as liquefaction susceptibility. Sources of information for the UK are given in BS 5930 (BSI, 1999) and Clayton *et al*., 1995. The Association of Geotechnical and Geoenvironmental Specialists (AGS), whose contact details are given in Appendix A, also give useful advice and sources of reference. Records of historical mining activity and previous land use are especially important. In the UK, the British Geological Survey (BGS) has made available a digital atlas of hazards, including mining (but not coal), collapsible materials, swelling and compressible soils, landslides and noxious gas. Landslide hazard maps are published in the USA for southwest California and in Hong Kong, as discussed below.

4.3.2.2 Air photograph interpretation

Air photographs can be extremely useful for examining sites. Pairs of overlapping photographs can be examined in 3D using stereographic viewers, and skilled operators can provide many insights into the geology and geomorphological conditions (Allum, 1966; Dumbleton & West, 1970). Historical sets of photographs help to reveal the site development and to assess the risk from natural hazards such as landslides. In Hong Kong, it is normal practice to set out the site history for any new project through air photo interpretation (API) of sets of photos dating back to the 1920s. The role of API in helping to assess the ground conditions at a site is illustrated in Box 4-3.

Box 4-3 Role of air photo interpretation (API)

Overlapping air photos allows a skilled earth scientist to examine the site topography in three dimensions. According to Styles (personal communication), in order to do it well you must put yourself on the ground mentally and walk across the terrain looking around in oblique perspective. Topographic expression and other features such as the presence of boulders, hummocky ground, arcuate steps and vegetation, can be interpreted in terms of terrain components and geomorphological development: landslide morphology, degree of weathering, and distribution of superficial deposits such as colluvium and alluvium. Broad geological structure such as major joint systems, faults and folds, may be observed, interpreted and measured in a way that would be more difficult working only by mapping exposures on the ground (Figure B4-3.2).

Where landslides are identified on photographs, debris run-out can be measured, which may help in assessing the degree of risk for existing and future developments. River channels can be traced and catchments measured. Where a series of historical photographs is available, an inventory of landslide events can be compiled and related to historical rainfall records. Anthropogenic development and use of sites can be documented.

It is important that API is checked by examination in the field and this is known as ground truthing, which is an integral part of site reconnaissance and field mapping. Similarly, interpreted site history should be checked and correlated against other documentary evidence such as old maps and photographs. The preliminary ground model developed from API and field studies can then be investigated further by trial pits and boreholes, as necessary. Conversely, a ground investigation in an area of variable topography, without prior API, reconnaissance and desk study, may be ineffective and poorly focused. An introduction to the use of air photographs, with particular consideration of landslide investigations, is given by Ho *et al.* (2006).

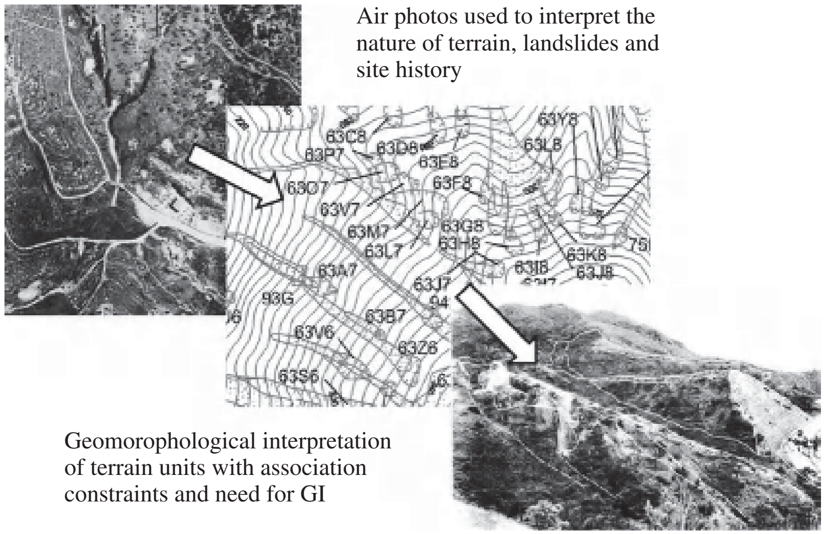


Figure B4-3.1 Process of API. Pairs of overlapping photographs can be examined stereographically to give a 3D image. Major terrain features can be identified and if historical series of photographs are available, then land development and site history can be ascertained, in this example, in terms of landslide history. In the second image above, interpreted landslides have been mapped (with date of the photo in which the landslide is first seen). These interpretations can then be checked in the field (Devonald *et al.*, 2009).
In addition, terrain can be split into units on the basis of surface expression, underlying geology, activity and vegetation, as described by Burnett *et al.* (1985). Third photo and overlay provided by K. Styles.

Figure B4-3.2 Major structural lineations visible in aerial photograph and controlling river development of Zambezi River above Victoria Falls between Zimbabwe and Zambia.

Figure 4.6 Route of Ching Cheung Road, Hong Kong, superimposed on 1949 aerial photograph (after Hencher, 1983c; Hudson & Hencher, 1984).

Even with little training, the importance of air photographs can be immediately clear, as in Figure 4.6, which is an air photograph from 1949 on to which has been marked the route of the Ching Cheung Road in Hong Kong, constructed in 1963. Various ground hazards are evident in the photo (landslides and deep gulleying) and it is no surprise that these led to later problems with the road, as addressed in Chapter 7 and discussed by Hudson & Hencher (1984).

Systematic interpretation of air photographs for determining geotechnical hazards has been carried out in several countries. For example, the whole of Hong Kong was mapped, in terms of perceived geotechnical hazard, from air photographs in the 1980s at a 1:20,000 scale and locally at 1:2,500 and, whilst never intended for site-specific interpretation, these were very useful for urban planning (Burnett *et al.*, 1985; Styles & Hansen, 1989). Air photos can be used for detailed measurement by those trained to do so. Figure 4.7 shows displacement vectors for the slow-moving rock landslide at Pos Selim, Malaysia. The 3D image was prepared from as-built drawings and oblique air photographs taken from a helicopter and linked to surveyed control points. The vectors produced (up to 15 m drop in the rear scarp) are considered accurate to about 0.2 m. Topographic surveys can also be carried out using terrestrial or airborne LIDAR surveys and these can be repeated to monitor ongoing movements in landslides or in volcanic eruptions (e.g. Jones, 2006). In some situations, especially for remote sites lacking good air photo coverage, satellite images may be helpful, although often the scale is not large enough to provide the detailed interpretation required and stereo imagery is impossible – unlike for purpose-flown aerial photograph sequences. Use of false spectral

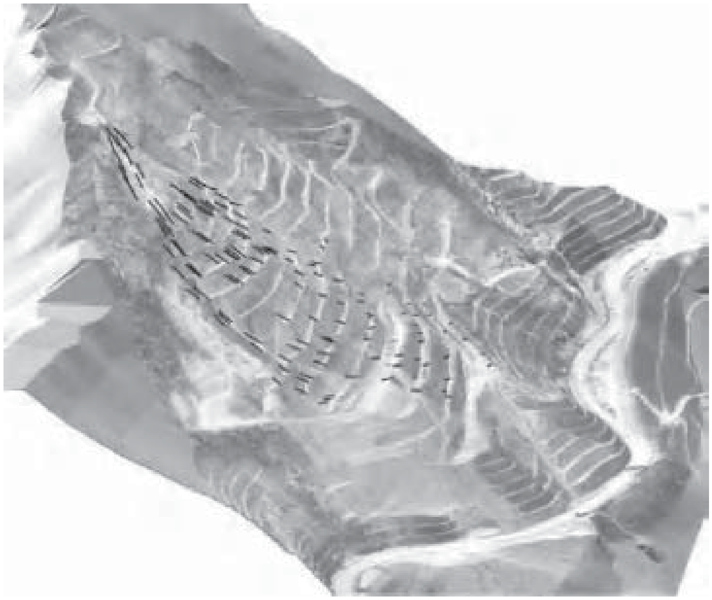


Figure 4.7
Visualisation of Pos Selim landslide, Malaysia, showing displacement vectors over a two-year period (after Malone *et al*., 2008).

images such as infra-red can help interpretation, for example, of vegetation and seepage.

4.3.3 Planning a ground investigation

BS 5930 and most textbooks on site investigation provide good information on techniques and procedures but little advice on how to plan a ground investigation or on how to separate and characterise geotechnical units within a geological model. They also say little about how to anticipate hazards, which is a key task for the engineering geologist. It is important to take a holistic view of the geological and hydrogeological setting – the 'total geological model' approach of Fookes *et al*. (2000), as discussed in Chapter 3 – but the geological data need to be prioritised to identify what is really important to the project and to obtain the relevant parameters for safe design.

The problem is that there are so many things that might potentially go wrong at sites and with alternatives for cost-effective design that it is sometimes difficult to know where to start in collecting information. One might hope that simply by following a code of practice, that would be enough, but, in practice, the critical detail may be overshadowed by relatively irrelevant information collected following routine drilling and logging methodology.

One approach that can be useful for planning and reviewing data from a ground investigation, and focusing on critical information, is to consider the different aspects of the site and how they might affect the project in a checklist manner (Knill, 1976, 2002; Hencher & Daughton, 2000; Hencher, 2007). The various components and aspects of the project and how different site conditions might affect its success are considered one by one and in an integrated way. This is similar to the rock engineering systems methodology of Hudson (1992), in which the various parameters of a project are set out and their influence judged and measured in a relative way (Hudson & Harrison, 1992). This is also akin to the concept of a risk register for a civil engineering project at the design and construction stages, whereby each potential hazard and its consequence is identified and plans made for how those risks might be mitigated and managed. This is addressed in Chapter 6.

The three verbal equations of Knill (1976) are set out in Table 4.1. The first part is to consider geological factors: material and mass strengths and other properties. The second is to assess the influence of environmental factors such as *in situ* stress, water and earthquakes. The final consideration is how these factors affect, and are affected by the construction works. A very similar process has been proposed for addressing risk by Pöschl & Kleberger (2004), particularly for tunnels.

4.3.3.1 Equation 1: geological factors

The first equation encourages the investigator to consider the ground profile (geology) and its properties at both the material and mass scales.

Table 4.1 Engineering geology expressed as three verbal equations (after Knill, 1976).

Equation 1 GEOLOGY **MATERIAL PROPERTIES + MASS FABRIC ⇒ MASS PROPERTIES**
The first equation includes the geology of the site and concerns the physical, chemical and engineering properties of the ground at small and large scales. It essentially constitutes the soil and rock ground conditions.
Equation 2 + ENVIRONMENT **MASS PROPERTIES + ENVIRONMENT ⇒ ENGINEERING GEOLOGICAL SITUATION**
The second equation relates to the geological setting within the environment. Environmental factors include climatic influences, groundwater, stress, time and natural hazards.
Equation 3 + CONSTRUCTION **ENGINEERING GEOLOGICAL SITUATION + INFLUENCE OF ENGINEERING WORKS ⇒ ENGINEERING BEHAVIOUR OF GROUND.**
The third equation relates to changes caused by the engineering works such as loading, unloading and changes to the groundwater levels. It is the job of the engineer to ensure that the changes are within acceptable limits.

MATERIAL SCALE

The material scale is that of the intact soil and rock making up the site. It is also the scale of laboratory testing, which is usually the source of engineering parameters for design. Typical factors to review are given in Table 4.2. They include the chemistry, density and strength of the various geological materials and contained fluids making up the geological profile. Hazards might include adverse chemical attack on foundations or ground anchors, liquefaction during an earthquake, swelling or low shear strength due to the presence of smectite clays, abrasivity or potential for piping failure. Inherent site hazards associated with geology include harmful minerals such as asbestos and erionite. Granitic areas, phosphates, shale and old mine tailings are sometimes linked to relatively high levels of radon gas, which is estimated to cause between 1,000 and 2,000 deaths each year in the UK (Health Protection Agency). Talbot *et al.* (1997) describe investigations for radon during tunnelling. Gas hazards are especially important considerations for tunnelling and mining but are also an issue for completed structures, as illustrated by the Abbeystead disaster of 1984 when methane that migrated from coal-bearing strata accumulated in a valve house and exploded killing 16 people (Health and Safety Executive, 1985). These are all material-scale factors linked to the geological nature of the rocks at a site.

Locating sources for aggregate, armourstone and other building materials is often a task for an engineering geologist. Other than the obvious considerations of ensuring adequate reserves and cost, one must consider durability and reactivity, and this will involve geological characterisation and probably testing. Two examples in Chapter 7 relate how adverse material properties of sourced fill and aggregate material led to severe consequences. At Carsington Dam, UK, a

Table 4.2 Examples of material-scale factors that should be considered for a project

FACTOR	CONSIDERATIONS	EXAMPLES OF ROCK TYPES/SITUATIONS
mineral hardness	abrasivity, damage to drilling equipment	silica-rich rocks and soils (e.g. quartzite, flint, chert)
mineral chemistry	reaction in concrete	olivine, high temperature quartz, etc.
	oxidation – acids	pyrites
	swelling, squeezing dissolution	mudrocks, salts, limestone
	low friction	clay-infilled discontinuities, chlorite coating
loose, open texture	collapse on disturbance or overloading, liquefaction, piping, low shear strength	poorly cemented sandstone, completely weathered rocks (V); loess; quick clays

chemical reaction was set up between the various rocks used to construct the dam, which resulted in acid pollution of river courses and the production of hazardous gas, with the death of two workers; at Pracana Dam, Portugal, the use of reactive aggregate led to rapid deterioration of the concrete. The latter phenomenon has been reported from many locations around the world and is associated with a variety of minerals, including cryptocrystalline silica (some types of flint), high-temperature quartz, opal and rock types ranging from greywacke to andesite. Details of how to investigate whether aggregate may be reactive and actions to take are given in RILEM (2003).

MASS SCALE

Mass-scale factors include the distribution of different materials in different weathering zones or structural regimes, as successive strata or as intrusions. It includes structural geological features such as folds, faults, unconformities and joints (Table 4.3). Discontinuities very commonly control the mechanical behaviour of rock masses and some soils. They strongly influence strength, deformability and hydraulic conductivity.

Table 4.3 Examples of mass-scale factors that should be considered for a project

FACTOR	CONSIDERATIONS	EXAMPLES OF ROCK TYPES SITUATIONS
lithological heterogeneity	difficulty in establishing engineering properties, construction problems (plant and methodology)	colluvium, un-engineered fill, interbedded strong and weak strata, soft ground with hard corestones
joints/natural fractures	sliding or toppling of blocks, deformation, water inflows, leakage/migration of radioactive fluids	slopes, foundations, tunnels and reservoirs, nuclear repository
faults	as joints, sudden changes in conditions, displacement, dynamic loads	tunnels, foundations, seismically active areas
structural boundaries, folds, intrusions	heterogeneity, local stress concentrations, changes in permeability – water inflows	all rocks/soils
weathering (mass scale)	mass weakening; heterogeneity (hard in soft matrix), local water inflow, unloading fractures	all rocks and soils close to Earth's surface, especially in tropical zones; ravelling in disintegrated rock masses
hydrothermal alteration	as weathering, low strength and prone to collapse especially below water table	generally for igneous rocks especially near contacts

One of the main geological hazards to engineering projects at the mass scale is faults. Faults can be associated with zones of fractured and weathered material, high permeability and earthquakes. Alternatively, faults can be tight, cemented and actually act as barriers to flow, as natural dams rather than zones of high permeability. Faults should always be looked for and their influence considered. There are many cases of unwary constructers building on or across faults, with severe consequences, sometimes leading to delays to projects or a need for redesign. Consequence is sometimes difficult to predict but should be considered and investigated. Other examples of mass factors that would significantly affect projects include boulders in otherwise weak soil, which might preclude the use of driven piles or would comprise a hazard on a steep slope.

An example of where a formal review of the potential for large-scale structural control might have helped is provided by the investigation for a potential nuclear waste repository at Sellafield in the UK, as explained in Box 4-4. It appears that early boreholes and tests did not sample relatively widely spaced master joints within the stratum and, therefore, an incomplete picture was formed of the factors controlling mass permeability. In hindsight, the true nature of the rock might have been anticipated by desk study and field reconnaissance of exposures.

Box 4-4 Anticipating mass characteristics: the Brockram and the Sellafield Investigations

The UK Government specification for acceptable risk from any nuclear waste repository was set to be extremely onerous and necessitated intensive investigation combined with intensive modelling. Ground investigation has been conducted at Sellafield, Cumbria, since 1989, aimed at determining whether or not the site is suitable as a repository for radioactive waste. The target host rock is the Borrowdale Volcanics at a depth of more than 500 m. Part of the modelling has involved trying to predict groundwater flow and the movement of radio-nucleides. For this, a good ground model was necessary with estimates of permeability for the full rock sequence. Several high-quality boreholes have been put down at the site and logged very carefully. A general model has been developed, as illustrated in Figure B4-4.1 (ENE is to the right). The geological model has the Borrowdale Volcanics, which contain saline water, separated from the overlying sandstones, containing fresh water, by a bed called the Brockram, which is typically 25–100 m in thickness and cut by faults. The Brockram and associated evaporites and shale further west evidently play a very important potential role as a barrier to flow of groundwater (flow into the repository) and, hence, radio-nucleides migrating away from the repository.

Early modelling

For most early numerical simulations, the Brockram was modelled with very low conductivity (2×10^{-10} to 1×10^{-9} m/s), based largely on borehole tests and 'expert elucidation' (Heathcote *et al.*,

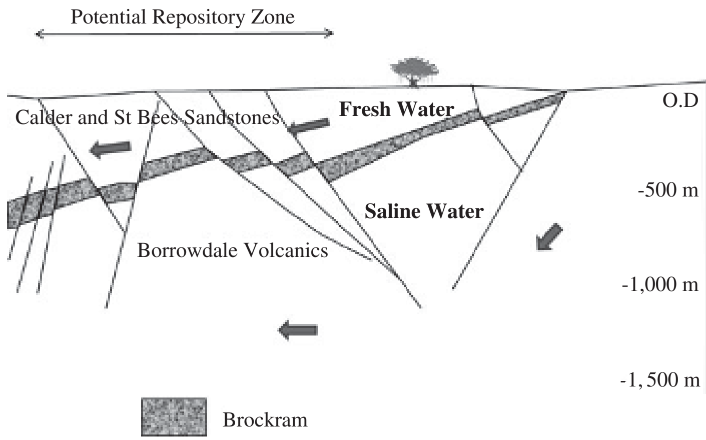


Figure B4-4.1 Cross section across the potential repository zone, showing basic geology and directions of flow (modified from Chaplow, 1996).

1996). These values are similar to those measured for the Borrowdale Volcanics – 50% measured over 50m lengths, with conductivity < 1×10^{-10} m/s, according to Chaplow (1996).

Later tests

At a later stage, field tests were carried out that yielded 'significant flows' in the Brockram, and the earlier modelling had to be revised. Michie (1996) reports hydraulic conductivity measurements within the Brockram with a maximum of 1×10^{-5} m/s, i.e. four orders of magnitude higher than adopted for the early models.

A surprise?

The changed perception for this important stratum might be considered just part of what is to be expected in any progressive ground investigation. However, the potential for locally high permeability associated with extremely widely spaced and persistent joints, at spacing such that they will be rarely sampled in boreholes, could have been anticipated, partly because such joints can be observed directly at exposures in the Lake District. At Hoff's Quarry to the east of the Lake District, the rock can be examined, and at a material scale a low permeability would be anticipated (Figure B4-4.2).

However, at a larger scale, the rock at Hoff can be seen cut by near-vertical master joints which would affect the mass permeability in a dramatic way (as evidenced from the Sellafield test).There were also indications from the literature that the Brockram might be permeable at a scale of hundreds of metres. For example Trotter *et al.* (1937) commented on the possibility of pathways through the Brockram, with reference to the distribution of haematite mines within the Carboniferous Limestone underlying the Brockram.

Lessons: it is very important not simply to rely on site-specific data when elucidating parameters for design. There is a need to consider the geological setting, origins and history – with all that entails – as per the 'total geological approach' advocated by Fookes *et al.* (2000). Furthermore, when looking at data from boreholes, especially ones with a strong directional bias, one should consider all the field evidence that might offer some clues as to the validity of the expert elucidation process.

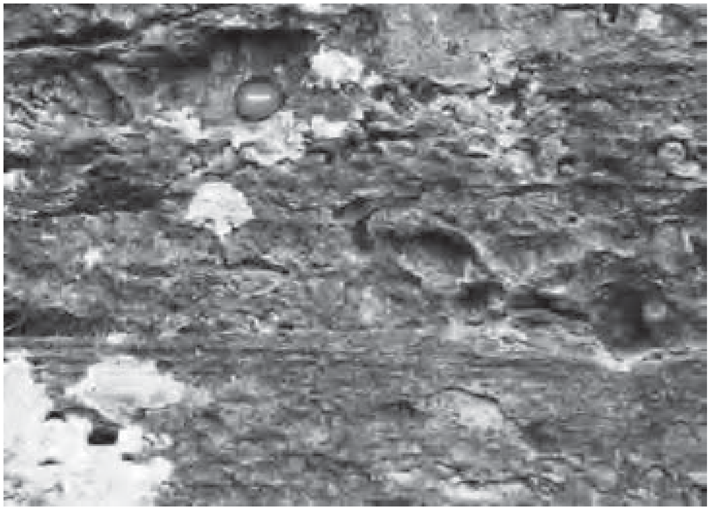


Figure B4-4.2 Close up of Brockram rock, at Hoff's Quarry, Vale of Eden, UK. The rock is a cross-bedded, limestone-rich, well-cemented breccia. It contains fossiliferous blocks of Carboniferous Limestone as well as more rare rocks such as Whin Sill dolerite. It has the appearance of a wadi-type deposit – poorly sorted, probably rapidly deposited by flash floods. From field assessment, it has a low permeability at the material scale. Lens cap (58 mm) for scale.

Figure B4-4.3 More distant view of Brockram at Hoff's Quarry. Note the fully persistent, near-vertical master joints about 40 m apart, which will control mass permeability. Evidently joints from this set would only be intersected using inclined rather than vertical boreholes.

4.3.3.2 *Equation 2: environmental factors*

Environmental factors, some of which are listed in Table 4.4, including hydrogeological conditions, should be considered part of the ground model for a site, but are best reviewed separately from the basic geology, although the two are closely interrelated. The environmental factors to be accounted for depend largely on the nature, sensitivity and design life of structures and the consequence of failure. It is usual practice to design structures to some return period criterion such as a 1 in 100 year storm or 1 in 1,000 year earthquake, the parameters for which are determined statistically through historical review. In some cases, engineers will also want to know the largest magnitude event that might occur, given the location of the site and the geological situation. Then some thought can be given as to whether or not it is possible to make some provision for that maximum credible event. For earthquakes, for example, a structure might be designed to behave

Table 4.4 Examples of environmental factors that should be considered for a project

FACTOR	CONSIDERATIONS	EXAMPLES OF ROCK TYPES SITUATIONS
in situ stresses	high stress: squeezing, overstressing, rockbursts	mountain slopes and at depth, shield areas, seismically active areas
	low stress: open fractures, high inflows, roof collapse in tunnels	extensional tectonic zones, unloaded zones, hillside ridges
natural gases	methane, radon	coal measures, granite, black shales
seismicity	design loading, liquefaction, landslides	seismically active zones, high consequence situation in low seismic zones
influenced by man	unexpectedly weak rocks, collapse structures	undermined areas
	gases and leachate	landfills, industrial areas
groundwater chemistry	chemical attack on anchors/nails foundations/materials	acidic groundwater, salt water
groundwater pressure	effective stress, head driving inflow, settlement if drawn down	all soils and rocks
ice	ground heave, special problems in permafrost/tundra areas, freeze-thaw jacking and disintegration	anywhere out of tropics
biogenic factors	physical weathering by vegetation, rotted roots leading to piping, insect attack	near-surface slopes weathered rocks causing tree collapse

elastically (without permanent damage) for a 1 in 1,000 year event but for a, very unlikely, maximum credible event, some degree of damage would be accepted.

The factors to review at this stage include natural hazards such as earthquake loading, strong winds, heavy rain and high groundwater pressures or flooding. Anthropogenic factors to consider include industrial contamination and proximity of other structures and any constraints that they may impose.

4.3.3.3 Equation 3: construction-related factors

The third verbal equation of Knill & Price (Knill, 2002) addresses the interaction between the geological and environmental conditions at a site and the construction and operation constraints (Hencher & Daughton, 2000). Excavation will always give rise to changes in stresses, and the ground may need to be supported. Excavations may also result in changes in groundwater, and the consequences need to be addressed and mitigated if potentially harmful. Similarly, loading from structures has to be thought through, not only because of deformations but also because of potentially raising water pressures, albeit temporarily.

There will also be hazards associated specifically with the way the project is to be carried out. For example, a drill and blast tunnel is very different to one excavated by a tunnel boring machine and will have specific ground hazards associated with its construction (Chapter 6). Similarly, the construction constraints are very different for bored piling compared to driven piles (Table 4.5). The systematic review and investigation of site geology and environmental factors, discussed earlier, needs to be conducted with specific reference to the project at hand. This will hopefully allow the key hazards to be identified and design to be robust yet cost-effective. Nevertheless, models are always simplifications, and the engineer must adopt a cautious and robust approach when designing, especially where the geological conditions are potentially variable and where that variability might cause difficulties, as illustrated by the case of a tunnel failure reported in Chapter 7 (Grose & Benton, 2005).

Table 4.5 Examples of the influence of engineering works

FACTOR	CONSIDERATIONS
loading/unloading – static/dynamic	settlement, failure, opening of joints, increased permeability in cut slopes, blast vibrations
change in water table	increased or decreased pressure head, change in effective stress, drawdown leading to settlement, induced seismicity from reservoir loading
denudation or land clearance	increased infiltration, erosion, landsliding

4.3.3.4 Discussion

It is evident that site investigation cannot provide a fully detailed picture of the ground conditions to be faced. This is particularly true for tunnelling, because of the length of ground to be traversed, the volume of rock to be excavated and often the nature of the terrain, which prevents boreholes being put down to tunnel level or makes their cost unjustifiable. Instead, reliance must be placed on engineering geological interpretation of available information, prediction on the basis of known geological relationships and careful interpolation and extrapolation of data by experienced practitioners. Factors crucial to the success of the operation, need to be judged and consideration given to the question: *what if*? It is generally too late to introduce major changes to the methods of working, support measures, etc. at the construction stage, without serious cost implications.

Site investigation must be targeted at establishing those factors that are important to the project and not to waste money and time investigating and testing aspects that can be readily estimated to an acceptable level or aspects that are simply irrelevant. This requires a careful review of geotechnical hazards, as advocated above. Even then, one must remain wary of the unknowns and consider ways in which residual risks can be investigated further and mitigated, perhaps during construction, as addressed in Chapter 6.

There is a somewhat unhealthy belief that standardisation (for example, using British Standards, Eurocodes, Geoguides and ISRM Standard Methods) will provide protection against ground condition hazards. Whilst most standards certainly encompass and encourage good practice, they often do so in a generic way that may not always be appropriate to the project at hand and they may not provide specific advice for coping with a particular situation. Ground investigations are often designed on the basis of some kind of norm – a one-size-fits-all approach to ground investigation. It is imagined that a certain number of boreholes and tests will suffice for a particular project, essentially irrespective of the actual ground conditions at the site. This ignores the fact that ground investigations of average scope are probably unnecessary for many sites but will fail to identify the actual ground condition hazards at rare, but less forgiving sites. Similarly, an averaging-type approach will mean that many irrelevant and unnecessary samples are taken and tested whilst the most important aspects of a site are perhaps missed or poorly appreciated. This is, unfortunately, commonplace.

If the hazards are considered in a systematic way, as discussed earlier, then the risks can be thought through fully and this will help the ground investigation to be better focused. The process is illustrated in Box 4-5 for a hydroelectric scheme involving the construction of a dam, reservoir, power station and associated infrastructure.

Box 4-5 Planning a site investigation for a new hydroelectric scheme

Project concept: high arch dam with high-pressure penstock tunnels (120m hydraulic head) leading to underground power house, tailrace tunnels and surge chamber. Structures to be considered include reservoir, ancillary buildings, roads, power lines and diversion tunnel. Sources of concrete aggregate need to be identified, as well as locations for disposing construction waste.

General setting: valley with narrowing point suitable for arch dam (high stresses). Topography and hydrology adequate for reservoir capacity. Steep slopes above reservoir.

Geology from preliminary desk study: major fault along valley, maybe more. Right abutment (looking downstream) in granitic rock, sometimes deeply weathered. Left side, ancient schist, greywacke, mudstone and some limestone. Folded and faulted with many joints. Alluvial sediments along valley.

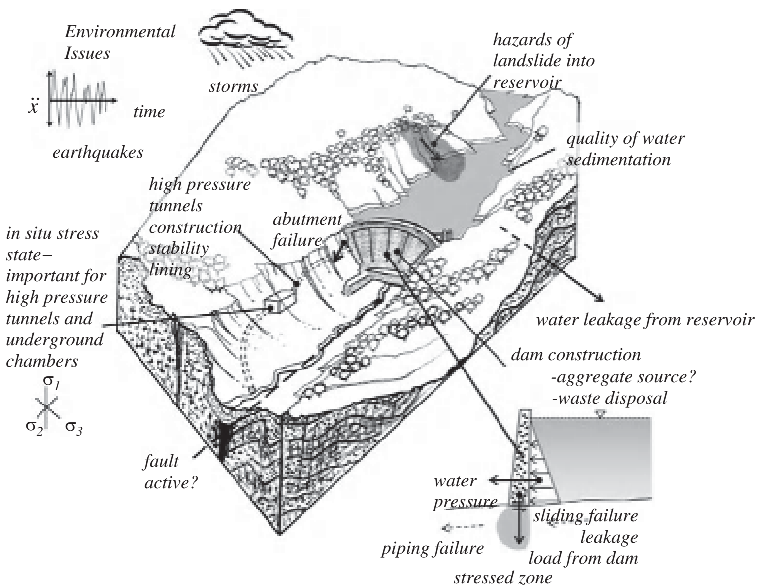


Figure B4-5.1 Schematic model of site for new hydroelectric scheme with some of the most important hazards that need to be quantified during the site investigation.

Key issues for investigation:

Dam: stability of foundations and abutments, settlement, leakage, overtopping from landslide into reservoir, silting up.

Tunnels and powerhouse: rock quality, *in situ* stress state, construction method, stability, lining and support requirements.

Reservoir: leakage, siltation, water quality.

Construction: source of aggregate, waste disposal, access, river diversion.

Main geotechnical considerations when conducting site investigation

Issues	*Equation 1*		*Equation 2*	*Equation 3*
	Geology		*Environment*	*Construction*
	Material	*Mass*		
Arch dam stability and construction	Strength, deformability and durability of foundation materials, including highly stressed abutments	Geological profile, depth to bedrock Presence of discontinuities, allowing failure in abutments or sliding failure below dam Fault reactivation	Seismicity Water pressure in foundations and abutments Check history of mining	Adequate source of non-reactive aggregate Waste disposal locations (fill embankments)
Leakage below dam and from reservoir	Permeability (need for grouting/cut offs) Potential for piping	Leakage on main fault and other faults/weathered zones Limestone might be karstic	Groundwater profile in surrounding terrain Existing throughflow paths	Options for grouting and/or cut-off structures
Landslides into reservoir	Material strength	Adverse discontinuities, aquitards causing perched water pressure to develop Landslide history	Response of groundwater to storms and to lowering of water in reservoir Seismic loading	Need for stabilisation such as drainage or option to remove hazardous ground
Powerhouse and high-pressure tunnels	Rock strength Abrasivity for tunnel equipment	Fracturing (rock mass classification allows judgement of stabilisation required) Weathered zones	*In situ* stress state (potential squeezing or leakage and need for steel liners) Groundwater pressure and permeability (inflows or water loss for operating tunnels)	Method of excavation Ground movement due to excavation Blasting vibration Groundwater changes

4.4 Field reconnaissance and mapping

4.4.1 General

At many sites, geologists can get a great deal from examining the landscape, mapping and interpolating information from exposures, and this is one of the most important aspects of geological education and training. This, together with desk study information, should allow preliminary ground models to be developed, which can then be used to form the basis for planning any necessary ground investigation. The preliminary model should allow an initial layout of the components of the project and, for buildings, some insight into the types of foundation that might be required. For tunnels, decisions can be made on locations for portals and access shafts. The degree to which walk-over studies and field mapping can be cost-effective is often overlooked, as illustrated by a case example in Box 4-6.

Box 4-6 Case example: cost-effectiveness of site reconnaissance – bridge abutment, Lake District, UK

The first ground investigation that the author was involved with was for a bridge abutment in the Lake District, UK. Figure B4-6.1 is a view of the rock cliff that was to form the abutment, and halfway down the cliff is a platform. Figure B4-6.2 is a side view of the platform. The man in the middle of the photograph is logging a borehole, using a periscope that has been inserted into a hole, inclined at about 45 degrees, drilled into the rock from the same platform. In the foreground, rock can be seen with a fabric dipping roughly parallel to the cliff. For reasons that are unimportant now, a question arose regarding the geological structure being logged by the periscope.

The site engineer was asked for his geological map of the rock along the river (including the 100% exposed cliff). He replied, 'what map?'

Figure B4-6.1 Drilling platform on cliff.

Figure B4-6.2 Borehole periscope in use.

There we were, perched on a precarious and extremely expensive platform. A drilling rig had been brought in and lowered down the cliff to drill an inclined borehole of perhaps 73 mm diameter, at great cost, and we had been brought to the site from London to log the hole using a periscope. Meanwhile, the full rock exposure was available to be mapped and interpreted at very little cost, which would have allowed a much better and more reliable interpretation of the geological structure than was possible from a single borehole.

Lesson: Use the freely available information first (desk study and walk-over/mapping) before deciding on what ground investigation is necessary at a site.

Mapping can be done in the traditional geological manner, using base maps and plans, or on air photographs, which may need to be rectified for scale. Observations such as spring lines (Figure 4.8) are not only important in delineating probable geological boundaries but also in their own right for hydrogeological modelling. Observation points can be marked in the field, to be picked up accurately later by surveyors. Alternatively, locations can be recorded by GPS and input directly into a computer, as illustrated in Figure 4.9. The success of preliminary mapping can be enhanced by letting an early contract to clear vegetation, allow safe access and to put down trial pits and trenches on the instruction of the mapping geologist (Figure 4.10).

Soils and rock can be examined, described and characterised in natural exposures and in trial pits and trenches, and full descriptions should be provided, as discussed later. Samples can be cut by hand for transfer to the laboratory, with relatively slight disturbance (Figures 4.11 and 4.12).

Access can be facilitated by using hydraulic platforms or by temporary scaffolding (Figure 4.13). Trial pits and trenches should not be entered unless properly supported, and care must be taken in examining any steep exposure; as a general rule, for safety reasons, field work should be conducted by teams of at least two people.

Figure 4.8 Spring line revealed following heavy rain at base of Carboniferous Limestone, north of Kilnsey Crag, West Yorkshire, UK.

Figure 4.9 Hand-held computer with ortho-corrected air photographs and terrain maps, used to locate and map natural terrain landslide. GPS used to get accurate locations of identified features.

Apart from the general benefits to be gained from mapping freely available or cheaply created surface exposures to determine local geology, they are particularly important for characterising aspects of rock structure such as roughness and persistence of discontinuities, which cannot be determined in boreholes. As for all measurements, however, extrapolation should only be made with caution and with awareness that structure and rock quality may change rapidly from location to location (Piteau, 1973). Exposed soil may be desiccated and stronger than soil at depth; exposed rock will often be more weathered with closer and more persistent fractures than rock only a few metres in from the exposed surface.

Figure 4.10 Local labourers employed to dig some trial pits during preliminary field mapping. Tlemcen University and Hospital site, Algeria.

Information gained from desk study and site reconnaissance can be analysed and draped over 3D digital models using GIS, as illustrated in Figure 4.14, which greatly assists visualisation, interpretation and planning of GI, including access.

4.4.2 Describing field exposures

The task of describing a large field exposure, say in a cut slope, can be daunting, and the following procedure is recommended. The exposure (natural or man-made) should be split initially into zones, layers or units, by eye. The primary division will often be geological, i.e. rock and soil units of different age, but then differentiated by rock or soil mass quality such as degree of weathering or closeness of fracturing. Differentiation on strength can be made quickly by simple index tests such as hitting or pushing in a hammer. The split might be on structural regime, i.e. style and orientation of discontinuities. The process is

Figure 4.11 Hand trimming a sample to size in the field, for transportation to laboratory and triaxial testing.

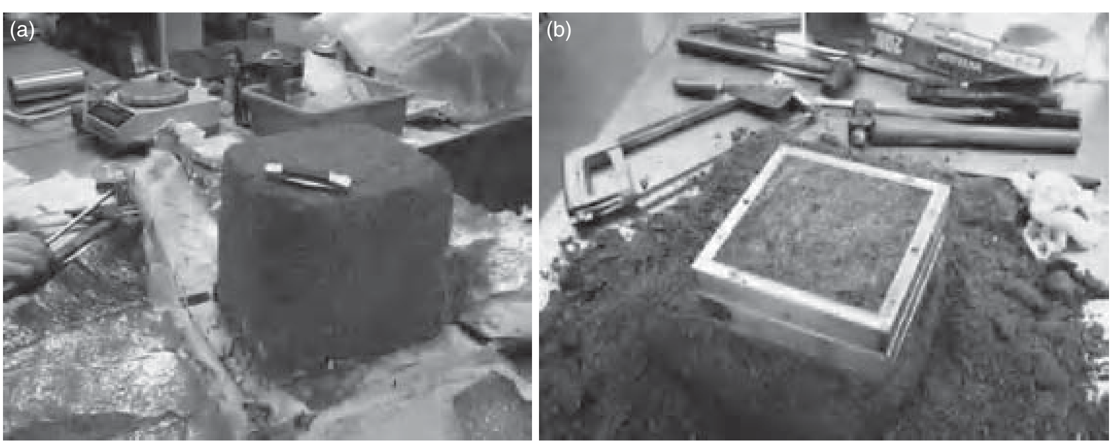


Figure 4.12 (a) Block sample cut into grade IV weathered sedimentary rock and transported to the laboratory. (b) The sample trimmed by hand to fit into a Leeds direct shear box.

Figure 4.13 Cherry picker platform used to examine recently failed rock slope to allow remedial action to be determined, Hong Kong.

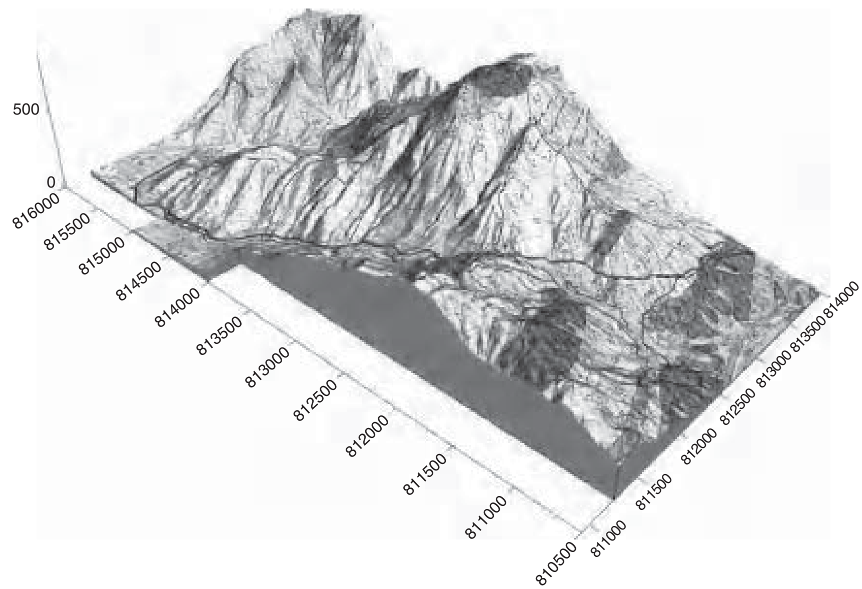


Figure 4.14 Surface geology draped onto topographic representation, for assessment of new road.

illustrated in Figure 4.15. Once the broad units or zone boundaries have been identified, then each needs to be characterised by systematic description and measurement, as shown schematically in Figure 4.16. Evidence of seepage should be noted; lush vegetation can be indicative of groundwater. The distinction between engineering geological mapping and normal geological practice is the emphasis on characterising units in terms of strength, deformability and permeability, rather than just age (Dearman & Fookes, 1974).

Some of the equipment that might be used in field characterisation of exposures includes safety harness, tape measures, hammer, knife, hand penetrometer, Schmidt hammers (type N and L), compass/clinometer

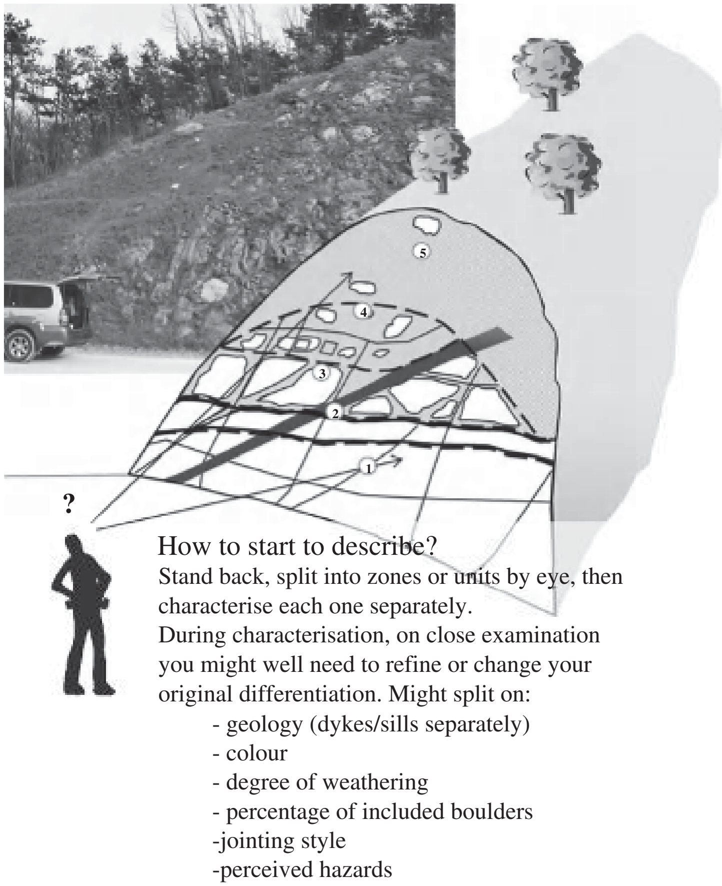


Figure 4.15 Approach to characterise rock mass. First stage is to split into units by eye. Units/zones will be used in later analysis and design.

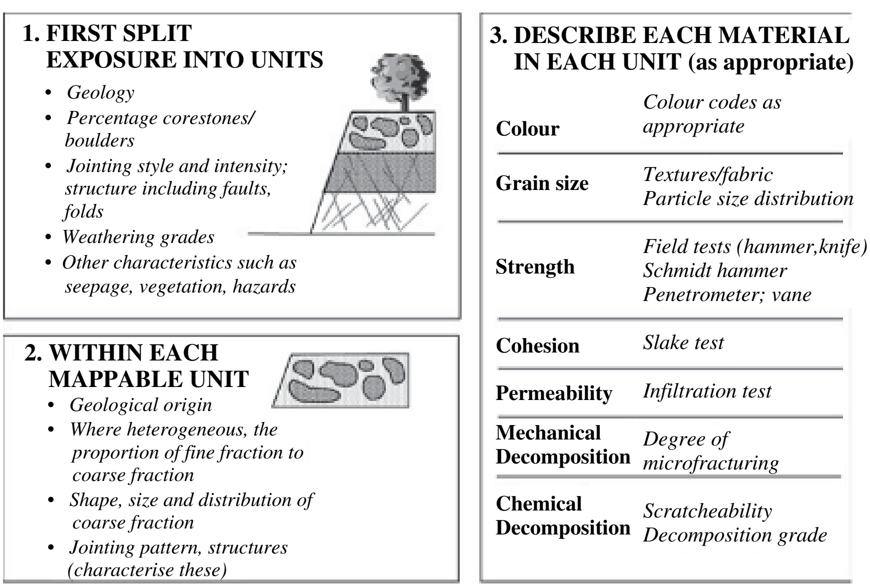


Figure 4.16 Once the broad units/ layers have been identified, each needs to be characterised.

and hand lens. Water and a container are useful for conducting index tests such as slake tests and for making estimates of soil plasticity and grading. Where appropriate, strength can be measured using such tools as a hand vane, and point load testing, which can be carried out on irregular lumps of rock. Whatever measurements are taken at exposures, the end user needs to be aware that it may be inappropriate to extrapolate properties because of the effects of drying out or softening from seepage and possibly the effects of weathering.

Guidance on geological mapping and description is given in a five-volume, well-illustrated handbook series by the Geological Society of London, which deals with Basic Mapping, the Field Description of Igneous, Sedimentary and Metamorphic Rocks (referenced in Chapter 3) and Mapping of Geological Structure, each with more than 100 pages (www.geolsoc.org.uk). Much of the detail that could be recorded by a geologist, however, might prove irrelevant to an engineering project, but what is or is not important might not be immediately obvious. It is worth bearing in mind the observation of Burland (2007):

> 'It is vital to understand the geological processes and man-made activities that formed the ground profile; i.e. its genesis. I am convinced that nine times out of ten, the major design decisions can be made on the basis of a good ground profile. Similarly, nine failures out of ten result from a lack of knowledge about the ground profile.'

Despite this observation, current standards codes and textbooks dealing with ground investigation tend to take a very simplified, prescriptive, formulaic approach in their recommendations for the description of geological materials and structure. The reason dates back to the 1960s when Deere (1968) noted that:

> 'Workers in rock mechanics have often found such a classification system [geological] to be inadequate or at least disappointing, in that rocks of the same lithology may exhibit an extremely large range in mechanical properties. The suggestion has even been made that such geologic names be abandoned and that a new classification system be adopted in which only mechanical properties are used.'

Deere went on to introduce classifications based on compressive strength and elastic modulus and the Rock Quality Designation (RQD), and these or similar classifications are now used almost exclusively for logging rock core, with geological detail rarely recorded.

Deere at the same time noted, however, 'the importance to consider the distribution of the different geologic elements which occur at the site'. This sentiment would have been supported by Terzhagi (1929), some of whose insightful observations on the importance of geological detail are revisited by Goodman (2002, 2003). Restricting geological description to a few coded classifications, as in industry standards, is over-simplistic but it is a fine balance between providing too much geological information and too little.

Generally, GI loggers tend to provide minimal summary descriptions, as per the examples given in BS5930 and other standards, and avoid commenting on unusual features, although it varies from company to company and, of course, the knowledge and insight of the logger. Some guidance on standard logging is given in Appendix C and examples of borehole logs are provided in Appendix D and discussed later. Fletcher (2004) provides many examples of the kind of geological information that can be obtained from logging of cores for engineering projects, most of which would be missed if following standard guidelines for engineering description and classification.

There is much to be said for the engineer informing the GI contractor of his preliminary ideas regarding the ground model, based on desk study and reconnaissance, so that the contractor knows what to look for and can update the model as information is gained.

Rock exposures are particularly important for characterising fracture networks. Orientations are usually measured using a compass clinometer, as illustrated in Figure 4.17, with different diameter plates used to help characterise the variable roughness at different scales (Fecker & Rengers, 1971). Electronic compass/clinometers are under

Figure 4.17 Joint survey underway using Clar compass clinometer attached to aluminium plates. Investigation for Glensanda Super Quarry, Scotland.

development, which will avoid the need to level the instrument, which can be difficult, especially in the underground mapping of tunnels.

Data are usually collected by systematic scan-line or window surveys but these are tedious to carry out, seem to be routine to the unknowledgeable, and therefore sometimes delegated to junior staff who may be unable or reticent to exercise independent judgement on what is or is not significant. Such surveys can give a false impression of rigorous characterisation, whilst the important element of geological interpretation, best done in the field, is lacking. Experienced engineering geologists with training in structural geology should be able to assess the rock conditions by eye, both with respect to the geological conditions and potential for instability in a slope, and therefore can carry out a subjective survey (Figure 4.18). The recommended approach for collection and interpretation of discontinuity data from rock exposures is set out in Box 4-7.

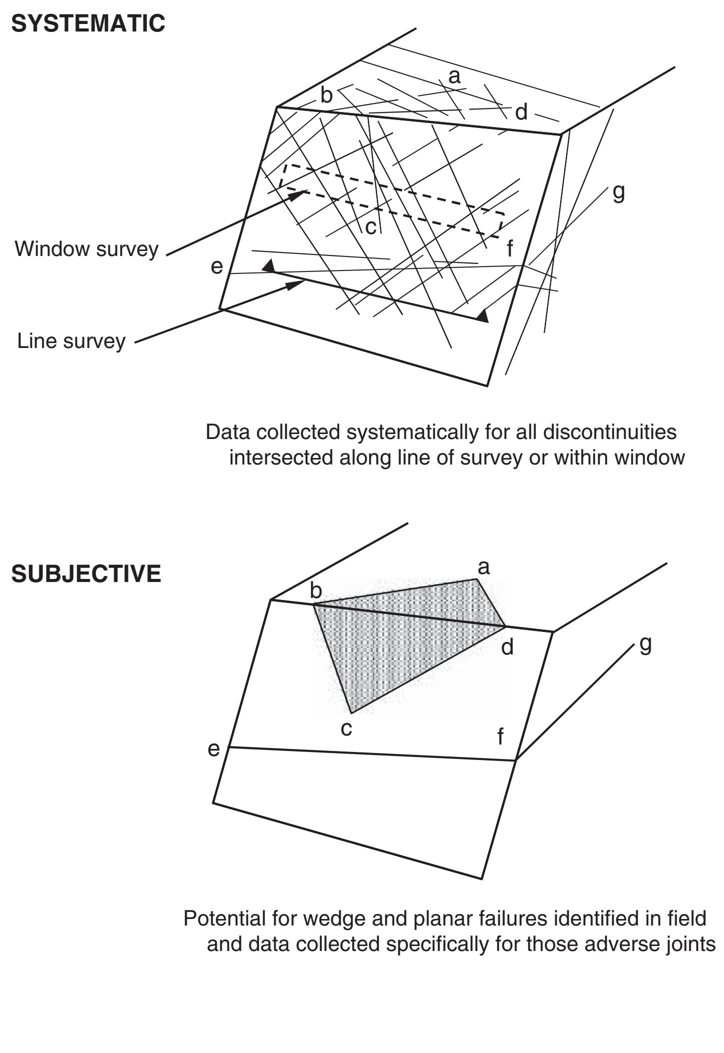


Figure 4.18 Distinction between objective surveys (line/ window) and subjective surveys (from Hencher, 1987).

Box 4-7 Collection of discontinuity data in exposures (modified from Hencher & Knipe, 2007)

1. First take an overview of the exposure. Examine it from different directions.
2. Develop a preliminary geological model and split it into structural and weathering zones units. Sketch the model.
3. Broadly identify those joint sets that are present, where they occur, how they relate to geological variation and what their main characteristics are, including spacing, openness as mechanical fractures (or otherwise), roughness, infill and cross cutting or terminations in intact rock or against other discontinuities. Surface roughness characteristics such as hackle marks should be noted as these are indicative of origin and help differentiate between sets.
4. Measure sufficient data to characterise each set geologically and geotechnically. Record locations on plans and on photographs. This might be done using line and window surveys but quite often these are time consuming and not very productive. It is generally best to decide what to measure and then measure it, rather than hope that the answer will be revealed from a statistical sample.
5. Plot data and look at geometrical relationships. Consider how the various sets relate to one another and to geological history as evidenced from faults, folds and intrusions (Chapter 3).
6. Search for missing sets that might have been expected given the geological setting.
7. Analyse and reassess whether additional data are required to characterise those joints that are most significant to the engineering problem.

Where the data collection point is distant from the project location, consider whether the collected data might be unrepresentative.

Remote measurement of fracture networks is becoming more reliable using photogrammetry (Haneberg, 2008) or ground-based radar (Figure 4.19) and research is progressing into the automatic interpretation of laser-scanned data into rock sets (orientation and spacing) (Slob, 2010). Currently, this approach, however, lacks any link to an interpretation of origin of the discontinuities and their geological inter-relationships (Chapter 3), which would make it much more valuable. In the author's opinion, probably the best use for laser scanning at the moment is as an aid to the field team, in particular for measuring data in areas of an exposure with difficult access, but they cannot replace mapping and characterisation by experienced persons at the current stage of development.

Rock joint data are generally represented on stereographic projections, as illustrated in Figure 4.20. The technique allows sophisticated analysis of geological discontinuity data (Phillips, 1973), but its most common use in engineering geology is for determining the potential for specific rock discontinuities to cause a failure in a cut slope or in an underground opening (Hoek & Bray, 1974 and Chapter 6). Plotting of data, statistical grouping and comparison to slope geometry is now easily done using software such as Dips (Rocscience), but care should be taken in interpretation and especially against masking important but relatively rare data (Hencher, 1985). Bridges (1990) demonstrates the importance of differentiating sets on the basis of geological characteristics rather than just geometry.

Figure 4.19 Ground-based radar being used to generate a digital image of cut slopes near Seoul, Korea. Point clouds can be used to measure discontinuity geometry remotely.

4.5 Geophysics

Geophysical techniques are used to identify the disposition of soil and rock units, based on differences in physical properties such as strength, density, deformability, electrical resistance and magnetism. They can sometimes be used successfully to identify cavities such as mine workings or solution hollows and for identifying saturated ground. Geophysics really comes into its own for offshore investigations where drilling is very expensive. Geophysics can provide considerable information on geological structure and rock and soil mass quality, which is relevant to engineering design, although such techniques are rarely used by themselves but as part of a wider investigation involving boreholes. Many engineering geologists and geotechnical engineers have both good and bad experience of engineering geophysics. Darracott & McCann (1986) argue that poor results can often be attributed to poor planning and the use of an inappropriate technique for the geological situation. More specifically, key constraints are:

- penetration achievable
- resolution
- signal-to-noise ratio, and
- lack of contrast in physical properties.

When geophysics works well, the results can be extremely useful and the method cost-effective. The main options and constraints are set out in BS 5930: 1999 and Clayton (1995).

4.5.1 Seismic methods

Seismic refraction techniques, using an energy source ranging from a sledgehammer to explosives, can be useful on land and in shallow water for finding depth to bedrock, for example, to identify buried channels that could otherwise only be proved by numerous boreholes or probes. Large areas can be investigated quite cheaply and quickly. The method works best where there is a strong contrast in seismic

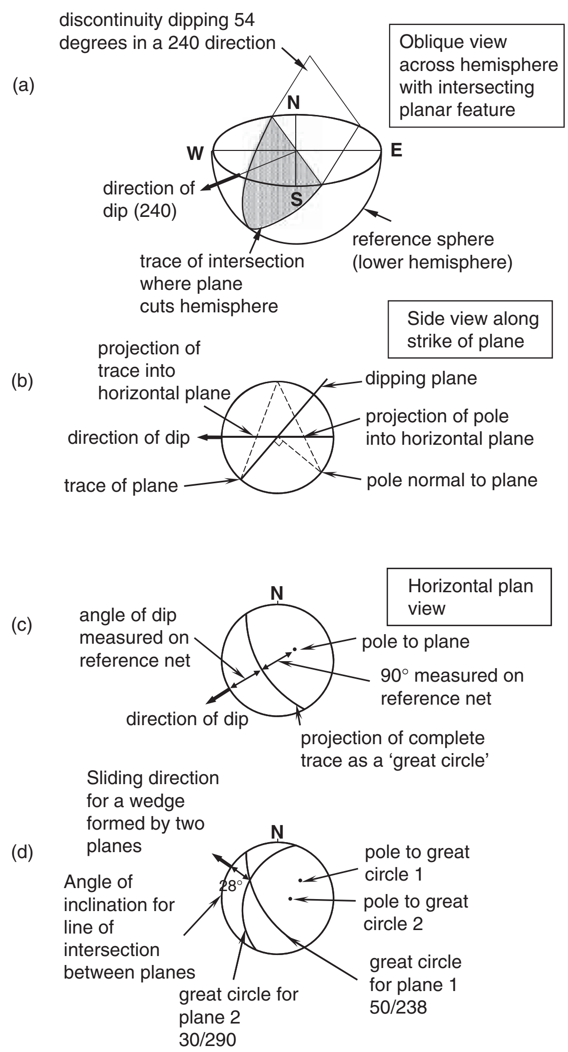


Figure 4.20 Representing discontinuity data as great circles or as poles (after Hencher, 1987).

velocity between the overlying and underlying strata and some knowledge of the geological profile, preferably from boreholes. Otherwise, results will be ambiguous. Where weak (low velocity) strata underlie stronger materials, these may not be identified by seismic survey. Wave velocity (compressive and shear) can be interpreted directly in terms of rock mass quality, deformation modulus and ease of excavation, as reviewed comprehensively by Simons *et al.* (2001). Seismic reflection is a key technique in offshore investigations.

4.5.2 *Resistivity*

Resistivity is another cheap and rapid method that can prove very effective, particularly in identifying groundwater (low resistance) and voids (high resistance). The technique has been used successfully in the investigation of landslide profiles, in particular for identifying water-bearing strata at depth. Figure 4.21 shows the results of a resistivity survey in Hong Kong to identify underground stream channels as zones of high resistance (voids), which it did extremely well (Hencher *et al.*, 2008).

4.5.3 *Other techniques*

There are a host of other techniques reported in the literature, with various success rates. Ground-based radar can be useful for finding shallow hidden pipes, etc. Other techniques such as magnetic and micro-gravity rely on particular physical properties of the rock or feature being searched for. Both have been used for locating old mine

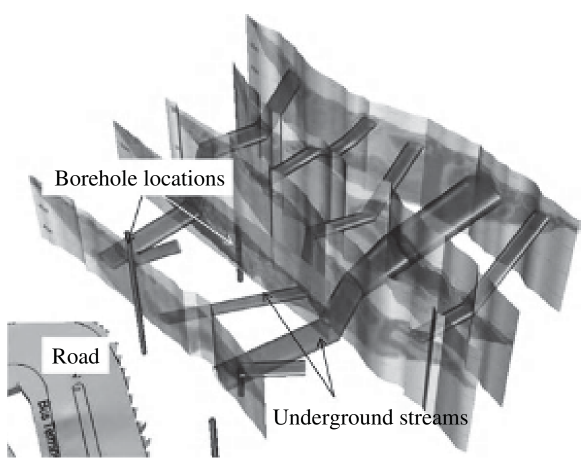


Figure 4.21 Digital image interpretation of resistivity surveys across hillside above Yee King Road, Hong Kong. Tubular features of low resistivity are interpreted as underground streams (Hencher *et al.*, 2008).

shafts – because the brick lining might have a magnetic signature and the void is low gravity. Generally, such techniques are used as a first pass across a site to identify any anomalies, which are then investigated more fully using trial pits, trenches and boreholes. For such investigations, percussive holes, as used for forming holes for quarry blasting (no coring), can be very quick and relatively cheap – the presence of voids is indicated by lack of resistance to drilling and loss of flushing medium. The voids can later be examined using TV cameras, periscopes or sonic devices to try to quantify size and shape. For many reasons, such surveys are not always successful and therefore are not to be relied upon to give a definitive answer (Clayton *et al*., 1995). Sewell *et al*. (2000) demonstrate the usefulness of marine magnetic and gravity surveys for identifying geological structures.

4.5.4 Down-hole geophysics

As with seismic reflection, down-hole geophysics is used routinely in oil and gas exploration, in mining and in sophisticated GI linked to nuclear waste disposal studies. Tools can be used to determine minor stratigraphic contrasts and rock properties. These tools are less used for engineering, with the exception of rock joint orientation (using cameras and geophysical tools) and sometimes for identifying clay-rich layers. These tools are discussed below, together with logging and description.

4.6 Sub-surface investigation

Methods and techniques for sub-surface investigation are dealt with in many publications, including BS 5930 (BSI, 1999), Clayton *et al*. (1995), GCO (1987), Hunt (2005) and Mayne *et al*. (2001).

4.6.1 Sampling strategy

There are usually four main objectives in sub-surface investigation:

1. to establish the geological profile
2. to determine engineering properties for the various units within the eventual ground model
3. to establish hydrogeological conditions, and
4. to monitor future changes in ground conditions through instrumentation.

At many sites, it is best to use preliminary boreholes in an attempt to establish the geological profile accurately. This will require sampling over the full depth and with sufficient boreholes to establish lateral and vertical variability. If recovery is low, then boreholes may need to be repeated; it is often the pieces of core that are not recovered that are the most important, because they are also the weakest. It is wise to include a clause in specifications for the GI contractor, setting out a minimum acceptable recovery, to encourage diligent work. A good driller can generally achieve good recovery in almost any ground, providing he has the right equipment and adjusts his method of working to suit the ground conditions. If he does not have suitable equipment (or flushing medium), then that might be the fault of the engineer who specified the investigation, rather than the contractor, and this may need rectification by issuing a variation order to the contract.

Once the preliminary geological model has been established adequately at a site, then additional boreholes can be put down as necessary to take samples for testing or to carry out *in situ* testing and to install instruments for monitoring changes such as response of water table to rainfall. The same approach (sample first to prove the geological model and to identify any geological hazards, followed by a second phase for testing and instrumentation) should be used for any investigation where geological features may be important. This can only be judged by a competent engineering geologist aware of both the local geological conditions and the factors that will control the success or otherwise of the particular civil engineering project.

In practice, boreholes are often put down using a strategy of intermittent sampling and *in situ* testing within a single borehole, which means that the full ground profile is not seen. This can be cost-effective for design when the site is underlain by relatively uniform deposits and where the ground profile is already well-established from previous investigations. The danger is that site-specific geological features might be missed yet prove important for the project.

4.6.2 Boreholes in soil

There are many different tools that can be used to investigate soils and many of these are described by Clayton *et al*. (1995). In the UK, the most commonly used machine for investigating soils is the shell and auger, otherwise known as the cable-percussive rig, as illustrated in Figure 4.22. Such rigs are very manoeuvrable and can be towed behind a field vehicle or winched to the point where the hole is to be put down. They can cope with a wide range of soils, which makes for their popularity in the UK, where mixed glacial soils are common. The hole is advanced by dropping a heavy shell (Figure 4.23). Material between sampling points is usually discarded, although it should be examined and recorded by the drilling contractor and disturbed bulk samples are taken in bags, if specified for the contract. All samples, of course, should be sealed and labelled. If boulders are encountered in the soil profile, these are broken up with a heavy chisel dropped down the hole. Engineers usually specify alternate undisturbed samples for

Figure 4.22 Shell and auger rig in action, Leicester, UK. Casing, used to support the hole, is standing out of ground and a shell is being dropped down hole to excavate further. In the foreground is a U100 sampling tube attached to a down-hole hammer, ready for placing down hole and taking a sample once the hole has been advanced to the required depth. Leaning against the wheel is one of the drillers and also a trip hammer for SPT testing – also awaiting use at appropriate depths and changes in strata.

laboratory testing and *in situ* strength tests at perhaps 1.5 m intervals or changes in strata. The standard penetration test (SPT) is commonly used to measure strength, as discussed below under *in situ* testing. Vane tests might be carried out rather than SPTs, especially in clay soils. USA practice for investigating and sampling soils is described by Hunt (2005). One cheap and quick way of sampling/testing is to use wash boring, whereby the hole is advanced by water jetting as rods are rotated. SPT tests, and possibly other samples, are taken at intervals. None of these methods gives continuous sampling, so geological detail may be missed.

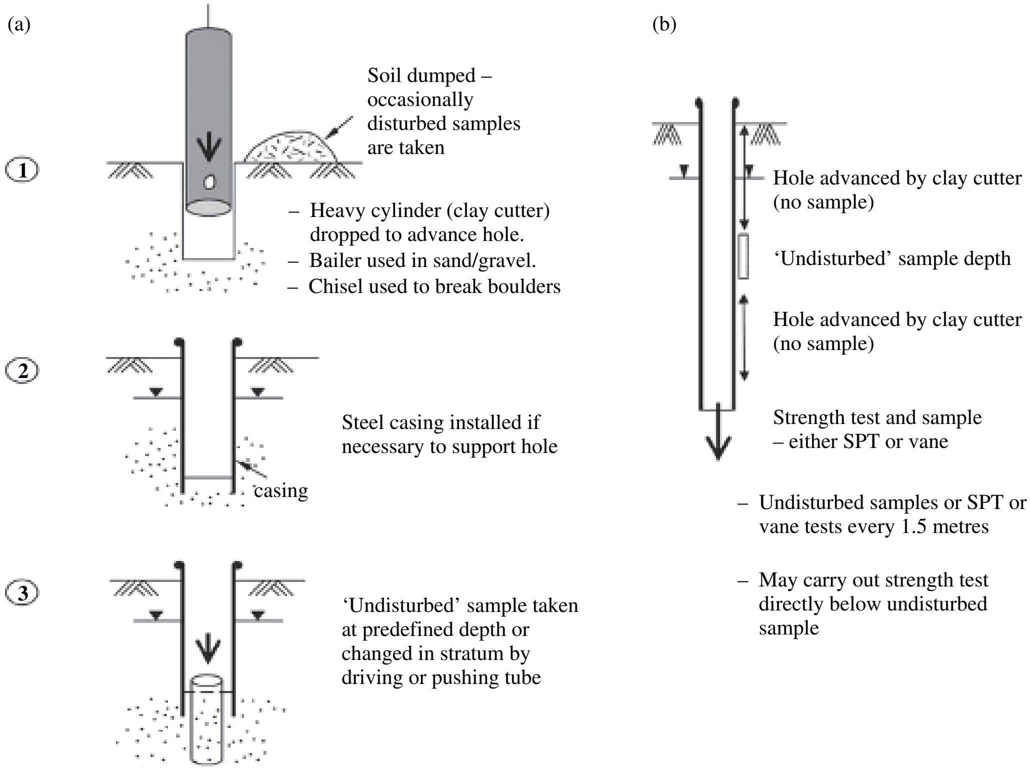


Figure 4.23 (a) Methodology for shell and auger advancement of boreholes. (b) Sampling strategy.

Undisturbed samples are usually taken using a relatively thin walled sampler of diameter 100 mm (U100), and much of the published empirical relationships that are relied upon by designers are based on tests on samples achieved in this way. This sampling method does not, however, meet the more stringent requirements of Eurocode 7 for class 1 sampling and testing, because of fears over disturbance. This is rather naïve in that it implies that thinner sampling tubes can take an undisturbed sample, which is not the case. Any sample taken from depth, squeezed into a tube and then extruded at the laboratory, will inevitably be disturbed to some degree. Further disturbance occurs during preparation of samples for laboratory testing and initial loading and saturation, as expressed schematically in Figure 4.24 and investigated by Davis & Poulos (1967). The engineering geologist and geotechnical engineer need to be aware of the likely disturbance to any tested samples and take due care in interpretation. Furthermore, the scaling up of results from laboratory to project scale requires careful consideration because it must include the effect of mass fabric and structure, including fractures and discontinuities. This is discussed further in Chapter 5.

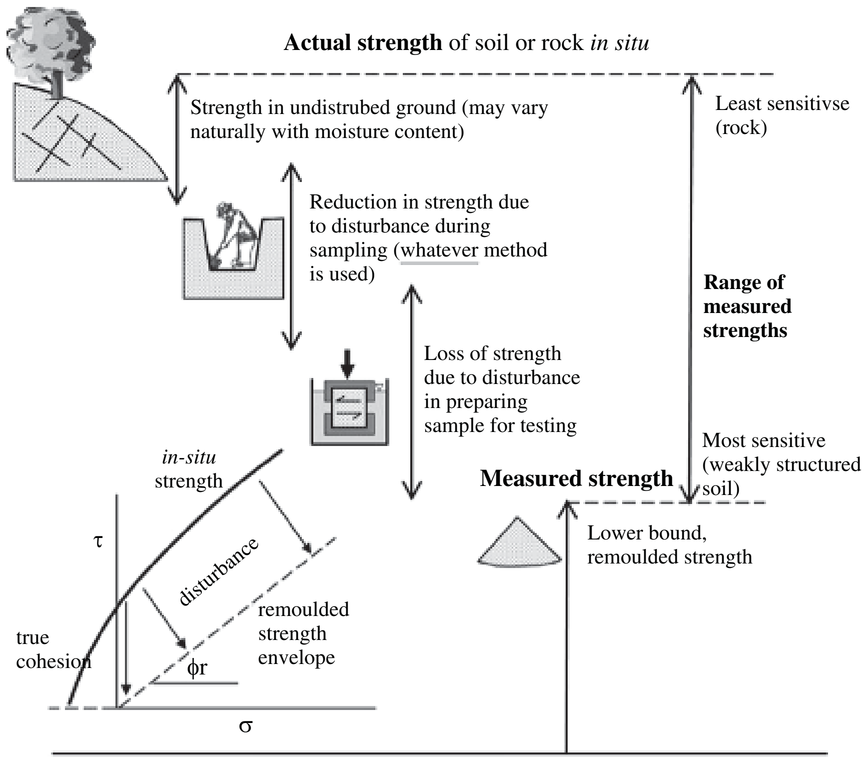


Figure 4.24
Potential sources of sampling disturbance leading to much lower strengths being measured in the laboratory compared to those *in situ*.

4.6.3 Rotary drilling

Rotary drilling is used in all rocks but can also be used to obtain good samples in weaker materials, including colluvium (mixed rock and soil), weathered rock and soil. In weaker ground, a similar investigation strategy is often adopted as for soils, whereby sections are cored followed by SPT tests, although as for soils there is the risk that important geological features may be missed.

A drilling rig rotates a string of drilling rods whilst hydraulic cylinders apply a downward force. At the lower end of the drilling string there is a hollow annulus bit, usually coated with diamonds or tungsten carbide. As the bit is rotated, a stick of core enters into a core barrel at the bottom of the drilling string. The retained core is prevented from falling out as the barrel is brought back to the surface, by some form of core-catching device. Air, water, mud or foam is used to cool the bit and carry rock cuttings back to the surface (Figure 4.25). Where cored samples are not required over a particular length of hole, it can be advanced more quickly using rock roller bits, down-the-hole hammers and water jets, as used in much oil and gas drilling.

At the most basic level, a single-barrel well-boring rig can be used to take core samples but these are often highly disturbed (Figure 4.26). Most drilling is carried out using double-barrel systems in which the outer barrel rotates around an inner barrel that takes in the core. A

Figure 4.25 Rotary drilling above fatal landslide at Fei Shui Road, Hong Kong. Polymer foam (white) is being used as the drilling flush to try to improve recovery.

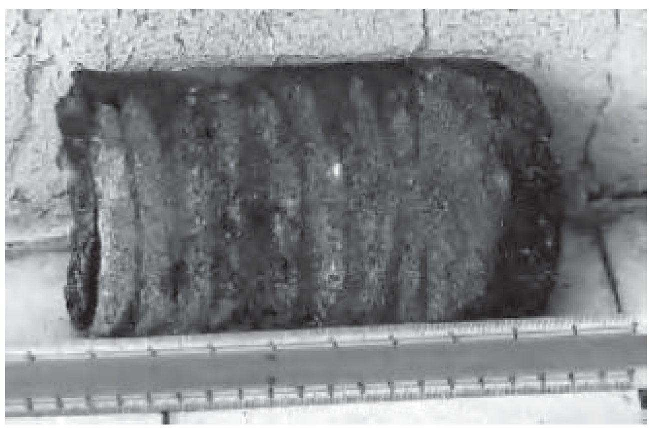


Figure 4.26 Sample obtained from single-barrel Russian well drilling rig, El Hadjar steelworks, Annaba, Algeria (see preface). Previous logging of similar samples had interpreted the layering as some kind of varved sequence of silt and sand. Actually, the horizon *in situ* is fairly uniform weathered (grade IV) gneiss (the pale material). The dark-brown silt horizons represent occasions when the Algerian driller, bored with the slow drilling progress from his worn-out bits, raised the drilling string and then dropped it again with some force down the hole, letting in a layer of the silty drilling mud, which then became baked by the heat from the drilling process … The thickness of the pale layers are an indication of the driller's boredom threshold – generally pretty consistent.

problem with the double-tube system is that the flushing medium flows between the core and the core barrel and can wash away some of the cored material, but it is still used internationally because it is relatively inexpensive and can be mass produced. The problems can be reduced by using a triple-tube system. In this system, the core enters a split inner tube, which does not rotate; the flushing medium flows between the inner tube and an outer tube without touching the core. Such equipment has low manufacturing tolerances so must be bought off the shelf, and the bits are very expensive and only last perhaps 8 to 12 m of coring before they need to be replaced, which precludes its use on many projects.

Usually, the larger the diameter of the core barrel, the better the recovery and quality of sample, and it is prudent to start using a large diameter and reduce diameter as necessary with depth. The wide range of casing, core barrel and drill rod sizes are listed in ASTM (1999), which also discusses good practice. When there is good-quality rock overlying soil material, retrieving the softer material can be a problem. As for soil boring, the hole may need to be cased temporarily during drilling to prevent it collapsing. Drillers generally try to recover about 1.5 m of core per run before pulling all the drill string back to the ground surface and dismantling it all. If recovery is low, then the driller might try to reduce the core run to 1 m or even less, but this does not always produce better results. Other parameters such as thrust, torque and flushing medium may have more influence on recovery, and much depends on the experience, knowledge and attitude of the drilling crew.

Wire line drilling employs large-diameter rods, which effectively support the hole as it advances. After each core run, the core barrel is pulled up the centre of the drill rods, the core extracted, then dropped back down the hole to lock into the bottom of the hole, ready to start drilling again. The cutting bit stays at the bottom of the drill rods and is not extracted with the core barrel. To change the cutting bit, however, the whole drill string has to be removed.

A system that is very commonly used in Hong Kong and elsewhere for sampling weathered rock and mixed rock and soil is a Mazier core barrel. This has a soil cutting shoe which is spring loaded and extrudes in advance of an outer rock cutting bit when cutting through relatively weak soil-like material (Figures 4.27 and 4.28). As conditions get harder, the soil cutter is pushed back and the outer coring bit takes over. This system, especially where combined with polymer foam flush, has been shown to produce good recovery of material in weathered and mixed materials (Phillipson & Chipp, 1982). The sample is taken in a plastic tube, which is later cut open so that the sample can be examined, described and tested (Figure 4.29). Drilling contractors will not open tubed samples without instruction to do so, and, in practice, geotechnical engineers sometimes order Mazier samples (from the office) but then never get round to opening and examining the samples, which is poor practice. The author was recently involved in an arbitration where 20 boreholes had been put down with alternate Mazier sampling in soft clays and then SPTs. The project was then designed on the basis of the SPT data alone and went badly wrong, ending in arbitration. The samples had not been opened up for examination or testing. A similar system to the Mazier, used in the USA, is the Dennison sampler (Hunt, 2005).

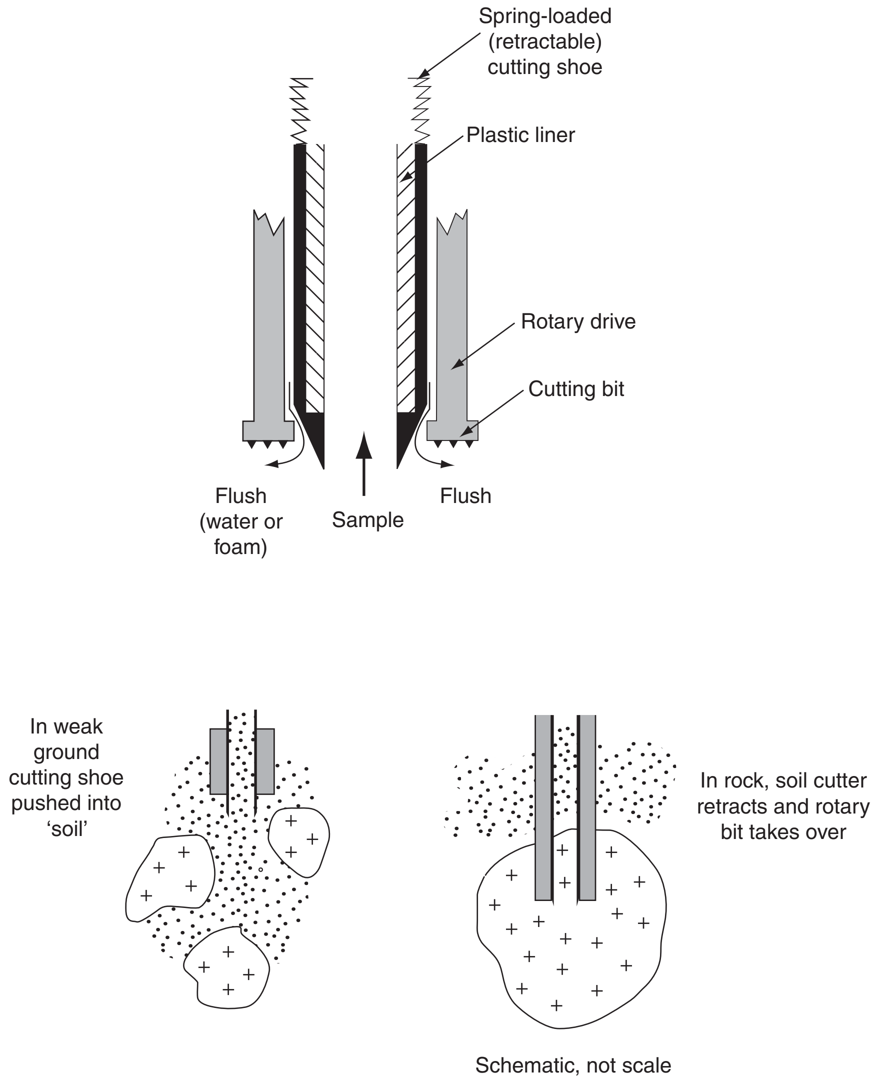


Figure 4.27
Principles of Mazier sampling.

4.7 *In situ* testing

Many parameters are obtained for design by laboratory testing, as discussed in Chapter 5, but the potential for disturbance is obvious, as discussed earlier, especially for granular soil that disaggregates when not confined. There are therefore many reasons for attempting to test soil and to a lesser extent rock *in situ*. Most tests are conducted in boreholes, but some are conducted by pushing the tools from the ground surface or from the base of a borehole to zones where the soil is relatively undisturbed. A self-boring pressuremeter, suitable for clay and sand, drills itself into the ground with minimal disturbance before carrying out a compression test at the required level.

The SPT is probably the most commonly used *in situ* test, whereby the number of blows to hammer a sample tube into the ground is

Figure 4.28 Mazier sampler with nicely recovered weathered granite – the right side third is stained with iron oxides. Spring-loaded cutting shoe is seen extending from the rock cutting bit outside. When the material strength becomes too high for the cutting shoe (exceeds spring stiffness), the outer bit takes over the cutting.

Figure 4.29 Mazier sample plastic tube being cut for examination.

recorded. Soft soils are penetrated easily, hard soils and weak rocks with more difficulty. The SPT data can be interpreted in terms of shear strength and deformability (Chapter 5) and for making predictions of settlement directly (Chapter 6). The split spoon sampler used for the SPT is a steel tube with a tapered cutting shoe. It is lowered down the borehole, attached to connecting rods, and then driven into the ground by a standard weight, which drops a standard height, as illustrated in Figure 4.30 and shown in action from a rotary drilling rig in Figure 4.31. The number of blows for each penetration of 75 mm is recorded; blows for the first 150 mm are recorded but essentially ignored (considered disturbed); the blows for the final 300 mm are

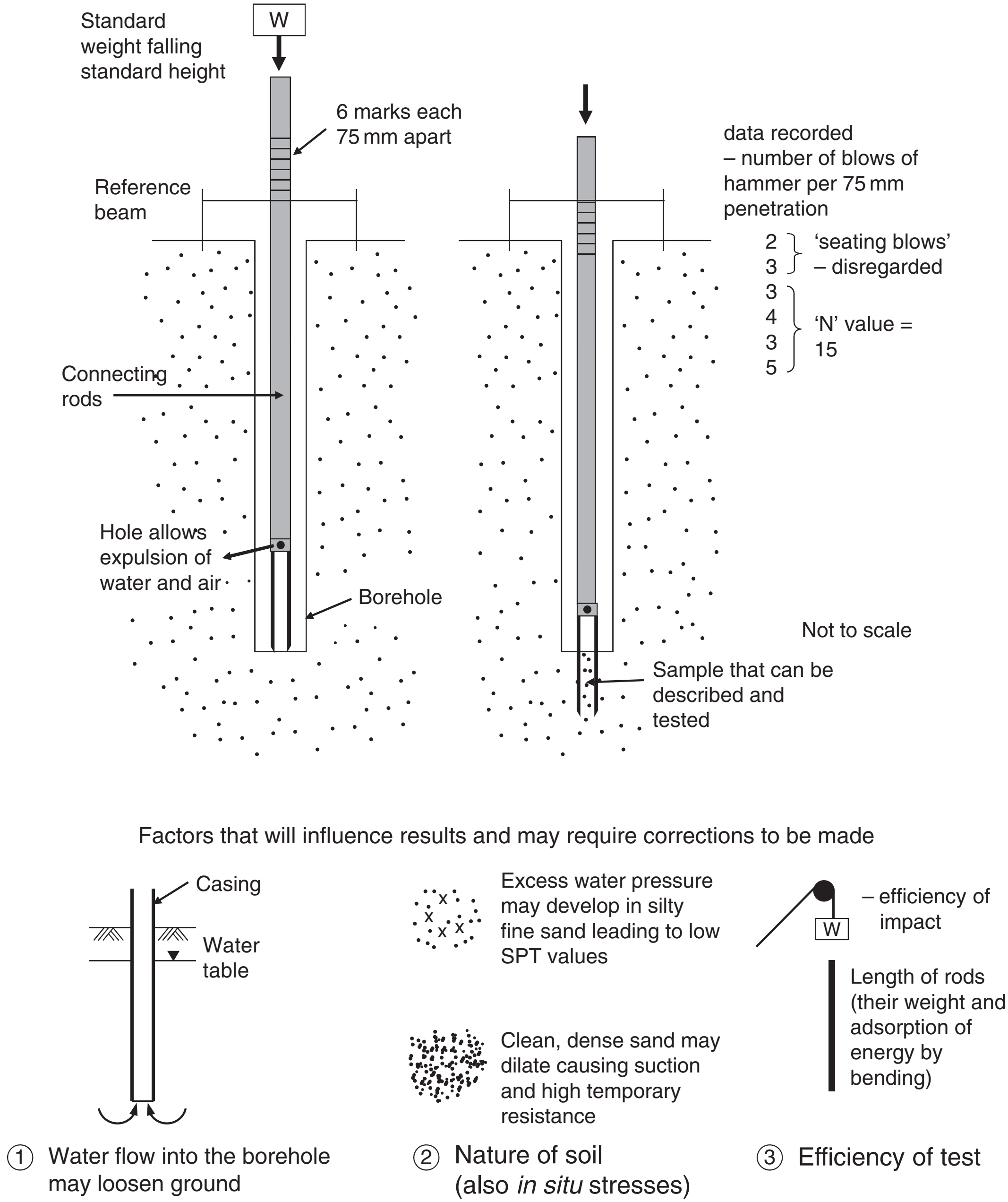


Figure 4.30 Principles and details of the SPT test.

added together as the N-value. Care must be taken in soil that the external water table is balanced, otherwise water may flow in from the bottom of the hole, causing softening and too low an N value. There are various corrections suggested for tests conducted in silty sand and for depth of overburden. Details are given in Clayton (1995).

Figure 4.31 SPT test underway in Hong Kong (1980s). Nowadays, a helmet would be worn.

The SPT test is much maligned for associated errors but nevertheless is still the most common basis for design in many foundation projects, mainly because no-one has come up with anything better. It is also actually quite a useful sampling tool, as illustrated in Figure 4.32. In

Figure 4.32 Split spoon sample of completely weathered granite. Note presence of relict joints and lack of visible disturbance.

the UK, it is normal to stop a test when 50 blows fail to advance the split spoon the full 300mm and instead to record the penetration achieved for the 50 blows. Depending on the ground conditions and sample retrieved, it might be valid to extrapolate the blow count to an equivalent N-value *pro rata*. Overseas, it is common practice to continue the test for 200 blows or more in weathered rock, and designs are often based almost solely on such data, which is rather questionable practice in a profile that might comprise a heterogeneous mix of harder and softer materials. Tests carried out in this way may damage equipment and are tedious for the drilling contractor, who might well be tempted to cut corners if no-one is supervising. The interpretation of SPT testing in weak and weathered rock is discussed in more detail in Chapter 5.

The vane test involves rotation of a cruciform steel tool at a slow rate within the soil (Figure 4.33). The test is especially suitable for soft clay where SPTs are inappropriate because of the indeterminate nature of pore pressure changes brought about by rapid loading. The vane test is assumed to give a direct measure of undrained shear strength for the shape sheared by the rotating tool but interpretation can be difficult, especially in bedded soils.

The static cone penetrometer is a conical tool (like an SPT) that is pushed rather than driven into the ground, usually from a heavy lorry (Figures 4.34 and 4.35). The end force on the cone tip, and drag on the sides of the tool, are measured independently and can be interpreted in terms of strength and deformability. Clay, being cohesive, grips the side proportionally more than sand or gravel, so the ratio between end resistance and the side friction can be used to interpret the type of soil

Figure 4.33 Field vane used for measuring strength of clay down borehole or sometimes pushed from the ground surface. Once at test location, the vane is rotated to measure shear strength of the cylinder of soil defined by the vane geometry. To the left is a sleeve used to protect the vane during installation.

Figure 4.34 Electric static cone penetrometer with piezometric ring. Forces on the cone tip are measured independently from the force on the shaft section above. A combination of all three measurements (including water pressure) gives a good indication of soil type as well as strength characteristics.

Figure 4.35 Heavy lorry being used to conduct static cone penetrometer tests.

as well as strength. A further refinement (piezocone) allows water pressures to be monitored as the cone is pushed in, which again can help in interpreting the soil profile.

Large-scale direct shear tests are sometimes carried out in the field (Figure 4.36), in the hope that scale and disturbance effects might be reduced. In reality, lack of control in the testing process, as well as questions over representation of samples, however large, often outweighs any advantages. The derived data are generally less reliable than those from a series of laboratory tests, which themselves would need very careful interpretation before use at the mass scale, as discussed in Chapter 5.

Figure 4.36 In situ direct shear test in trial trench, Hong Kong.

Small-scale deformability tests down boreholes include the use of inflated rubber packers in soil (pressuremeter) or the Goodman jack in rock where two sides of the borehole are jacked apart. All such tests are very small relative to the mass under consideration and need to be interpreted with due care as to their representativeness. Deformation at project scale is better predicted from loading tests involving large volumes. The inclusion of very high capacity Osterberg jacking cells set within large diameter, well-instrumented bored piles, as discussed in Chapter 6, gives the prospect of deriving much more representative parameters (e.g. Seol & Jeong, 2009). In practice, most rock mass parameters tend to be estimated from empirical relationships derived from years of project experience together with numerical modelling, rather than small-scale tests, as discussed in Chapter 5.

Field tests are really the only option for measuring hydraulic conductivity (also for oil and gas). Simple tests include falling or rising head tests in individual boreholes, whereby water is either added to or pumped out of a hole and then the time taken for water to come back to equilibrium measured. For realistic indications of behaviour at field scale, however, larger-scale pumping tests are required. Even then, water flow is often localised and channelled so tests may not always be readily interpreted.

4.8 Logging borehole samples

Data from ground investigations are generally presented in a report comprising factual data as well as an interpretation of conditions (if the GI contractor is requested to do so). One of the important jobs for an engineering geologist is to examine and record the nature of samples retrieved from boreholes. The data from individual boreholes is usually presented in a borehole log, which provides a record not only of the ground profile but many details of how the borehole was carried out. In the oil industry, where the hole is advanced by a rock-roller bit or similar destructive method, logging is done by examining small chips of rock carried in the flushing mud (well logging); in civil engineering, we generally have rather better samples to examine.

Logging is generally conducted using a checklist approach and employing standard terminology to allow good communication, for example, on the apparent strength of a sample. Such standardisation can, however, result in over-simplification and lack of attention to geological detail. The task might be delegated to junior staff who might not have the experience and training to fully understand what they are examining. In addition, GI contractors will not routinely describe all features of samples recovered, partly because they want to avoid disturbing the samples before the client/design engineer has made a decision on which samples he wishes to select for laboratory testing. Several examples of borehole and trial pit logs are provided in Appendix D. The examples prepared by GI contractors in the UK and Hong Kong demonstrate good practice, whereby the whole process of drilling a hole, testing down the hole and sampling are recorded. The materials encountered are described following standard codes and normal practice. Given the limitations discussed above, designers and investigators may need to examine samples and core boxes themselves and not rely on those produced by the contractor. In Appendix D, examples are given of logs prepared by engineering geologists who have the responsibility for the overall site investigation. These are supplementary to the logs produced by the GI contractors. The Australian example is from an intensive investigation of a failing slope that was threatening a road. There is considerable attention to detail, especially regarding the nature of discontinuities and far more so than in the contractor's logs. In practice, even this level of logging may be inadequate to interpret the correct ground model, and selected samples and sections of core will need to be described in even more detail by specialists, perhaps employing techniques such as thin-section microscopy, radiometric dating and chemical analysis. In all cases and at all levels, logs should be accompanied by high-quality photographs with scales included.

As discussed in Appendix C, guidance on standardised terminology is given in BS5930: 1999, in the GEO guide on rock and soil description (GCO, 1988) and the ISRM guidance on rock mass description (ISRM, 1981). There are many different standards and codes of practice in use worldwide – USA practice is far removed from that in the UK, as is that for Australia, China, New Zealand, Japan and Korea, which leads to confusion, particularly as similar terminology is often used to mean different things. A consequence of this fuzzy standardisation is that when projects go wrong geotechnically, as they sometimes do, then legal arguments often hinge on incorrect or misinterpretation of terminology. The engineering geologist needs to do his homework before practising in any region.

Another criticism made earlier regarding field mapping, but equally applicable to logging, is that standard guides and codes to rock and soil description tend to comprise a series of limited classifications that one has sometimes to force on an unwilling rock mass. For example, rock masses, as exposed in quarries, can seldom be simply described as widely or closely jointed, but loggers are required to apply such classifications to core samples. In the author's opinion, it is far better to concentrate on recording factual data, which can then be interpreted as the overall ground model becomes clearer. An example of over-simplified rock classification terminology is given in Box 4-8 with reference to the term aperture. The problem is that by using such terms it is implied that the feature has properly been characterised, which is not the case. De Freitas (2009) discusses the same point and also notes that many terms and indeed measured values such as porosity are lumped parameters and therefore rather insensitive and uninformative.

Box 4-8 Defining aperture: an example of poor practice by geotechnical coding committees

This example is used to illustrate the inadequacy of current geotechnical standards for soil and rock description to convey an accurate or realistic representation of the true nature of the geological situation.

Mechanical aperture is the gap between two rock discontinuity walls (three-dimensional) and a very important characteristic with respect to fluid flow and grouting. It is expressed in most codes and standards as a one-dimensional scale of measurement, in the same way as joint spacing. The various attempts at revising description of aperture over 25 years (leading to the current BS/Eurocode 7 requirements discussed later) have simply reinvented the measurement scales and terminology but have failed to address or inform users about the fundamental difficulties in measuring and characterising this property.

What is aperture?

It is the mechanical gap between two walls of a rock discontinuity such as a joint or a fault. An example of a small section of joint with a gaping aperture (because the block has moved down slope and dilated over roughness features) is shown in Figure B4-8.1, which is a photograph of a section of sheeting joint in granite from Hong Kong.

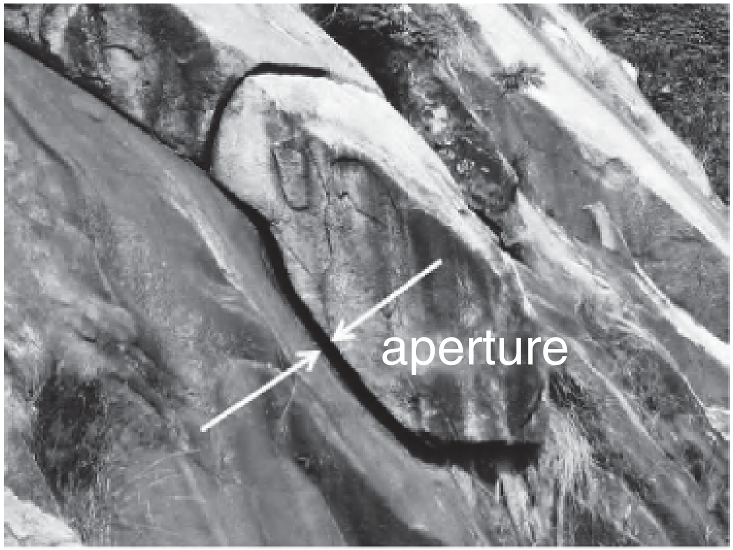


Figure B4-8.1 Part of sheeting joint with gaping aperture where seen. Evidently, away from the exposure the aperture is tight and the rock walls are in contact. Example is near Sau Mau Ping, Hong Kong.

In the second example, of a fault exposed at a beach, also in Hong Kong (Figure B4-8.2), it is not quite so easy; there is a groove along the feature but the astute geologist might interpret this as preferential erosion. Some authors advise measuring aperture using feeler gauges. Others have attempted to characterise aperture volumetrically by injecting resin or liquid metals.

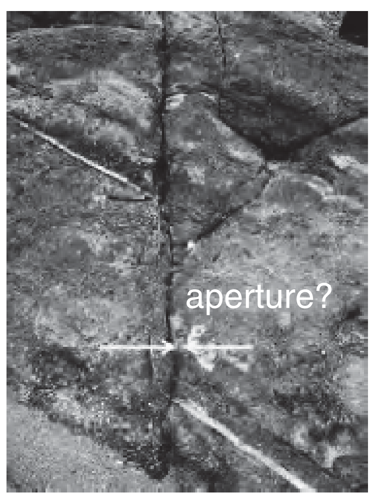


Figure B4-8.2 Minor fault exposed on beach, Peng Chau Island, Hong Kong.

Does it matter?

It is an extremely important property of the rock mass, controlling fluid flow and also related to shear strength. The problem is it is a very complex and unpredictable characteristic, as is the associated fluid flow. A single joint can be locally tight and impermeable, whilst elsewhere can be open allowing huge volumes of water to flow, as discussed by Kikuchi & Mito (1993). Investigation and characterisation can be a nightmare – if a borehole hits a conductive section, then high permeabilities will be measured and an installed instrument will be responsive to changes in water pressure, but this is literally a hit or miss business, as evidenced by many examples in investigations associated with nuclear waste (e.g. Thomas & La Pointe, 1995). The author has the experience of working in a deep tunnel 150m below the sea, where over one section, the rock was highly jointed but dry, but elsewhere, at the same level, there was a steady inflow through what was apparently intact rock. Clearly, it is not just local aperture that matters, but the characteristics of the full fracture network and its connectivity leading to the point of observation. It is an important area for research and for observation linked to geochemical and structural studies together with an appreciation of coupled mechanisms (e.g. Olsson & Barton, 2001; Sausse & Genter, 2005). Without getting to grips with the concept of channelised flow on rock joints and through joint networks, it may be impossible to ever make a safety case for nuclear waste disposal, with all the corollaries, i.e. no nuclear power, global warming and the end of civilisation. Well, perhaps slightly overstated, but not that much.

Apart from the natural variability of fracture networks, are there any other considerations?

Yes. Most rock joints are sampled in boreholes where aperture simply cannot be measured. Furthermore, it is very unlikely that any borehole sample would be representative of the discontinuity at any great distance. Down-hole examination with cameras and periscopes can be used to examine borehole walls, but again there is a problem with sampling and representativeness. In exposures such as quarries or tunnels, exposure is better but there is a question of disturbance – blasting, stress relief and block movement and whether observations at one location are relevant to the rock mass as a whole.

So what advice is given in recommended methods and standards?

1978 ISRM. The discussion on aperture is very useful. Its importance is recognised and many of the difficulties in measurement and interpretation are highlighted.

For description purpose and where appropriate, apertures are split into closed, gapped and open features, each subdivided into three. It is advised that:

a. modal (most common) apertures should be recorded for each discontinuity set
b. individual discontinuities having apertures noticeably wider or larger than the modal value should be carefully described, together with location and orientation data, and
c. photographs of extremely wide (10–100cm) or cavernous (>1m) apertures should be appended.

1999 UK BS5930 (BSI, 1999). Says little about aperture other than noting that it cannot be described in core. Five classes are introduced, which use some of the same terms as ISRM but with different definitions.

2003 INTERNATIONAL STANDARD ISO 14689–1 (BSI, 2003) (for Eurocode 7 users). Provides a new mandatory terminology for one-dimensional measurement that differs from that of BS5930: 1999 and ISRM (1978), as illustrated in Table 4 B8.1 (see below).

Table 4 B8.1 Terms for the description of aperture.

Aperture size term	*ISRM 1978*[1]	*BS5930 1999*	*ISO 14689–1: 2003*
<0.1mm	Very tight	Very tight	Very tight
0.1–0.25mm	Tight	Tight	Tight
0.25–0.5mm	Partly open		Partly open
0.5–2.5mm	Open	Moderately open	Open
2.5–10mm	Moderately wide	Open	Moderately wide
10–100mm	Very wide	Very open	Wide
100–1,000mm	Extremely wide		Very wide
>1,000mm	Cavernous		Extremely wide

[1] In detail, there is further confusion in that ISRM also defines a term *wide* for *gapped* features >10mm; the other terms above, also for apertures >10mm are for *open* features but the difference is not fully obvious.

Hydraulic aperture vs. mechanical aperture

For completeness, it is worth emphasising here that even if we could measure mechanical aperture meaningfully, the actual associated flow characteristics of the rock mass (hydraulic aperture) would be very difficult to estimate or predict. It clearly makes sense to observe and characterise rock masses as best we can, with respect to openness of the fracture network, but hydraulic conductivity can only be measured realistically using field tests, as discussed elsewhere, and even these are often open to different interpretations (e.g. Black, 2010).

Conclusions

After 25 years to digest the ISRM discussion and intensive international experience on research in measuring gaps in discontinuities and associated fluid flow, especially with respect to nuclear waste disposal investigations, the requirement for site investigation in Europe is a new set of linear measurements that are inconsistent with previous ones. No mention is made of the difficulty of characterising aperture in this way. Meanwhile, the New Zealand Geotechnical Society (2005) has produced yet another classification for aperture, which uses a selection of the same terms as in the above table but defined differently (e.g. wide = 60 mm to 200 mm) and introduces a new set of classes for the middle range: very narrow, narrow, moderately narrow.

Apologies

Apologies for being so critical, but it seems to this author that many codes and classifications oversimplify geological description and constrain/stifle good practice. This is especially so where it is mandated that some particular but fundamentally inadequate terminology shall be used. Unfortunately, inexperienced geotechnical engineers and engineering geologists are led to believe that such codification adequately deals with description and characterisation of the feature, which is not the case.

4.9 Down-hole logging

Down-hole logging technology has largely come from the oil industry and partly from mining. At the simplest level, a TV camera or borehole periscope is lowered down an uncased borehole and used to identify defects or to examine discontinuities. A borehole can be pumped dry of water and observations made of locations of water inflow, although this might need to be inferred from temperature or chemical measurements (Chaplow, 1996). Borehole impression packers were introduced in the 1970s and can be used to measure the orientation of discontinuities. Using an inflatable rubber packer, paraffin wax paper is pressed against the walls of the borehole and when retrieved, the traces of indented joints are clearly visible (Figure 4.37). Dip of the joints is easily determined from the geometry of the borehole but measuring direction relies upon whatever device is used to orientate the packer and, from experience, this can be a major source of error. It is good practice when using the impression packer to specify overlapping sections of measurement down the hole (by perhaps 0.5 m) so that consistency can be checked. In one borehole we found a 70 degree difference between consecutive sections, resulting from the packer being deflated before the compass had set in position – the contractor was asked to redo the work. A more modern tool is the Borehole Image

Figure 4.37 Impression packer. Paraffin wax paper has been pushed against the walls of the borehole by a rubber inflatable packer. A series of pale-grey traces can be seen, which represent a set of fairly planar joints dipping at about 70 degrees. Direction is obtained from a compass set in glue at the base of the packer. Other options for orienting devices now include flux gate magnetometers and gyroscopes.

Processing System (BIPS), which gives a continual visual record of the borehole wall (Kamewada *et al*., 1990). The tool is lowered down the borehole and a video camera takes a 360 degree image millimetre by millimetre down the hole through a conical mirror (Figure 4.38). Despite modern instrumentation for this tool, whereby azimuth can be measured by magnetic flux gates or gyroscopes, studies have revealed errors of up to 20 degrees in this measurement (Döse *et al*., 2008). Care must also be taken in interpretation of discontinuities logged in boreholes, especially if boreholes are all vertical. There will be obvious bias to the measurements – steep joints will be undersampled in vertical boreholes. As an example, during the Ching Cheung Road landslide investigation (Halcrow Asia Partnership 1998a), BIPS measurements were taken in vertical boreholes and

Figure 4.38 Output from BIPS down-hole discontinuity orientation device, being used during logging of rock core, Taejon Station, South Korea.

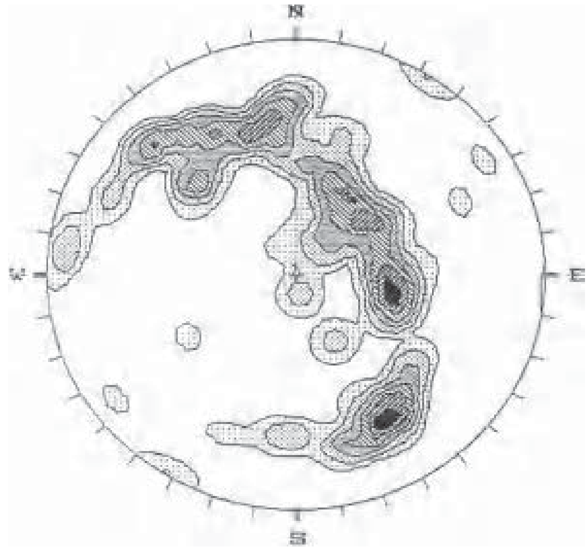
indicated a completely different style of jointing to those measured in exposed faces (essentially along horizontal scan lines). The data are presented in Figure 4.39 and it can be seen that the borehole data essentially defined a girdle of joints at 90 degrees to the main pole concentration that was measured from the horizontal scan line data. Both sets of data were required to provide the correct geological picture.

Other down-hole tools include resistivity and gamma ray intensity (even in cased holes) which, whilst often useful for oil exploration and coal mining, generally have rather limited application to civil engineering, other than possibly for locating clay-rich horizons.

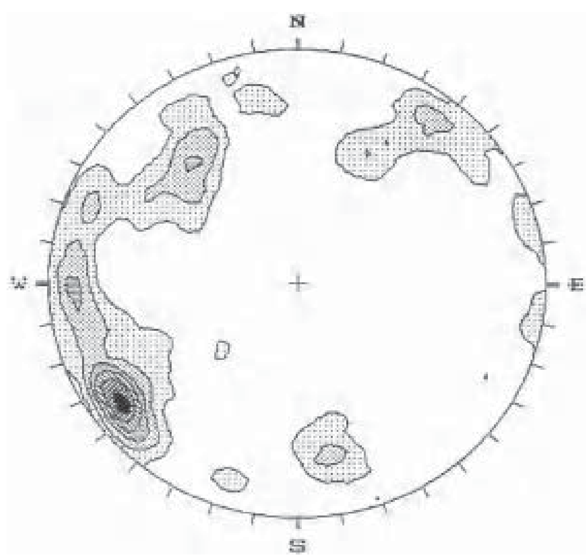
4.10 Instrumentation

Instrumentation is used to establish baseline ground conditions at a site, most commonly in terms of natural groundwater fluctuations. It is also used to monitor changes at a site brought about by construction activities such as excavation or blasting. Instrument systems need to be designed carefully so that they are reliable; there needs to be built-in redundancy for instruments that may fail or become damaged by site works or by vandalism. Incoming data must be readily interpretable if some action is to be taken as a consequence. Instruments are often used during the works to check performance against predictions. Displacements and water levels can be monitored and compared to those anticipated. First (ALERT) and second (ALARM) level trigger conditions can be defined with prescribed action plans. Data can be sent remotely to mobile phones or by email to engineers who have the responsibility for safety and the power to take action such as closing a road or evacuating a site. Other instruments that might be

Figure 4.39 Comparison between discontinuity data recorded by BIPS (vertical drillhole) and from surface mapping (horizontal scan lines). After Halcrow Asia Partnership (1998a).



Discontinuities measured in borehole (vertical)



Discontinuities measured in scanlines

employed during a large construction project include sound and vibration meters, especially where blasting is to be carried out.

Piezometers are commonly installed as part of ground investigations to measure water pressures. Detailed information on these and other instruments are given by Dunnicliff (2003). The simplest device is an open-tube standpipe with a porous tip, installed in a

Figure 4.40 Standpipe piezometer tip about to be placed in borehole. Another has already been installed at a deeper level. It is not very good practice to install more than one piezometer in a borehole, because of potential leakage between the different horizons being monitored, but can work providing great care is taken in installation. Portsmouth dry dock, UK.

sand pocket within the borehole, as shown in Figure 4.40. There are also push-in versions available. The water level in the standpipe is dipped, perhaps on a weekly or monthly basis, using a mechanical or an electronic device lowered down the hole; for an electric dipmeter, the water closes a circuit to activate a buzzer. To measure high rises in water level between visits by monitoring personnel, Halcrow plastic buckets can be installed on fishing line with a weight at the bottom of the string, at perhaps 0.5 m intervals down standpipes. The buckets are pulled out of the hole when the site is visited – the highest one that is filled with water indicates the maximum level of water (Figure 4.41). At a more sophisticated level, standpipes can be set up so that readings are taken automatically at regular intervals using pressure transducers (divers) or through an air bubbler system (Pope *et al.*, 1982). Data can be recorded on data loggers that can be set up to transmit

Figure 4.41 Halcrow buckets retrieved at Yee King Road landslide investigation (Hencher *et al*., 2008). These are unusual in that they contain sediments (from turbulent flows down the borehole). Normally, they would just contain water (or not), indicating the highest level that the water has risen in the borehole between inspections. Left side bucket is attached by fishing line to lead weights used to lower the buckets down the borehole.

information by telemetric systems. Other instruments include pneumatic or vibrating wire piezometers that respond very quickly to changes in pressure (Figure 4.42). Because they require almost no water flow to record change of pressure (unlike a standpipe), they can be grouted in place in the borehole and several instruments can be installed in the same hole, which can save cost (Vaughan, 1969; Mikkelsen & Green, 2003).

Instruments that are used to measure displacement include strain gauges, tilt meters, inclinometers and extensometers. They can be mechanical or electrical, for example, using vibrating wire technology. Figure 4.43 shows the end of an extensometer anchored deep behind the working face of a large copper mine in Spain and fitted with lights and a claxon horn to give warning if

Figure 4.42 Pneumatic piezometer being used to take measurements of rapidly changing water pressures during pile driving. Only small volume changes are necessary to measure pressure changes, so readings could be taken every ten seconds or so. Water pressures measured went off scale at about three times overburden pressure (Hencher & Mallard, 1989), Drax Power Station, Yorkshire, UK.

Figure 4.43 Extensometer with claxon and flashing lights used to warn workers at Aznalcollar mine, Spain, of danger from moving slope.

Figure 4.44 Exhumed inclinometer tubing. Four grooves inside (ridges outside) are guides for the wheels on the inclinometer instrument. The device with arms is a spider, which becomes fixed in position against the walls of a borehole whilst the tube can pass up or down inside. It is magnetic and a probe down hole can locate it and measurements can be made of settlement (as well as inclination).

the anchored point moves towards the mining area. Other instruments used to monitor performance at that site included deep inclinometers and a Leica total station, whereby numerous targets on the slope surfaces were surveyed remotely and automatically on an hourly basis, with the data sent to the site office (Hencher *et al.*, 1996). An inclinometer is a tubular torpedo (with wheels), which is lowered down a grooved tube set into a borehole or built into embankment fill. Figure 4.44 shows a section of inclinometer casing with the two sets of orthogonal grooves for the wheels. The torpedo (Figure 4.45) is first lowered down aligned by the first set of grooves, then removed and lowered down the second set of grooves. The section on the figure also has magnetic spiders with magnets, through which the tube can slide and can therefore be used to monitor vertical settlement where the tube is installed in fill. Strain gauges within the torpedo measure tilt, which is recorded against depth. The orthogonal measurements can be resolved to give the true direction and amount of displacement.

4.11 Environmental hazards

4.11.1 General

Site investigation needs to include a review of the potential environmental hazards as well as the immediate ground conditions. There may be risk from natural landslides and rockfall threatening the project, potential for natural subsidence or collapse (say in

Figure 4.45 Inclinometer torpedo about to be lowered down grooved tube. Tuen Mun Highway, Hong Kong.

areas underlain by salt deposits, old mine workings or karst), coastal erosion, wind, rain or earthquakes (Bell, 1999). As noted earlier, for some locations there are published hazard maps, but such maps cannot usually be relied upon on a site-specific scale. It is up to the site investigation team to identify the potential hazards for the project throughout its life (maybe 50 to 100 years) and to quantify these. In some cases, such an assessment might lead to a decision not to proceed with a project. Elsewhere, the hazard can be dealt with by careful design, and the main example of so doing is the hazard of earthquakes.

4.11.2 Natural terrain landslides

Landslides from natural terrain (rather than man-made slopes) are a hazard in most mountainous regions and can range from minor rock and boulder falls to massive landslides which involve >20 million m^3 of rock and occur on average every three or four years worldwide (Evans, reported by Eberhardt *et al.*, 2004). Landslides like the one that destroyed Yungay, Peru, in May 1970, and killed about 20,000 people, are very difficult to predict and impossible to engineer. All one can do is identify the landform, the degree of risk and perhaps monitor displacements or micro-seismicity, with a plan to evacuate people and close roads if necessary.

Smaller and more common natural terrain landslides can be predicted and mitigated to some degree by engineering works. The starting point is generally historical records of previous landslides, such as incidents on active roads through mountainous regions. These may allow areas of greatest hazard to be identified and some prioritisation of works. It should be noted, however, that small rockfalls at one location can be indicative of much larger and deep-seated landslides, and minor incidents should be reviewed in this light. Where there is good historical air photograph coverage, sources of landslides can be identified and these correlated to susceptibility maps prepared using geographical information systems (e.g. Devonald *et al*., 2009). Typical factors that might be linked to probability of landslide occurrence include geology, thickness of soil, vegetation cover, slope angle, proximity to drainage line and catchment area. Once a best fit has been made linking landslide occurrence to contributing factors, maps can be used in a quantitative, predictive way. Consequence of a landslide depends on location relative to the facility at risk (e.g. road, building), volume, debris run-out, possibility of damming a watercourse and eventually impact velocity. From studies in Hong Kong (Moore *et al*., 2001; Wong, 2005), it is apparent that the greatest risk is generally from channelised debris flows (outlets of streams and rivers) and to facilities within about 100 m of hazardous slopes (the typical limit of debris run-out in Hong Kong). A broader discussion is given by Fell *et al*. (2005). A decision can be made on the resources that are justified to mitigate the hazard, once one has determined the level of risk (which can be quantified in terms of risk to life). There are many options, including barriers and debris brakes in stream courses and catch nets, especially for rockfall and boulder hazards. In some cases, a decision might be made to stabilise the threatening natural terrain using drainage, surface protection, netting and anchors, as for man-made slopes, dealt with in Chapter 6.

4.11.3 Coastal recession

Coastal recession is a common problem and rates can be very rapid. For example, parts of the Yorkshire coast are retreating at up to 2 m per year (Quinn *et al*., 2009). Many studies have been carried out on mechanisms, but the harsh fact is that many properties and land near the coast are at risk and many houses have to be abandoned. Coastal protection measures can be designed successfully but these sometimes fail in a relatively short time and, constructing works at one location, can have consequences for others along the coast, as suspected for the damage to the village of Hallsands in Devon, which had to be largely abandoned (Tanner & Walsh, 1984).

4.11.4 Subsidence and settlement

An excellent review on ground subsidence – natural and due to mining, is given by Waltham (2002). Ground subsidence occurs naturally due to lowering of the water table from water extraction, oil and gas extraction, shrinkage of clay, and dissolution of salt deposits, limestone and other soluble rocks (e.g. Cooper & Waltham, 1999). Subsurface piping can occur associated with landslides in any rocks, including granite (Hencher *et al.*, 2008). The results can be dramatic, with sudden collapses of roads or even loss of buildings. Care must therefore be taken to consider these possible hazards during site investigation.

Underground mining dates back thousands of years in some areas (e.g. flints from chalk) and on a major scale for hundreds of years. Consequently, there are very incomplete records. In desk study, the first approach will always be to consult existing records and documents, but wherever there is some resource, such as coal, that might have been mined, the engineering geologist needs to consider that possibility. Investigations can be put down on a pattern, specifically targeted at the suspected way that mining might have been carried out (pillar and stall or bell pit, for example). Air photograph interpretation will often be useful and geochemical analysis of soil can give some indication of past mining activities.

4.11.5 Contaminated land

Many sites around the world are severely contaminated, often because of man's activities. This means that if the site is to be used for some new purpose, it may need to be cleaned up to be made habitable. Similarly, when constructing near or through possibly contaminated land, this needs to be investigated and the contamination mitigated, possibly by removing the contaminated soil to a treatment area. Barla & Jarre (1993) describe precautions for tunnelling beneath a landfill site. Guidance on investigation is given in BSI (2001), CIRIA (1995) and many other sources of information are given by the AGS (Appendix A). Sometimes the contamination is dealt with at site. Desk study can often identify projects where there are severe risks because of previous or current land use. Industrial sites such as old gas works, tanneries, chemical works and many mines are particularly problematical. Severe precautions need to be taken when dealing with such sites and works will probably be controlled by legislation.

4.11.6 Seismicity

4.11.6.1 Principles

Design against earthquake loading is an issue that needs to be considered in many parts of the world, depending upon the importance of the project and risks from any potential damage. In some locations, because of inherently low historical seismicity (UK) or severity of other design issues (e.g. typhoon wind loading in Hong Kong), seismicity might be largely ignored for design other than for high-risk structures like nuclear power plants. Elsewhere, seismicity needs to be formally assessed for all structures and taken into account for design.

4.11.6.2 Design codes

Many countries have design codes for aseismic design and these are generally mandatory. Nevertheless, it is often prudent to carry out an independent check and in particular to consider any particular aspects of the site that could affect the impact of an earthquake. For example, the local soil conditions might have the potential to liquefy. These issues are considered in more detail in Chapter 6.

Design codes, where well written and implemented, reduce the earthquake risks considerably. The USA, for example, has a high seismic hazard in some areas but fatalities are few and this can be attributed to good design practice and building control. China also has a high seismic hazard in some areas, but earthquakes commonly result in comparably large loss of life, which might be attributed to poor design and quality of building. Structures can be designed to withstand earthquake shaking, and even minor improvements in construction methods and standards of building control (quality of concrete, walls tied together, steel reinforcement, etc.) can prevent collapse and considerably reduce the likely loss of life (Coburn & Spence, 1992).

4.11.6.3 Collecting data

The first stage is to consider historical data on earthquakes, which are available from many sources, including the International Seismological Centre, Berkshire, and the US Geological Survey. These historical data can be processed statistically using appropriate empirical relationships to give probabilistic site data – for example, of peak ground acceleration over a 100 or 1,000-year period. This can be done by considering distance from site of each of the historical earthquake data or linked to some source structure (such as possible active faults). Dowrick (1988) addresses the process well, and some guidance is presented in Chapter 6. In some cases, estimates are made of the largest earthquake that might occur within the regional tectonic regime and similar regimes around the world, to derive a maximum credible event. This postulated worst case could be used by responsible authorities for emergency planning and is also used for some structures – a safe-shutdown event for a nuclear power station design.

4.12 Laboratory testing

Generally, a series of laboratory tests are specified for samples recovered from boreholes, trial pits and exposures, often employing the same GI contractor who carried out the boring/drilling. Geotechnical parameters and how to measure or estimate them are addressed in Chapters 5 and 6.

4.13 Reporting

The results of site investigation are usually presented as factual documents by the GI contractor – one for borehole logs, a second for the results of any laboratory testing. In addition, specialist reports might be provided on geophysics and other particular investigations. These reports may include some interpretation, perhaps with some cross sections if the contractor has been asked to do so, but such interpretation may be rather general and unreliable, not least because the GI contractor will not be aware of the full details of the planned project.

Generally, it is up to the design engineer to produce a full interpretation of the ground model in the light of his desk study, including air photo interpretations and the factual GI (that he has specified). This might be done supported by hand-drawn cross sections and block diagrams – which should ensure that the data are considered carefully and should enable any anomalies and errors to be spotted. There is a tendency now to rely upon computer-generated images, with properties defined statistically to define units (e.g. Culshaw, 2005; Turner, 2006), which might reduce the chance that key features of the model are properly recognised by a professional.